सत्यमेव जयते

MANUAL ON SEWERAGE AND SEWAGE TREATMENT SYSTEMS

PART A: ENGINEERING

THIRD EDITION - REVISED AND UPDATED

MINISTRY OF URBAN DEVELOPMENT, NEW DELHI

http://moud.gov.in

CENTRAL PUBLIC HEALTH AND ENVIRONMENTAL ENGINEERING ORGANIZATION

IN COLLABORATION WITH

JICA

JAPAN INTERNATIONAL COOPERATION AGENCY

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In keeping with the advancements in this sector, updates as and when found necessary will be hosted in the Ministry website: http://MoUD.gov.in/ and the reader is advised to refer to these also.

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शहरी विकास एवं संसदीय कार्य मंत्री
भारत

**MINISTER OF URBAN DEVELOPMENT
AND PARLIAMENTARY AFFAIRS
INDIA**

MESSAGE

While the population of urban areas has increased from 19.9% to 31.2% between 1971 and 2011, the contribution of urban areas to GDP growth has shown a phenomenal increase from 38% to 60% over the same period. Providing sanitation and hygiene to a growing population of more than 1.21 billion with higher aspiration levels is a major challenge. This increase in population has created a significantly enhanced demand on water supply, health, hygiene and environmental sanitation.

To tackle this, the Government of India has initiated programs and given policy directions to States and Cities through interventions like the launch of the Jawaharlal Nehru National Urban Renewal Mission (JnNURM) and adoption of National Urban Sanitation Policy, 2008. JnNURM seeks to promote cities as engines of economic growth through improvement in the quality of urban life by facilitating the States for creation of quality urban infrastructure, with assured service levels and efficient governance.

The National Urban Sanitation Policy (NUSP), 2008 pertains to management of human excreta and associated public health and environmental impacts, including 100% sanitary and safe disposal of human excreta and liquid wastes from all sanitation facilities like sewers and toilets.

I am confident that the revised and updated manual in three parts - Engineering, Operation & Maintenance and Management will further enable the practicing professionals in design and operation & maintenance of the sewerage and sewage treatment systems economically, efficiently and effectively.

I would like to acknowledge the support extended by the Japan International Cooperation Agency (JICA), Government of Japan and also the efforts of the officials of MoUD in this endeavour. I am hopeful this effort would contribute towards achieving the Ministry's vision of the creation of economically vibrant, inclusive, efficient and sustainable urban habitats.

(KAMAL NATH)

डॉ. सुधीर कृष्ण
Dr. Sudhir Krishna

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FOREWORD

India is passing through a dynamic phase of development wherein the Government of India is striving hard to provide all the necessary infrastructure facilities to urban population in order to achieve sustainable economic growth. As per the 2011 census, the share of urban population is 31.2% as against 28% of 2001 census of the total population of the country which is expected to be 50% by the mid of the century. Infrastructure facilities being provided for such an unprecedented growth are unable to meet with the requirement due to various compelling circumstances. Water supply and sanitation is one of the basic infrastructure facilities, which has a direct impact on the urban population to meet the desired levels of quantity and quality.

Inadequate and unsafe water supply and sanitation services have a direct effect on the health of the community and an indirect effect on the economy of the country.The report on “The Economic Impact of Inadequate Sanitation in India” released by the Water and Sanitation Program (WSP), World Bank states that inadequate sanitation costs India almost US$ 54 billion (about Rs. 2.7 lakh crore) or 6.4% of country’s GDP in 2006. In view of this huge cost to be paid for inadequate sanitation, it is really necessary on all the concerned agencies dealing with water supply and sanitation sector in the country including the community to find ways for how best this loss to the nation could be minimized.

I appreciate the cooperation extended by the Japan International Cooperation Agency (JICA) and the Government of Japan through their financial and expert support in completing this task of Revision and Updating of the Manual on Sewerage and Sewage Treatment Systems, which was last published by the Ministry during 1993.Untiring efforts of the experts from JICA Study Team and India culminated in bringing out such an exhaustive manual in three parts, is worthy of appreciation.

I am confident that the three parts of the manual will certainly achieve the program objectives of the Government of India as stated in the “National Urban Sanitation Policy” adopted in 2008. I also sincerely hope that this Manual would serve as a guide to policy makers, planners, and all practicing professionals in the field of sewerage and sewage treatment systems so as make the systems economically viable to accrue benefits in the long term on a sustainable basis.

Finally, I would like to acknowledge the untiring efforts of all people who are associated with the task of accomplishing the commendable job of formulation of this exhaustive manual for the benefit and improvement of the sanitation sector.

19.11.2013

(Sudhir Krishna)

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PREFACE

Over the years, there has been continuous migration of people from rural and peri-urban areas to cities and towns. The proportion of population residing in urban areas has increased from 28.0% in 2001 to 31.2% in 2011. The number of towns has increased from 5161 in 2001 to 7935 in 2011. The uncontrolled growth in urban areas has left many Indian cities and towns deficient in infrastructural services such as water supply, sewerage & sanitation, storm water drainage and solid waste management.

Sewerage and sewage treatment is a part of public health and sanitation, and according to the Indian Constitution, falls within the purview of the State List. Since this is non-exclusive, non-rivalled and essential, the responsibility for providing the services lies within the public domain. The activity being local in nature, it is entrusted with the Urban Local Bodies. The Urban Local Body undertakes the task of sewerage and sewage treatment service delivery, with its own staff, equipment and funds. In few cases, part of the said work is contracted out to private enterprises.

Cities and towns which have sewerage and sewage treatment facilities are unable to cope-up with the increased burden of providing such facilities efficiently to the desired level. Issues and constraints that are encountered by the urban local bodies, responsible for providing sewerage and sanitation facilities, are compounded due to various reasons. The main cause of water pollution is the unintended disposal of untreated, partially treated and non-point sources of sewage and more important is its effect on human health & environment.

While the conventional sewerage is an effective system for sewage collection, transportation and treatment, it also remains as highly resource-inefficient in terms of technology. Consequently, high capital and recurrent costs for the O&M of this system at a significant level, prohibits its widespread adoption in all sizes of urban areas in the country.

As per the 2011 Census, only 32.7% of urban households are connected to a piped sewer system whereas 38.2% dispose of their wastes into septic tanks and 8.8% households are having pit latrines (single & double, etc.) and 1.7% of households are having other latrines (connected to open drains, night soil removed by human etc.). About 18.6% of urban households still do not have access to individual toilets – about 6.0% use public /community toilets and 12.6% are forced the indignity of open defecation.

According to the report on the Status of Wastewater Generation and Treatment in Class-I Cities and Class-II towns of India, December 2009 published by Central Pollution Control Board,

Continued

the estimated sewage generation from 498 Class-I cities and 410 Class-II towns (Population estimated for 2008 based on 2001 census) together is 38,524 MLD, out of which only 11,787 MLD (30.5%) is being treated with a capacity gap of 26,737 MLD.

The National Urban Sanitation Policy (NUSP) adopted by the Ministry of Urban Development in 2008 envisions"All Indian Cities and towns become totally sanitised, healthy and liveable and ensure and sustain good public health outcomes to all their citizens, with a special focus on hygienic and affordable sanitation facilities for the urban poor and women". With a view to promote sanitation very rapidly in urban areas of the country and also to recognise the excellent performance in this sector by the cities, the Government of India has instituted an annual award scheme for rating of the cities on certain selected sanitation parameters. The overall goals of NUSP, is to transform the urban sanitation into community driven, totally sanitized, healthy and liveable.

The Millennium Development Goals (MDGs) enjoins upon the signatory nations to extend access to improved sanitation to at least half the urban population by 2015, and 100% access by 2025. This implies extending coverage to households without improved sanitation, and providing proper sanitation facilities in public places to make cities and towns free of open defecation. The Ministry proposed to shift the focus on infrastructure in urban water supply and sanitation (UWSS) to improve the service delivery and formulated in 2008 a set of Standardized Service Level Benchmarks for UWSS as per International Best Practice & brought out the "Handbook on Service Level Benchmarking" on water supply and sanitation.

The Manual on Sewerage and Sewage Treatment (second edition) published in 1993 mainly gave thrust to engineering aspects of the sewerage and sewage treatment systems. The topics additionally covered in the current revised and updated revision are emphasis on O&M and management of sewerage and sewage treatment systems, not dealt with in detail in the earlier edition and are to create awareness amongst the practicing and field engineers on the importance of sustainability of the systems in the long-term. The present Manual on Sewerage and Sewage Treatment Systems has been divided into three parts, as Part – A on 'Engineering', Part – B on 'Operation and Maintenance', and Part – C on 'Management'.

On behalf of the Ministry I would like to highly appreciate and acknowledge the financial and physical support provided by the Japan International Cooperation Agency (JICA), Government of Japan for the preparation of this exhaustive and informative manual.

The Ministry of Urban Development places on record its appreciation of the Expert Committee for the revision and updating of the Manual on Sewerage and Sewage Treatment Systems and the untiring services rendered by Dr. M. Dhinadhayalan, Joint Adviser (PHEE) & Member Secretary of the Expert Committee who acted as the fulcrum between the Ministry of Urban Development, GOI and the Japan International Cooperation Agency (JICA) to maintain an extremely balanced relation throughout the period of preparation of the Manual so as to accomplish the task.

I also extend my thanks to all those people who were directly or indirectly instrumental in giving such a praise-worthy shape to the manual

(Ashok Singhvi)

Dr. M. DHINADHAYALAN
Joint Adviser (PHEE)
CPHEEO

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ACKNOWLEDGEMENT

Ever since the publication of the Manual on Sewerage and Sewage Treatment in 1993 a number of new developments and changes have occurred in the range of technologies for on-site and off-site sanitation systems, including collection, transportation, treatment and reuse of treated sewage & sludge for various uses during the last two decades. While revising the Manual a broad approach was adopted for the need for revision and updating of the manual on the three important aspects, such as i) Engineering, ii) Operation & Maintenance, and iii) Management of sewerage and sewage treatment systems. Additional topics on operation & maintenance and management were added so as to create awareness amongst the practicing and field engineers regarding the importance of these two topics for the long-term sustainability of the systems.

The revision and updating of the existing manual (1993), aims to meet the important requirement of providing advice on the technology options for urban sanitation, for the new infrastructure or upgrading of existing services. It is applicable both for small interventions in specific locations and for larger programs that aim to improve sanitation on a citywide scale. The manual would help the practitioners in the selection of technologies with various options for providing techno-economic solutions keeping in view the health of the community and safeguarding the environment so as to provide a wide range of options to the planners and designers.

The National Urban Sanitation Policy (NUSP) was adopted by the Ministry of Urban Development (MoUD) in 2008. It envisions that “All Indian cities and towns become totally sanitized, healthy and liveable and ensure and sustain good public health and environmental outcomes for all their citizens with a special focus on hygienic and affordable sanitation facilities for the urban poor and women”. With a view to promote sanitation very rapidly, in urban areas of the country and also to recognise the excellent performance in this sector by the cities, the Government of India (GOI) instituted an annual award scheme for rating of cities on certain selected sanitation parameters. The overall goal is to transform Urban India into community driven, totally sanitized, healthy and liveable cities and towns.

In view of the importance attached and impetus given to sanitation by the GOI in cities and towns of the country, the MoUD decided to revise and update the existing Manual on Sewerage and Sewage Treatment under the aegis of Japan International Cooperation Agency (JICA), who appointed a JICA Study Team (JST) in July 2010 comprising of experts from Japan.

The JST visited about 40 Sewage Treatment Plants across 8 States during 2010-2011 and gathered first-hand experience on planning, implementation and O & M of sewerage systems and factual knowledge on the social, engineering, financial and management issues relevant to India. The JST retained an Indian Study Team (IST) to assist in the preparation of the manual.

Continued

The MoUD constituted 3 Expert Committees (ECs) (Annex-1) (1st & 2nd in August 2010 and the 3rd in November 2011) by nominating experts from Central Ministries / Departments, academic & research institutions, senior engineers from State Departments & Utilities for reviewing and finalizing the drafts of the JST. Two numbers of each one week long study tours were conducted in November, 2011 and January 2012 by JST for the members of the EC to study the sewerage and sewage treatment systems in Japan. This helped the members of the EC to get the first hand information on the technologies adopted in sewerage and sewage treatment and how the sewerage systems are being operated and maintained. The tours were facilitated by JICA.

The ECs, JST and IST interacted in 16 meetings at New Delhi to give a final shape to all the three parts of the manual. The manuals prepared by the JST, ECs and IST address the following. :

Part – A on 'Engineering' addresses the core technologies and updated approaches towards the incremental sanitation from on-site to decentralized or conventional sewerage systems including collection, conveyance, treatment and reuse of the misplaced resource of sewage and sludge and is simplified to the level of the practicing engineer for the day-to-day field guidance in understanding the situation and coming out with a choice of approaches to remedy the situation.

Part – B on 'Operation and Maintenance' addresses the issues of standardizing the human and financial resources. These are needed to sustain the sewerage and sanitation systems which are created at huge costs without slipping into an edifice of dis-use for want of codified requirements for O&M so that it would be possible to address the related issues. These financial and related issues are to be addressed at the estimate stage itself, thus enabling to seek a comprehensive approval of fund allocations and human resources. This would also usher in the era of public private partnership to make the projects self-sustaining. This also covers aspects such as guidelines for cleaning of the sewers and septic tanks besides addressing the occupational health hazards and safety measures of the sanitation workers.

Part – C on 'Management' is a refreshing approach to modern methods of project delivery and project validation and gives a continual model for the administration to foresee the deficits in allocations and usher in newer mechanisms. It is a tool for justifying the chosen project delivery mechanism and optimizing the investments on need based allocations instead of allocations in budget that remain unutilized and get surrendered in the end of fiscal year with no use to anyone.

These draft manuals were discussed with an All India audience in the 2 National Workshops held at New Delhi on 20th & 21st September 2012 for finalization of Part A: Engineering and on 21st & 22nd January 2013 for finalization of Part B: Operation & Maintenance and Part C: Management, where in delegates from Central Ministries, State Government Departments, Urban Local Bodies, Parastatal Agencies, and representatives from Technology Providers participated and deliberated in detail regarding the contents of each part of the three manuals. These were further reviewed and brought to completion by the Editorial committee constituted by the MoUD with members as in (Annex-2). In all, 6 meetings of the Editorial committee were held at New Delhi.

The Editorial Committee while editing the Manual kept in view the TOR prescribed by the Ministry and also comments, suggestions, views offered by the delegates who participated in National Workshops and views received through e-mail were also accommodated suitably wherever necessary in all the three parts of the manual.

Continued

The Expert Committee places on record its gratitude to:

- The MoUD for the necessary support & encouragement in the preparation of the manual
- The JICA for funding the meetings, study tours, workshops and publishing the manuals.
- The PHE Departments, Water & Sewerage Boards, Urban Local Bodies, and individuals for their valuable suggestions on the draft of the manual.

The Expert Committee is highly indebted to Mr. Akira Takechi, JICA Study Team Leader for his wonderful guidance, whole hearted support and encouragement of the members of the Expert Committee during the entire period in fulfilling the task of preparation of the Manual.

The Expert Committee expresses its gratitude to Dr.S.Sundaramoorthy, Consultant, JST, Team Leader IST & Member Secretary, Editorial Committee as the architect of the manual and his team for giving final shape to all the three parts of the Manual.

I would like to extend my sincere thanks to Dr.S.R.Shukla, Former Adviser (PHEE), CPHEEO, MoUD, Co-Chairman of the Expert Committees, for chairing all the Expert / Editorial Committee meetings and for his continued involvement, guidance and support in preparation and finalization of three parts of the manual.

I express my sincere thanks and gratitude to Ms.E.P.Nivedita, then Director (LSG), for taking the initial efforts through coordination and chairing the deliberations of the EC meetings in laying a broader framework for revision and updation of the Manual.

I am also privileged to express my sincere thanks on behalf of the Expert Committee to Ms.Veena Kumari Meena, then Director (LSG) and Ms.Nandita Mishra, Director (PHE) for their support in finalization of the Manual.

The contribution made by Mr.V.K.Chaurasia, Joint Adviser (PHEE), Mr.J.B.Ravinder, Deputy Adviser (PHE), Mr. A.K. Saha, Assistant Adviser (PHE), and Dr.Ramakant, Assistant Adviser (PHE), CPHEEO for enriching the contents of the Manual is very much appreciated. The painstaking efforts taken by Dr.Ramakant, Assistant Adviser (PHE) and Member-Coordinator, during the entire process of preparation of the Manual is stupendous and laudable.

A special mention and deep appreciation is due for the meticulous and diligent efforts of Dr.S.Saktheeswaran, Editorial Consultant (JICA) & Copy-Editor to JICA for bringing out the manual in a concise form, through several stages of editing and incorporating all the feedbacks.

The help and contribution by Mr. Takashi Sakakibara, JICA expert, CPHEEO and Mr.C.Krishna Gopal, Consultant, NUSP Cell is highly appreciated.

The Committee also acknowledges the contribution and support of the representatives of SMEC India Pty Ltd for the first phase of the study and CH2M HILL for the second phase of the study for their excellent logistics support and facilitation throughout the period.

I would like to acknowledge all those connected individuals, organizations, institutions, Bilateral and Multilateral agencies for their efforts directly and indirectly, through their valuable contribution, suggestions and inputs.

(M. Dhinadhayalan)

Annex-1
JICA Study Team (JST)

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5	Shri I Kalyanasundaram	Sewerage Structures
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Annex – 1 (continued)
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Annex – 1 (continued)
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The list of Editorial Committee members is in the next page

Annex -2

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Editorial Committee-Part-B and Part-C

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ABBREVIATIONS

AA	Aguas Argentinas
AAI	Airports Authority of India
ABACUS	Advanced Billing and Collection Utility System
AC	Alternating Current
ADB	Asian Development Bank
AE	Assistant Engineer
AL	Aerated Lagoon
AM	Alandur Municipality
AM	Asset Management
AMF	Automatic Mains Failure Panel
ArSP	Alandur Sewerage Project
ASEM	Advisory Services in Environmental Management
ASP	Activated Sludge Process
ASR	Average Survival Ratio
AU	Anna University
AWS	Automatic Weather Station
BCP	Business Continuity Plan
BIS	Bureau of Indian Standards
BOD	Biochemical Oxygen Demand
BOO	Build Own Operate
BOOT	Build Own Operate Transfer
BOQ	Bill of Quantities
BOT	Build Operate Transfer
BSNL	Bharat Sanchar Nigam Limited
BTO	Build Transfer Operate
BWSSB	Bangalore Water Supply and Sewerage Board
CA	Constitutional Amendment
CAA	Constitutional Amendment Act
CAD	Computer-Aided Design
CB	Circuit Breaker
CBO	Community-based Organizations
CCS	Centre for Civil Society
CCTV	Closed Circuit Television
CD	Compact Disc
CE	Chief Engineer
CETP	Common Effluent Treatment Plant
CFC	Central Finance Commission
CFD	Computational Fluid Dynamics
CFL	Compact Fluorescent Lamp
CFR	Continuous Fed Reactor

CI	Cast Iron
CI	Context Information
CII	Confederation of Indian Industry
CMP	City Master Plan
CMWSSB	Chennai Metropolitan Water Supply and Sewerage Board
COD	Chemical Oxygen Demand
CPCB	Central Pollution Control Board
CPCL	Chennai Petroleum Corporation Limited
CPHEEO	Central Public Health and Env.Engineering Organization
CPL	Community Participation Law
CPM	Critical Path Method
CRZ	Coastal Regulation Zone
CS	Chemical Sludge
CSER	Corporate Social and Environmental Responsibility
CSIR	Council of Scientific and Industrial Research
CSO	Central Statistical Organization
CSP	City Sanitation Plan
CSR	Corporate Social Responsibility
CSS	Centralized Sewerage System
CTE	Consent to Establish
CTO	Consent to Operate
DAF	Dissolved Air Floatation
DALY	Disability Adjusted Life Year
DANIDA	Danish International Development Agency
DC	Direct Current
DCB	Demand Collection Balance
DCS	Distributed Control System
DDC	Direct Digital Control
DEA	Double Entry Accounting
DFID	Department for International Development
DG	Diesel Generator
DI	Ductile Iron
DMF	Dual Media Filters
DMHQ	Disaster Management Head Quarter
DMP	Disaster Management Plan
D O	Dissolved Oxygen
DOL	Direct On Line
DPC	District Planning Committee
DPR	Detailed Project Report
DWF	Dry Weather Flow
DWMS	Decentralized Wastewater Management System

EAM	Enterprise Asset Management
EBB	Eco Bio Block
EC	Electrical Conductivity
EE	Executive Engineer
EIA	Environmental Impact Assessment
EMI	Equated Monthly Instalment
EPA	Environment Protect Agency
EPC	Engineering Procurement and Construction
ERP	Enterprise Resource Planning
ES	Effective Size
ESR	Electron Spin Resonance
EWS	Economically Weaker Section
F/M	Food to Microorganism
FAR	Floor Area Ratio
FBAS	Fixed Bed Biofilm Activated Sludge
FBR	Fed Batch Reactor
FC	Faecal Coliform
FFR	Fixed Film Reactor
FI	Funding Institution
FIDIC	International Federation of Consulting Engineers
FIRE	Financial Institutions Reform and Expansion
FL	Facultative Lagoon
FRC	French Water Club
FSI	Floor Space Index
GA	General Arrangement
GAC	Granular Activated Carbon
GDP	Gross Domestic Product
GI	Galvanized Iron
GIS	Geographical Information System
GL	Ground Level
GOI	Government of India
GOS	Group Operated Switch
GOTN	Government of Tamil Nadu
GSM	Global System for Mobile Communications
HDA	Haldia Development Authority
HDPE	High Density Polyethylene
HIA	Health Impact Assessment
HP	Horsepower
HPEC	High Powered Expert Committee
HPP	High Pressure Pump
HRD	Human Resources Development

HRT	Hydraulic Retention Time
HT	High Tension
HUDCO	Housing and Urban Development Corporation
HUF	Hindu Undivided Family
HVAC	Heating Ventilation and Air Conditioning
IDWSSD	International Drinking Water Supply and Sanitation Decade
IEC	Information, Education and Communication
IIPA	Indian Institute of Public Administration
IIT	Indian Institute of Technology
IL&FS	Infrastructure Leasing & Financial Services Limited
IP	Internet Protocol
IPR	Indirect Potable Reuse
IRC	Indian Road Congress
IISER	Indian Institute of Science Education & Research
IS	Indian Standard
ISDN	Integrated Services Digital Network
ISO	International Organization for Standardization
ISRO	Indian Space Research Organization
IT	Information Technology
ITI	Industrial Training Institutes
ITN	International Training Network
ITV	Industrial Television
IWA	International Water Association
IWWA	Indian Water Works Association
JE	Junior Engineer
JICA	Japan International Cooperation Agency
JnNURM	Jawaharlal Nehru National Urban Renewal Mission
JSWA	Japan Sewage Works Association
KAPS	Knowledge, Attitude, Practice, Study
KIADB	Karnataka Industrial Areas Development Board
KMDA	Kolkata Municipal Development Authority
Kva	Kilovolts ampere
KWA	Kerala Water Authority
LAN	Local Area Network
LBFL	Local Bodies Finance List
LCD	Liquid Crystal Display
LDPE	Low Density Polyethylene
LLDPE	Linear Low Density Polyethylene
LIC	Life Insurance Corporation

LoC	Letter of Credit
LPS	Liters per second
LT	Low Tension
M&E	Monitoring and Evaluation
mA	milliamps
MBBR	Moving Bed Biofilm Reactor
MBR	Membrane Bio Reactor
MCGM	Municipal Corporation of Greater Mumbai
MDG	Millennium Development Goals
MF	Micro Filtration
MFL	Maximum Flood Level
MIS	Management Information System
MLD	Million Litres per Day
MLE	Modified Ludzack Ettinger
MLIT	Ministry of Land, Infrastructure, Transport and Tourism
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
MMRDA	Mumbai Metropolitan Region Development Authority
MMSD	Milwaukee Metropolitan Sewerage District
MoEF	Ministry of Environment and Forests
MoF	Ministry of Finance
MoHA	Ministry of Home Affairs, India
MoHUPA	Ministry of Housing and Urban Poverty Alleviation
MoNRE	Ministry of New and Renewable Energy
MoSJE	Ministry of Social Justice and Empowerment
MoUD	Ministry of Urban Development
MoUHPA	Ministry of Urban Housing and Poverty Alleviation
MP	Member of Parliament
MPC	Metropolitan Planning Committee
MPLADS	Member of Parliament Local Area Development Scheme
MPN	Most Probable Number
MSDS	Material Safety Data Sheet
MSWM	Municipal Solid Waste Management
MWCI	Manila Water Company
MWSI	Manila Water Services
MWSS	Manila Metropolitan Waterworks and Sewerage Services
NABARD	National Bank for Agriculture and Rural Development
NCB-W2	World Bank's Contract for National Competitive Bidding
NCC	National Cadet Corps
NDITA	Nabadiganta Industrial Township Authority

NEERI	National Environmental Engineering Research Institute
NF	Nano Filtration
NGO	Non Governmental Organization
NIC	National Informatics Centre
NICNET	Nation-wide Informatics Network
NILIM	National Institute for Land and Infrastructure Management, Japan
NITIE	National Institute for Training in Industrial Engineering
NRC	National Research Council
NRCD	National River Conservation Directorate
NRCP	National River Conservation Plan
NRSA	National Remote Sensing Agency
NSKFDC	National Safai Karamachari Finance and Development Corporation
NSS	National Social Service
NTADCL	New Tiruppur Area Development Corporation Ltd.
NURM	National Urban Renewal Mission
NUSP	National Urban Sanitation Policy
O&M	Operation and Maintenance
OHP	Overhead Projector
PA	Poverty Alleviation
PC	Personal Computer
PCB	Pollution Control Board
PCOM	Per Capita O&M Costs
PDCA	Plan-Do-Check-Act
PE	Polyethylene
PERT	Programme Evaluation and Review Technique
PF	Power Factor
PHE	Public Health Engineering
PHED	Public Health Engineering Department
PI	Performance Indicator
PIL	Public Interest Litigations
PLC	Programmable Logic Controller
PMC	Pune Municipal Corporation
PP	Polypropylene
PPP	Public Private Partnership
PPPP	Public Private People Partnership
PSP	Private Sector Participation
PUB	Public Utilities Board
PVC	Poly Vinyl Chloride
PVC-U	Unplasticized Poly Vinyl Chloride
PWD	Public Works Department

R O	Reverse Osmosis
R&D	Research and Development
RBC	Rotating Biological Contactor
RBI	Reserve Bank of India
RCC	Reinforced Cement Concrete
RDBMS	Relational Database Management System
RIF	Rural Innovation Fund
RS	Return Sludge
RTU	Remote Terminal Unit
SAFF	Submerged Aeration Fixed Film
SAR	Sodium Absorption Ratio
SAT	Soil Aquifer Treatment
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
SCS	Survival Curve using Survey
SDE	Spatial Database Engine
SDES	State Directorates of Economics and Statistics
SE	Superintending Engineer
SECAP	System Evaluation and Capacity Assurance Plans
SEMIS	Sewerage Mapping and Information System
SFBR	Submerged Fixed Bed Reactor
SFC	State Finance Commission
SFRC	Steel Fiber Reinforced Concrete
SHG	Self-Help Group
SLB	Service Level Benchmark
SMC	Surat Municipal Corporation
SMS	Short Message Services
SOQ	Standards of Quality
SOR	Surface Overflow Rate
SPC	Special Purpose Company
SPCB	State Pollution Control Board
SPS	Sewage Pumping Station
SRT	Solids Retention Time
SS	Suspended Solids
STP	Sewage Treatment Plant
SWD	Side Water Depth
SWM	Solid Waste Management
SWMD	Sewerage and Wastewater Management Department, Japan
SXF	Secure eXchange Format file

TAIMS	Tokyo Advanced Information Management System
TCPO	Town and Country Planning Organization
TDS	Total Dissolved Solids
TNUDF	Tamil Nadu Urban Development Fund
TNUIFSL	Tamil Nadu Urban Infrastructure Financial Services Limited
TUFIDCO	Tamil Nadu Urban Finance and Infrastructure Dev. Corporation Ltd.
TWAD	Tamil Nadu Water Supply and Drainage Board
TWICL	Tamil Nadu Water Infrastructure Company Limited
UA	Urban Agglomeration
UASB	Upflow Anaerobic Sludge Blanket
UF	Ultra Filtration
UGD	Underground Drainage
UIDSSMT	Urban Infrastructure Dev.Scheme for Small & Medium Towns
ULB	Urban Local Body
UNEP	United Nations Environment Programme
UNHABITAT	United Nations Human Settlements Programme
UNICEF	United Nations International Childrens Emergency Fund
UPS	Uninterruptible Power Supply
USAID	United States Agency for International Development
USEPA	United States Environmental Protection Agency
UV	Ultra violet
UWSS	Urban Water Supply and Sanitation Sector
VDU	Video Display Unit
VFD	Variable Frequency Drive
VSNL	Videsh Sanchar Nigam Limited
VSS	Volatile Suspended Solids
WAN	Wide Area Network
WB	World Bank
WEF	Water Environment Federation
WHO	World Health Organization
WSP	Water and Sanitation Program
WS	Waste Sludge
WSS	Water Supply and Sanitation
WSSB	Water Supply and Sewerage Board
WTP	Water Treatment Plant
ZLD	Zero Liquid Discharge

CHAPTERS

CHAPTER 1: INTRODUCTION

Perhaps we need to be reminded what Mahatma Gandhi said ***"For India, Sanitation is more important than independence".***

1.1 PREAMBLE

Over the years, there has been a continuous migration of people from rural and semi-urban areas to cities and towns. The proportion of population residing in urban areas has increased from 27.8% in 2001 to 31.2 % in 2011. The number of towns has increased from 5,161 in 2001 to 7,935 in 2011. The uncontrolled growth in urban areas has left many Indian cities deficient in infrastructure services as water supply, sewerage, storm water drainage and solid waste management.

Most of the urban areas inhabited by slums in the country are plagued by acute problems related to indiscriminate disposal of sewage. It is due to the deficient services of the town / city authorities that sewage and its management has become a tenacious problem, even though a large part of the municipal expenditure is allotted to it. It is not uncommon to find that a large portion of resources is being utilized on manning sewerage system by Urban Local Bodies (ULBs) for their operation and maintenance (O&M). Despite this, there has been a decline in the standard of services with respect to collection, transportation, treatment and safe disposal of treated sewage as well as measures for ensuring safeguard of public health, hygiene and environment. In many cities and towns in India, major portion of sewage remains unattended leading to insanitary conditions in densely populated slums. This in turn results in an increase in morbidity especially due to pathogens and parasitic infections and infestations in all segments of population, particularly the urban slum dwellers.

Sewerage and sewage treatment is a part of public health and sanitation and according to the Indian Constitution, falls within the purview of the State List. Since this is non-exclusive and essential, the responsibility for providing the services lies within the public domain. The activity being of a local nature is entrusted to the ULBs, which undertake the task of sewerage and sewage treatment service delivery, with its own staff, equipment and funds. In a few cases, part of the said work is contracted out to private enterprises. Cities and towns, which have sewerage and sewage treatment facilities are unable to cope-up with the increased burden of providing such facilities efficiently to the desired level. Issues and constraints that are encountered by the ULBs responsible for providing sewerage and sanitation facilities are compounded due to various reasons. The main cause of water pollution is the unintended disposal of untreated, partly treated and non-point sources of sewage and more important is its effect on human health and environment.

The reasons for the above cited position are:

1. Almost all local bodies not being financially resourceful to self-generate the required capital funds and looking up to the State and Central Governments for outright grant assistance
2. Lack of institutional arrangements and capacity building to conceive planning, implementation, procurement of materials, operate and maintain the sewerage system and sewage treatment plants (STP) at the desired level of efficiency

3. The fact that the collected sewage terminates far away beyond the boundaries of the ULB and is an “out of sight, out of mind” syndrome

4. The high cost of infrastructure investment, continual replacement and on-going O&M costs of centralized sewerage system (CSS) facilities take these systems beyond the financial grasp of almost any ULB in the country

5. It is also necessary to recognize that the practice of piped sewer collection is an inheritance from advanced countries with high water usages, which permit adequate flushing velocities. Due to their high per capita water supply rates, the night-soil does not settle in pipes and hence no choking and no sulphide gas generation. Whereas, in the Indian scenario, the per capita water supply is low and inequitable in many cities and that too intermittent and this results in settling down of night-soil in the sewers, choking, gasification, etc., which necessitates very often the extreme remedies of cutting open the roads to access and break open the pipes for rectification and so on.

While the conventional sewerage may be an effective system for sewage collection and transportation and treatment, it also remains as a highly resource-inefficient technology. Consequentially, high capital cost and continuing significant costs for O&M of this system prohibit its widespread adoption in all sizes of urban areas in the country.

There has been no major effort to create community awareness either about the likely perils due to poor sewage management or the simple steps that every citizen can take which will help in reducing sewage generation and promote effective management of its generation and treatment. The degree of community sensitization and public awareness is low. There is no system of segregation of black water (from toilets) and grey water (other liquid wastes) at household level. In most cities and towns no proper service connections have been provided to the toilets connecting to the sewers.

1.1.1 Need for Safe Sanitation System

Sanitation can be perceived as the conditions and processes relating to people’s health, especially the systems that supply water and deal with the human waste. Such a task would logically cover other matters such as solid wastes, industrial and other special / hazardous wastes and storm water drainage. However, the most potent of these pollutants is the sewage.

When untreated sewage accumulates and is allowed to become septic, the decomposition of its organic matter leads to nuisance conditions including the production of malodorous gases. In addition, untreated sewage contains numerous pathogens that dwell in the human intestine tract. Sewage also contains nutrients, which can stimulate the growth of aquatic plants and may contain toxic compounds or compounds that are potentially mutagenic or carcinogenic.

For these reasons, the immediate and nuisance-free removal of sewage from its sources of generation followed by treatment, reuse or dispersal into the environment in an eco-friendly manner is necessary to protect public health and environment.

1.1.2 Present Scenario of Urban Sanitation in India

The problem of sanitation is much worse in urban areas due to increasing congestion and density in cities. Indeed, the environmental and health implications of the very poor sanitary conditions are a major cause for concern. The study of Water and Sanitation Programme (WSP) of the World Bank observes that when mortality impact is excluded, the economic impact for the weaker section of the society accounting to 20 % of the households is the highest. The National Urban Sanitation Policy (NUSP) of 2008 has laid down the framework for addressing the challenges of city sanitation. The NUSP emphasizes the need for spreading awareness about sanitation through an integrated city-wide approach, assigning institutional responsibilities and due regard for demand and supply considerations, with special focus on the urban poor.

As per the 2011 Census, 81.4% households have toilet facilities within their premises. This includes 70.9% households having water closets; 8.8% households having pit latrines; 1.7% households having other toilets (connected to open drains, night soil removed by human etc., which are unsafe). Out of the 70.9% households, 32.7% households have water closets connected to sewer system and 38.2% households are having water closets with septic tank.

The remaining 18.6% households do not have toilet facilities within their premises. This includes 6.0% households using public toilets and 12.6% households defecating in the open. As per the 2011 census, the status of toilets in urban households in India is shown in Figure 1.1.

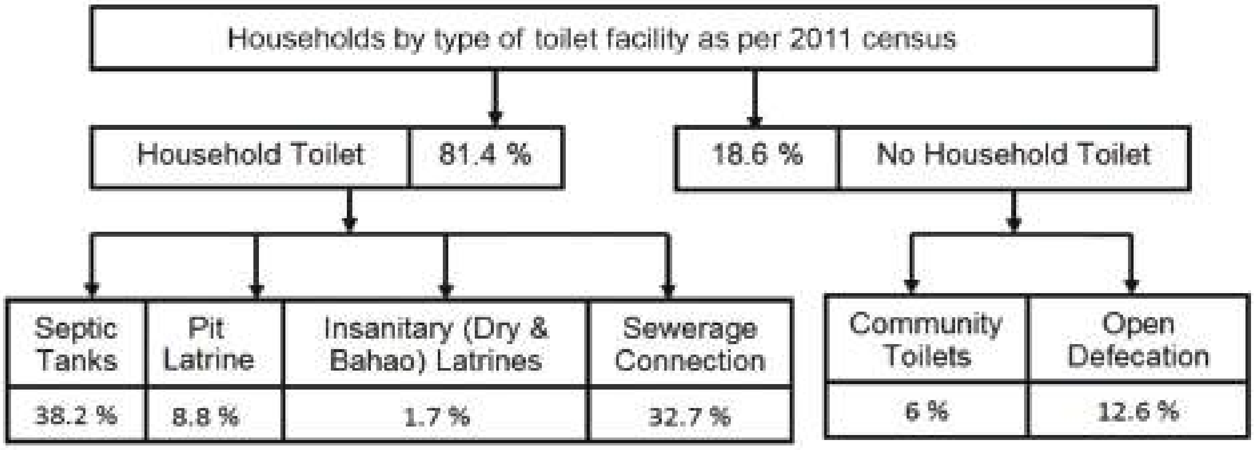


Figure 1.1 Status of Toilets in Urban Households in India

According to the report on the Status of Wastewater Generation and Treatment in Class-I Cities and Class-II towns of India, December 2009, published by Central Pollution Control Board (CPCB), the estimated sewage generation from 498 Class-I cities and 410 Class-II towns (Population estimated for 2008 based on 2001 census) together is 38,524 MLD. Out of this, only 11,787 MLD (31%) is being treated with a capacity gap of 26,737 MLD.

Sewer networks for collection and transportation of sewage from households in cities and towns are too inadequate to carry it to the STP. The STP capacities are inadequate due to many reasons.

These are poor planning and implementation of sewerage and STP and other appropriate sanitation facilities by ULBs due to inadequate financial resources and lack of adequate capacity of ULBs in the country.

This imposes significant public health and environmental costs to urban areas, that contribute more than 60% of the country's GDP. Impacts due to poor sanitation are especially significant for the urban poor (22% of total urban population), women, children and the elderly. The loss due to diseases caused by poor sanitation for children under 14 years alone in urban areas amounts to Rs. 500 crores at 2001 prices (Planning Commission-United Nations International Children Emergency Fund UNICEF, 2006). Inadequate discharge of untreated domestic / municipal sewage has resulted in contamination of more than 75 % of all surface waters across India.

1.1.3 Basic Philosophy of Sewage Treatment

Sewage when collected from communities can be perceived as a "water conveyor belt". Its treatment can be perceived as "unloading the conveyor belt" to make the belt useable again. The crucial issue is water in the conveyor belt. Hence, the treated sewage must ultimately return to the receiving water body or to the land or it could be reused for specific applications after appropriate treatment.

The complex question faced by the design engineers and the practicing engineers are :

What is the level of treatment that must be achieved in a given type of treatment beyond those prescribed by the discharge standards to ensure protection of the health of the community and the environment?

The answer to this question requires detailed analyses of local conditions and needs, application of scientific knowledge, engineering judgment based on past experience & consideration of central, state and local regulations. In some cases, a detailed assessment is required. The reuse and disposal of sludge are vexing problems for some ULBs and requires careful consideration.

1.1.4 Sewerage and Sewage Treatment Technology

Sewerage and Sewage treatment technology is the branch of environmental engineering. The basic principles of engineering are applied to solve the issues associated with collection. The basic principes of biochemistry are applied to the treatment and environmental issues in the disposal and reuse of treated sewage. The ultimate goal is the protection of public health in a manner commensurate with environmental, economic, social and political concerns. To protect public health and environment, it is necessary to have knowledge of:

1. Constituents of concern in sewage
2. Impacts of these constituents when sewage is dispersed into the environment
3. The transformation and long-term fate of these constituents in treatment processes
4. Treatment methods which can be used to remove or modify the constituents found in sewage
5. Methods for beneficial use or disposal of solids generated by the treatment systems

To provide an initial perspective in the field of sewerage and sewage treatment technology, a common terminology is first defined followed by:

1. A discussion of the issues that need to be addressed in the planning and design of sewerage management systems, and
2. The current status and new directions in sewerage and sewage treatment technology

1.1.5 Efforts of Concerned Agencies in Retrospect

Sewerage and sanitation were not accorded the due priority by the ULBs till the late seventies. The impetus of the International Drinking Water Supply and Sanitation Decade (IDWSSD), 1981 to 1990 had produced considerable efforts in urban areas in the country to improve health by investment in water supply and sanitation programmes. These comprise the sewerage and sanitation sub-sector the construction of sewers & on-site sanitation facilities using various types of toilets. Under certain hydrological conditions, unsewered sanitation can cause severe groundwater contamination by pathogens and nitrate, which may largely negate the expected health benefits of such programmes. In some circumstances, therefore, the low-cost-technologies may be incompatible.

Although the targets fixed for sewerage and sanitation coverage during the decade at the beginning of the IDWSSD were laudable, but could not be achieved due to resource constraints and other prevailing reasons. Due to these reasons, the condition of sanitation has worsened.

1.2 LOSS TO THE NATION DUE TO POOR SANITATION

1.2.1 Time and Money Loss in terms of DALYs

The Disability-Adjusted-Life-Years (DALY) is a measure of overall disease burden, expressed as the number of years lost due to ill health, disability or early death. Originally developed by the World Health Organization (WHO), it is becoming increasingly used in the field of public health and health impact assessment (HIA). It extends the concept of potential years of life lost due to premature death to include equivalent years of ‘healthy’ life lost by virtue of being in states of poor health or disability. In doing so, mortality and morbidity are combined into a single common-matrix.

As per the WHO report, 80 % of the diseases in human beings are water-borne and water-related. It is mainly due to water pollution or water contamination and water logging. Though water logging may be location and weather specific, water pollution and contamination is a common phenomenon which can occur at any place at any point of time if the community is not careful about adverse impact of indiscriminate disposal of sewage. The indiscriminate disposal of human excreta or sewage from habitations may contain hazardous micro-organisms (pathogens) for water pollution and harbouring vectors which act as carriers of pathogens.

The names of diseases mentioned in Table 1.1 (overleaf) might appear to be conventional which occur in many parts of the country. The occurrence of such diseases depends upon various factors relating to illiteracy, personal hygiene, standard of living, malnutrition, adulteration of food items, lack of community awareness among all stakeholders and other factors related to environmental pollution.

Table 1.1 Burden of water related diseases in India, 1990

(In millions of DALYs)

Diseases	Female	Male	Total
Diarrheal Diseases	14.39	13.64	28.03
Intestinal Helminths	1.00	1.06	2.06
Trachoma	0.07	0.04	0.11
Hepatitis	0.17	0.14	0.31
Total – water-borne and water-related Diseases	15.63	14.88	30.51

Source: World Bank, 1993

There is no doubt that these factors play an important role in the occurrence of diseases but unsafe disposal of untreated or partially treated sewage plays a vital role in aggravating the chances of occurrence of these communicable diseases.

If we merely consider the economic value of life years at the average per capita income of $ 300 per year, the annual loss of 30.51 million DALYs is worth of 30.51×300 = $ 9.153 billion (Exchange rate during 1993, $1 = Rs 40). Improvements in water supply and sanitation including management of municipal solid waste can substantially reduce the incidences and severity of these diseases, as well as infant mortality associated with diarrhoea as shown in the following box:

> Reduction in morbidity from better water supply and sanitation including safe disposal of municipal solid waste is estimated to be 26 % for diarrhoea, 27 % for trachoma, 29 % for ascaris, 77 % for schistosomiasis and 78 % for dracunculiasis. Mean reduction in diarrhoea-specific mortality can be 65 %, while overall child mortality can be reduced by 55 %.

Source: Esrey et. al., 1991

From the above statements and Table 1.1, it is evident that environmental pollution by liquid and solid wastes adversely affects the environment and human health directly or indirectly resulting in loss of life and heavy financial burden on exchequers.

1.2.2 Poor Sanitation Costs India $54 Billion

It has been reported from “The Economic Impact of Inadequate Sanitation in India” a report released by the Water and Sanitation Programme (WSP), states that inadequate sanitation costs India almost $54 billion or 6.4% of the country’s Gross Domestic Product (GDP) in 2006. Over 70% of this economic impact or about $38.5 billion was health-related with diarrhoea followed by acute lower respiratory infections accounting for 12% of the health-related impacts.

It is the poorest who bear the greatest cost due to inadequate sanitation. The poorest fifth of the urban population bears the highest per capita economic impact of $ 37.75, much more than the national average per capita loss due to inadequate sanitation, which is $ 21.35.

Health impacts, accounting for the bulk of the economic impacts, are followed by the economic losses due to the time spent in obtaining piped water supply and sanitation facilities, about $15 billion, and about $0.5 billion of potential tourism revenue loss due to India's reputation for poor sanitation, the report says. Table 1.2 gives a glimpse of 'How much we lose'.

Table 1.2 Poor sanitation cost to India

No.	Impact	Loss ($ billion)
1.	Health	38.5
2.	Access time (safe WSS)	15.0
3.	Tourism	0.5
	Total	54.0

Source: World Bank, 2006

The challenge of sanitation in Indian cities is acute. With very poor sewerage networks, a large number of urban poor still depend on public toilets. Many public and community toilets have no water supply while the outlets of many other toilets with water carriage systems are not connected to city's sewerage system. As per the estimate, over 50 million people in urban India defecate in the open every day. The cost in terms of Disability Adjusted Life Years (DALY) of diarrhoeal diseases for children from poor sanitation is estimated at Rs. 500 crores. The cost per DALY per person due to poor sanitation is estimated at Rs. 5,400 and due to poor hygiene practices at Rs. 900. A study by the WSP using data for 2006 shows that the per capita economic cost of inadequate sanitation including mortality rate in India is Rs. 2,180.

As mentioned above, the impacts of poor sanitation on human health are significant. Unsafe disposal of human excreta facilities are responsible for the transmission of oral-faecal diseases, including diarrhoea and a range of intestinal worm infections such as hookworm and roundworm. Diarrhoea accounts for almost one-fifth of all deaths (or nearly 535,000 annually) among Indian children who are under 5 years. In addition, rampant worm infestation and repeated diarrhoea episodes result in widespread childhood malnutrition. Moreover, India is losing millions of rupees each year because of poor sanitation. Illnesses are costly to families and to the economy as a whole in terms of productivity losses and expenditure on medicines, health care, etc. The economic toll is also apparent in terms of water treatment costs, losses in fisheries production, tourism, welfare impacts such as reduced school attendance, inconvenience, wasted time, lack of privacy & security for women. On the other hand, ecologically sustainable sanitation can have significant economic benefits that accrue from recycling nutrients and using biogas as an energy source.

1.3 SECTOR ORGANIZATION

Water supply and sanitation is treated as a state subject as per the Federal Constitution of India and, therefore, the States are vested with the constitutional right on planning, implementation, operation and maintenance and cost recovery of water supply and sanitation projects.

At the local level, the responsibility is entrusted by legislation to the local bodies like Municipal Corporation, Municipality, Municipal Council and Notified Area Committee/Authority for towns or on a State/Regional basis to specialized agencies. The economic and social programme of the country is formulated through five-year plans.

The Public Health Engineering Department (PHED) is the principal agency at State level for planning and implementation of water supply and sanitation programmes. In a number of States, statutory Water Supply and Sanitation Boards (WSSBs) have taken over the functions of the PHEDs. The basic objectives for creation of WSSBs have been to bring in the concept of commercialization of the water supply and sanitation sector management and more accountability. Such boards have been set up in Assam, Bihar, Gujarat, Karnataka, Kerala, Maharashtra, Orissa, Punjab, Uttar Pradesh and Tamil Nadu. The metropolitan cities of Bangalore, Hyderabad and Chennai have separate statutory Boards. The water supply and sanitation services in the cities of Ahmedabad, Delhi, Kolkata, Mumbai, Pune and few other cities are under the Municipal Corporations.

The Ministry of Urban Development (MoUD), Government of India (GOI) formulates policy guidelines in respect of Urban Water Supply and Sanitation Sector and provides technical assistance to the States and ULBs wherever needed. The expenditure on water supply and sanitation is met out of block loans and grants disbursed as Plan assistance to the States, and out of loans from financial institution like Life Insurance Corporation of India (LIC) and Housing and Urban Development Corporation (HUDCO). The Central Government acts as an intermediary in mobilizing external assistance in water supply and sanitation sector and routes the assistance via the State plans. It also provides direct grant assistance to some extent to water supply and sanitation projects in urban areas under the various programmes of GOI.

1.4 INITIATIVES OF GOVERNMENT OF INDIA

Government of India has taken number of initiatives during the last two decades by implementing number of reforms aimed at improving the working efficiency of ULBs in India. These reforms have been implemented in the form of Act (Amendment) and all the State Governments have been advised to implement these reforms by suitably modifying ULB's bye-laws so as to achieve the objectives of these reforms for the development of urban sector in the country. Few of the reforms such as institutional, financial, legal, etc., are in vogue. The reforms mainly relating to sewerage and sanitation are briefly described as under.

1.4.1 Initiative on Reforms – 74th Constitution Amendment Act, 1992

Quite often, multiplicity of agencies and overlapping of responsibilities are the reasons for ineffective and poor operation and maintenance of the assets created by civic bodies. In the light of 74th Amendment under the 12th Schedule of the Constitution, the role and responsibility of the ULBs have increased significantly in providing these basic facilities to the community on a sustainable basis. The new Amendment has enabled ULBs to become financially viable and technically sound to provide basic amenities to the community.

As per the 74th Constitution Amendment Act, 1992, the ULBs have been delegated with sets of responsibilities and functions; however, they are not supplemented with adequate financial resources. As a result, they are not able to perform their assigned functions in an efficient and effective manner. They are also not able to fix the rates of user charges and are heavily dependent upon the higher levels of Government grants. Consequent to the 74th Constitutional Amendment Act (74th CAA), the States are expected to devolve responsibility, powers and resources upon ULBs as envisaged in the 12th Schedule of the Constitution. The 74th CAA has substantially broadened the range of functions to be performed by the elected ULBs. The 12th Schedule brings into the municipal domain among others such as urban and town planning, regulation of land-use, planning for economic & social development and safeguarding the interests of weaker sections of the society.

The Constitution thus envisages ULBs as being totally responsible for all aspects of development, civic services and environment in the cities going far beyond the traditional role. The focus should not only be on the investment requirements to augment supplies or install additional systems in sanitation and water supply. Instead, greater attention must be paid to the critical issues of institutional restructuring, managerial improvement, better and more equitable service to citizens who must have a greater degree of participation. The 74th CAA also focuses on achieving sustainability of the sector through the adoption of adequate measures in O&M, the financial health of the utilities through efficiency of operations and levy of user charges, and conservation & augmentation of the water sources.

1.4.2 THE PROHIBITION OF EMPLOYMENT AS MANUAL SCAVENGERS AND THEIR REHABILITATION ACT, 2013

The Government of India has enacted The Prohibition of Employment as Manual Scavengers and their Rehabilitation Act, 2013 in September 2013 to remove certain anomalies in the erstwhile legislation of The Employment of Manual Scavengers and Construction of Dry Latrines (Prohibition) Act, 1993.

The 1993 Act served as a primary instrument to eradicate practice of manual scavenging, but the House listing data from Census, 2011 showed the existence of manual scavenging in many of the States. The Prohibition of Employment as Manual Scavengers and their Rehabilitation Act, 2013, provides for the following distinction very clearly to end this dehumanizing practice of manual scavenging and also to eliminate the hazardous cleaning of septic tanks and sewers by the sanitary workers.

- Prohibits insanitary latrines.
- Prohibits hazardous manual cleaning of sewers and septic tanks.
- Offences committed under the law is cognizable and non-bailable.
- Appropriate governments shall confer powers on local authority and district magistrates.
- Vigilance and Monitoring committee to be set up at the level of sub-division, District, State and Central to monitor the implementation of the Act.
- The responsibility of the local authorities under the Act is mandatory.

The Centrally sponsored scheme of Integrated Low Cost Sanitation scheme implemented by the Ministry of Housing and Urban Poverty Alleviation (MoHUPA) for liberation of the scavengers was started in year 1980-81. It is now being operated through MoHUPA.

As per the scheme's revised guidelines 2008, the objectives of the scheme are

- To convert / construct low cost sanitation units through sanitary two pit pour flush latrines with superstructures and appropriate variations to suit local conditions and
- To Construct new latrines where the economically weaker section (EWS) households have no latrines and
- To avoid the inhuman practice of defecating in the open in urban areas.

This would improve the overall sanitation in towns. The manual scavengers thus liberated if any or their dependents would have to be rehabilitated under the scheme by the State Governments simultaneously with the help of funds provided by the Ministry of Social Justice and Empowerment (MoSJE).

As per the Gazette of India dated October 2013, the Act shall come into force from December 6, 2013. The text of the Act as in the Gazette is in Appendix A 1.1. The time frame specified under the Act for the fulfilment of responsibilities and carrying out certain activities are mentioned in Appendix A 1.2.

1.5 NATIONAL URBAN SANITATION POLICY (NUSP), 2008

The NUSP adopted by the MoUD in 2008, envisions that "All Indian cities and towns become totally sanitized, healthy and liveable and ensure and sustain good public health and environmental outcomes for all their citizens, with a special focus on hygenic and affordable sanitation facilities for the urban poor and women"

According to the NUSP ''Sanitation is defined as safe management of human excreta, including its safe confinement treatment, disposal and associated hygiene-related practices''.

1.5.1 Key Sanitation Policy Issues

In order to achieve the above vision, following key policy issues must be addressed

- Poor Awareness: Sanitation has been accorded low priority and there is poor awareness about its inherent linkages with public health.
- Social and Occupational aspects of Sanitation: Despite the appropriate legal framework, progress towards the elimination of manual scavenging has shown limited success, Little or no attention has been paid towards the occupational hazard faced by sanitation workers daily.
- Fragmented Institutional Roles and Responsibilities: There are considerable gaps and overlaps in institutional roles and responsibilities at the national, state, and city levels.

- Lack of an Integrated City-wide Approach: Sanitation investments are currently planned in a piece-meal manner and do not take into account the full cycle of safe confinement, treatment and safe disposal.
- Limited Technology Choices: Technologies have been focussed on limited options that have not been cost-effective and sustainability of investments has been in question.
- Reaching the Un-served and Poor: Urban poor communities as well other residents of informal settlements have been constrained by lack of tenure, space or economic constraints, in obtaining affordable access to safe sanitation. In this context, the issues of whether services to the poor should be individualized and whether community services should be provided in non-notified slums should be addressed. However provision of individual toilets should be prioritized. In relation to “Pay and Use” toilets, the issue of subsidies inadvertently reaching the non-poor should be addressed by identifying different categories of urban poor.
- Lack of Demand Responsiveness: Sanitation has been provided by public agencies in a supply driven manner, with little regard for demands and preferences of households as customers of sanitation services.

1.5.2 National Urban Sanitation Policy Goals (NUSP)

The overall goal of this policy is to transform urban India into community-driven, totally sanitized, healthy, and liveable cities and towns. The specific goals are:

a. Awareness generation and behaviour change

b. Open defecation free cities

c. Integrated city-wide sanitation

1.5.3 Concepts of Totally Sanitized Cities

A totally sanitized city will be one that has achieved the outputs or milestones specified in the NUSP, the salient features are given below.

a. Cities must be open defecation free.

b. Must eliminate the practice of manual scavenging and provide adequate personnel protection equipment that addresses the safety of sanitation workers.

c. Municipal sewage and storm water drainage must be safely managed.

d. Recycle and reuse of treated sewage for non-potable applications should be implemented wherever possible.

e. Solid waste collected and disposed off fully and safely.

f. Services to the poor and systems for sustaining the results.

g. Improved public health outcomes and environmental standards.

1.6 SANITATION PROMOTION

In order to rapidly promote sanitation in urban areas of the country (as provided for in the NUSP and Goals 2008), and to recognize excellent performance in this area, the GOI intends to institute an annual rating award scheme for cities (NUSP 2008).

The MoUD is also promising a National Communication Campaign to generate awareness on sanitation both at the household level and at the service provider level. The aim of this exercise is to generate awareness of the benefits of hygiene and clean environment and thereafter bring about behaviour. The suggested real time Total Sanitation Model is given in Figure 1.2 .

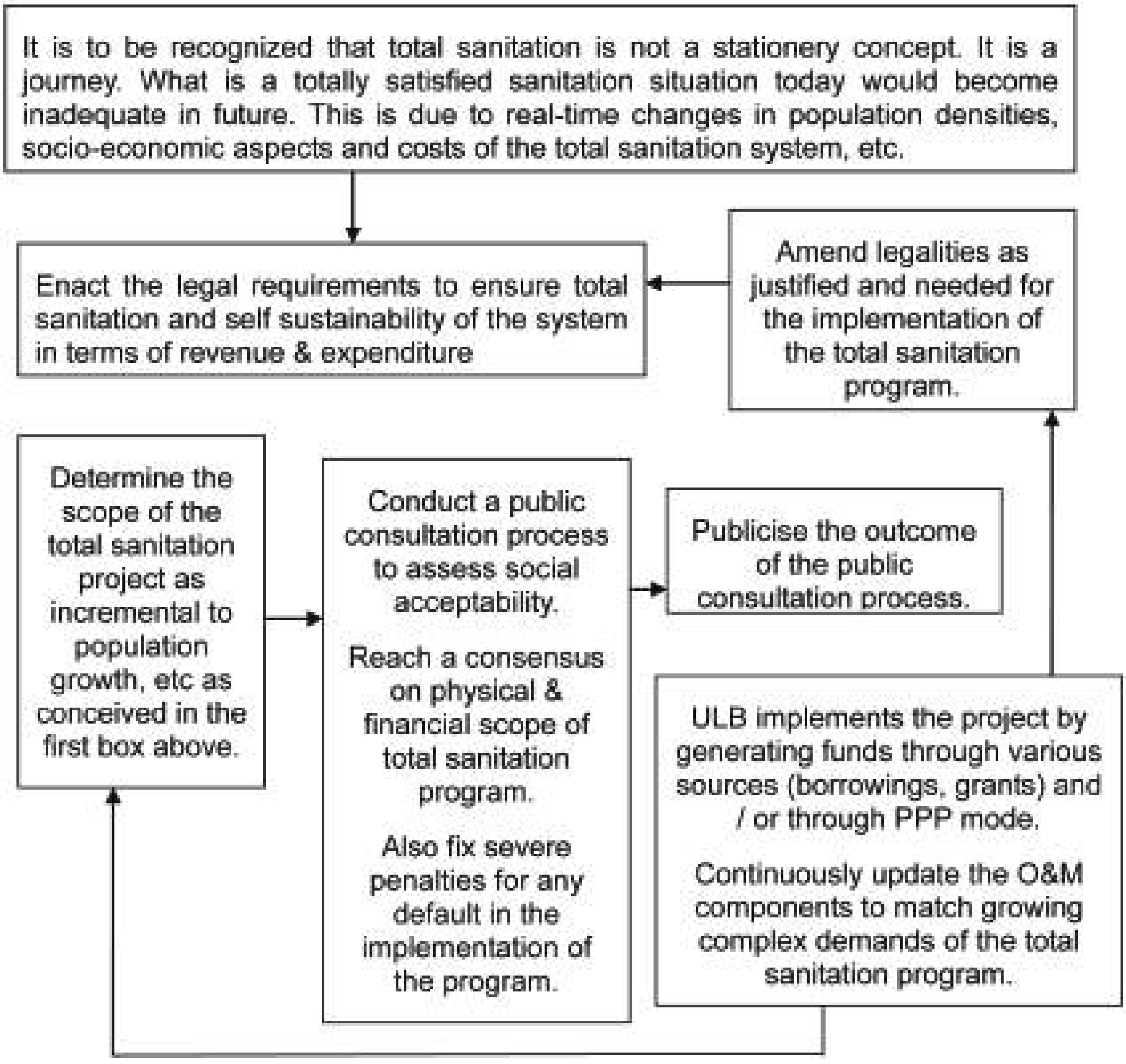


Figure 1.2 Suggested Real Time Total Sanitation Model

1.7 SERVICE LEVEL BENCHMARKING ON SEWAGE MANAGEMENT (SEWERAGE AND SEWAGE TREATMENT)

The Millennium Development Goals (MDGs) enjoins upon the signatory nations to extend access to improved sanitation to at least half the urban population by 2015, and 100% access by 2025.

This implies extending coverage to households with improved sanitation and providing proper sanitation facilities in public places to make cities and towns open-defecation free. The Ministry proposed to shift focus on infrastructure in urban water supply and sanitation sector (UWSS) to improve service delivery.

The Ministry formulated a set of Standardized Service Level Benchmarks (SLB) for UWSS as per International Best Practice and brought out a “Handbook on Service Level Benchmarking” on water supply and sanitation sector during the year 2008.

The SLB on Sewage Management (Sewerage and Sewage management) are given in Table 1.3 which are required to be achieved within a specified time frame.

Table 1.3 Sewage Management (Sewerage and Sanitation)

No.	Proposed Indicator	Benchmark
1.	Coverage of toilets	100%
2.	Coverage of sewage network services	100%
3.	Collection efficiency of sewage network	100%
4.	Adequacy of sewage treatment capacity	100%
5.	Quality of sewage treatment	100%
6.	Extent of reuse and recycling of sewage	20%
7.	Efficiency of redressal of customer complaints	80%
8.	Extent of cost recovery in sewage treatment	100%
9.	Efficiency in collection of sewage charges	90%

Source: MoUD, 2011

1.8 EMERGING TRENDS AND TECHNOLOGIES OF SEWERAGE AND SEWAGE TREATMENT

1.8.1 Recent Trend - Centralized vis-a-vis Decentralized Sewerage Systems

While the conventional sewerage may be a comprehensive system for sewage collection and transport, it also remains as a highly resource-intensive technology. Consequently, high capital cost and significant O&M cost of this system inhibits its widespread adoption in all sizes of urban areas.

The implementation of Centralized Wastewater Management System (CWMS) should not be considered as the only option available for collection, transportation and treatment of sewage. There are certain factors which govern the selection of options between CWMS and Decentralized Wastewater Management System (DWMS). These have been elaborately discussed in the relevant chapter of this Manual.

The DWMS may be designed as the collection, treatment, and disposal/reuse of sewage from individual houses, cluster of houses, isolated communities, industries or institutional facilities as well as from portion of existing communities at or near the point of generation of sewage.

The DWMS maintains both the solids and liquid fraction, although the liquid portion and any residual solids can be transported to a centralized point for further treatment and reuse.

Recognizing the many applications and benefits of sewage reuse, some important points may be kept in view such as

i. Review of the impact of the population growth rate

ii. Review of potential water reuse applications and water quality requirements

iii. Review of appropriate technologies for sewage treatment and reuse

iv. Considering the type of management structure that will be required in the future and

v. Identification of issues that must be solved to bring about water reuse for sustainable development on a broad scale.

It has been emphasized that, if the sewage from the urban and semi urban areas were reused for a variety of non-potable uses, the demand on the potable water supply would be reduced.

The choice of appropriate technology will also depend on several factors such as composition of sewage, availability of land, availability of funds and expertise. Different operation and maintenance options will have to be considered with respect to sustainable plant operation, the use of local resources, knowledge and manpower.

1.9 NEED FOR REVISION AND UPDATING OF THE EXISTING MANUAL ON SEWERAGE AND SEWAGE TREATMENT (1993)

Ever since the publication of the Manual on sewerage and sewage treatment in 1993 a number of new developments and changes have occurred in the complete range of technologies of collection, transportation, treatment and reuse of treated sewage and sludge for various usages during the last two decades. Broad approaches adopted for the need of revision and updating of the manual on sewerage and sewage treatment are as mentioned below:

i. A greater fundamental understanding of the mechanisms of the biological treatment.

ii. The application of advanced treatment methods for the removal of specific constituents.

iii. The increased emphasis on the management of sewerage and sewage treatment in general and management of sludge resulting from the treatment of sewage, and

iv. The issuance of more comprehensive and restrictive permit requirements for the discharge and reuse of treated sewage.

Even though the sewerage and sewage treatment practices have continued to evolve and grow during last two decades, no time period in the past can equal this intervening period in terms of technological development. In addition, awareness of the environmental issues among the national urban communities has reached a level not experienced before. This active awareness is a driving force for the agencies responsible for sewerage and sewage treatment to achieve the level of performance far beyond those envisioned even during the last two decades.

Pressure for environmental compliance today is greater than before. The need for sewerage and sanitation schemes in urban areas and regulatory requirements have, at present, become more forceful. Support from the Central and State Governments for environmental-related programmes are becoming a strong driving force than ever before. Communities are quite aware, well organized, and informed.

The revision and updating of the existing manual (1993) aims to meet some of those needs by providing advice on the selection of technology options for urban sanitation and whether new infrastructure or upgrading of existing services. It is applicable both to small interventions in specific locations and larger programmes that aim to improve sanitation citywide. The selection of technologies with various options for providing techno-economic solutions keeping in view health of the community and safeguarding the environment are listed below to provide a wide range of options to the planners and designers:

i. Decentralized sewerage system
ii. Sludge treatment and septage management
iii. Recent technologies on sewerage and sewage treatment
iv. New pipe materials for construction of sewers
v. Guidelines for recycling and reuse of treated sewage
vi. New guidelines for discharging treated sewage into water bodies used for drinking.

1.9.1 Guidelines for Preparation of City Sanitation Plan (CSP)

One of the most important objectives of revising and updating of this manual is ‘Preparation of City Sanitation Plan,’ which has been amply described in Chapter 10 so as to give proper guidance to decision makers, planners & designers and also suitably involve political initiatives as a tool to envision affordable upgrade of existing sanitation systems and futuristic sanitation systems in a self-sustaining basis.

The algorithm given in Chapter 10 is a very useful approach for decision makers and planners to adopt the most suitable strategy for providing safe sanitation to the urban community within the policy framework of the GOI in the country.

1.10 SETTING-UP OF ENVIRONMENTAL POLLUTION STANDARDS AT THE STATE LEVEL

While planning the citywide sanitation programme, concerned agencies must set-up standards and follow at the State Level (within overall framework of national standards) such as CPHEEO and BIS guidelines values as mentioned below:

a. Environment Outcome (e.g., State Pollution Control Boards standards on effluent parameters, diminishing water resources, impact of climate change, use of low energy intensive on-site/decentralized sewage treatment technologies, distributed utilities, etc.),

b. Public Health Outcomes (e.g., State Health Departments),

c. Processes (e.g., safe disposal of on-site septage) and infrastructure (e.g., design standards) (PHEDs/Parastatals) and coverage of the informal sector activities like disposal of sewage, solid waste, etc.,

d. Service delivery standards (e.g., by the Urban Development Departments),

e. Manpower issues such as adequate remuneration, hazardous nature of work, employment on transparent terms and conditions, use of modern and safe technology, provision of adequate safety equipment such as glove, boots, masks, regular health check-ups, medical and accident insurance, etc.,

f. States are recommended to not just emulate but also set their standards higher than the national standards in order to encourage its institutions and citizens to target higher standards of public health and environment.

1.11 RELATIONSHIP BETWEEN PART-A (ENGINEERING), PART-B (OPERATION AND MAINTENANCE), AND PART-C (MANAGEMENT) OF THIS MANUAL

The present manual is one of a set of three parts and which are interdependent as under:

i. Part – A on ‘Engineering’

ii. Part – B on ‘Operation and Maintenance’

iii. Part – C on ‘Management’

Part – A on ‘Engineering’ addresses the core technologies and updated approaches towards the incremental sanitation from on-site to decentralized or conventional collection, conveyance, treatment and reuse of the misplaced resource of sewage and is simplified to the level of the practicing engineer for day to day guidance in the field in understanding the situation and coming out with a choice of approaches to remedy the situation. In addition, it also includes recent advances in sewage treatment and sludge & septage management to achieve betterment of receiving environment. It is a simple guideline for the field engineer.

Part – B on ‘Operation and Maintenance’ addresses the issues of standardizing the human and financial resources. These are needed to sustain the sewerage and sanitation systems which are created at huge costs without slipping into an edifice of dis-use for want of codified requirements for O&M so that it would be possible to address the related issues. These financial and related issues are to be addressed at the estimate stage itself, thus enabling to seek a comprehensive approval of fund allocations and human resources. This would also usher in the era of public private partner-ship to make the projects self-sustaining. This also covers aspects such as guidelines for cleaning of the sewers and septic tanks besides addressing the occupational health hazards and safety measures of the sanitation workers. It is a simple guidance for the resource seeker and resource allocating authorities.

Part – C on ‘Management’ addresses the modern methods of project delivery and project validation and gives a continual model for the administration to foresee the deficits in allocations and usher in newer mechanisms. It is a tool for justifying the chosen project delivery mechanism and optimizing the investments on need based allocations instead of allocations in budget that remain unutilized and get surrendered at the end of the fiscal year with no use of the funds to anyone in that whole year. It is a straight forward refinement of a mundane approach over the decades.

It is important to mention here in the beginning of this Part- A of the Manual that trade names and technology nomenclatures, etc., where cited, are only for familiarity of explanations and not a stand alone endorsement of these.

CHAPTER 2: PROJECT PLANNING

2.1 VISION

The vision for urban sanitation in India as mentioned in the NUSP (2008) of GOI is:

“All Indian cities and towns become totally sanitized, healthy and liveable and ensure and sustain good public health and environmental outcomes for all their citizens with a special focus on hygienic and affordable sanitation facilities for the urban poor and women”.

2.2 OBJECTIVES

The objective of a sewage collection, treatment and disposal system is to ensure that sewage discharged from communities is properly collected, transported and treated to the required degree in short, medium and long-term planning and disposed-off/reused without causing any health or environmental problems.

Short term: Implies the immediate provision of on-site system. It is an interim arrangement until the implementation of the long-term plan. Short-term plans should be formulated for a target up to 5 years from the base year.

Medium term: Implies the provision of a decentralized (non-conventional) system of collection for rapid implementation of collection, transportation, treatment and disposal/local reuse to avoid sporadic sewage discharges into the environment and where conventional sewerage system is not feasible. Medium-term plans should have a target of 15 years from the base year.

Long term: Implies conventional sewage collection, transportation, treatment, and environmentally sound disposal/reuse. It encompasses the short term and medium term. Long-term plans should be formulated for a target of 30 years from the base year.

2.3 NEED FOR PROJECT PLANNING

The City sanitation plan is the pre-requisite for sewerage projects. The decision tree in selecting the appropriate technical option whether it is on-site, decentralised or conventional system as in Figure 10.2 shall be followed. While preparing the plan, the data similar to Figure 10.1, has to be first enumerated specific to the classification therein. Only after having assessed the above status, the plan for the city can be conceived in accordance with the NUSP. While doing so the real time total sanitation model shown in Figure 1.2 shall also be taken into consideration.

The city sanitation plan should include

1. Provision of individual and community toilets to prohibit open defecation
2. Conversion of insanitary toilets such as dry or bahao toilets (directly connected to open drain), single pit toilets, etc to sanitary toilets
3. Replacement of existing septic tanks, which are not as per the specifications and further improvements

4. Decentralised sewerage system including treatment and non-conventional sewers such as settled or simplified sewers / small bore sewers, twin drains where water is scarce

5. Septage management including desludging of septic tanks, transportation, treatment and its disposal

6. Mechanization or cleaning of sewers, septic tanks and safety devices for sanitation workers

7. Conventional sewerage system where per capita supply is more than 135 lpcd or augmentation of water supply in contemplated with 135 lpcd

8. Recycling and reuse of treated sewage

9. Provision for public toilets

However, while preparing the CSP, solid waste management and storm water drainage components should also be considered as envisaged under NUSP.

Sewage collection, treatment and disposal systems can be either short-term, medium-term or long-term. To keep overall costs down, most urban systems today are planned as an optimum mix of the three types depending on various factors.

Planning is required at different levels: national, state, regional, local and community. Though the responsibility of various organizations in charge of planning sewage collection, treatment and disposal systems is different in each case, they still have to function within the priorities fixed by the national and state governments and keep in view the overall requirements of the area.

2.4 BASIC DESIGN CONSIDERATIONS

2.4.1 Engineering considerations

2.4.2 Institutional aspects

2.4.3 Environmental considerations

2.4.4 Treatment process

2.4.5 Financial aspects

2.4.6 Legal issues

2.4.7 Community awareness

2.4.8 Inter and Intra departmental coordination

2.4.9 Geographical information systems

2.4.10 City master plan

2.4.11 City sanitation plan

2.4.1 Engineering Considerations

Topographical, engineering and other considerations, which figure prominently in project design, are mentioned below:

a) Design period, stage wise population to be served, expected sewage volume, sewage quality and fluctuation with respect to time

b) Topography of the general area to be served, its slope and terrain and geological considerations. Tentative sites available for STP, sewage pumping station (SPS) and disposal works, considering flooding conditions

c) Available hydraulic head in the system up to high flood level in case of disposal to a nearby river or high tide level in case of coastal discharge or the level of the irrigation area to be commanded in case of land disposal

d) Depth of groundwater table and its seasonal fluctuation affecting construction, sewer infiltration & structural design (uplift considerations)

e) Soil bearing capacity and type of strata expected to be met with in construction

f) On-site disposal facilities, including the possibilities of segregating the sullage water and sewage and reuse or recycle sullage water within the households

g) Existing water supply, sewerage and sanitation conditions

h) Water reliability, augmentation steps, drought conditions

i) Reuse in agriculture, farm forestry, non-potable urban usages and industries

j) Decentralized sewerage and progressive coverage

2.4.2 Institutional Aspects

a) Capability of existing local authority

b) Revenue collection and reliability

c) Capacity building needs

d) Public Private Partnership

2.4.3 Environmental Considerations

The following aspects should be considered during design:

a) Surface Water Hydrology and Quality

Hydrological considerations affect the location of outfalls to rivers with regard to protection of nearby water supply intake points either upstream or downstream, especially at low flow conditions in the river. Hydrological considerations also help determine expected dilutions downstream, frequency of floods and drought conditions, flow velocities, travel times to downstream points of interest, navigation, etc.

Surface water quality considerations include compliance with treated effluent standards at the discharge point with respect to parameters like BOD, suspended and floating solids, oil & grease, nutrients, coliforms, etc. Special consideration may be given to the presence of public bathing ghats downstream. The aquatic ecosystem (including fish) may also need protection in case of rivers through minimum dissolved oxygen (DO) downstream, ammonia concentrations in the water, uptake of refractory and persistent substances in the food chain, and protection of other legitimate uses to which the river waters may be put to.

b) Ground Water Quality

Another environmental consideration is the potential for ground water pollution presented by the STP proposed to be built. For example, in certain soils, special precautions may be needed to intercept seepage of sewage from lagoons and ponds. Land irrigation would also present a potential for ground water pollution especially from nitrates. In case of low cost sanitation involving on-site disposal of excreta and sullage, ground water pollution may need special attention if the ground water table is high and if the top soil is relatively porous.

c) Coastal Water Quality

Shoreline discharges of sewage effluents, though treated, could lead to bacterial and viral pollution and affect bathing water quality of beaches. Discharges have to be made offshore and at sufficient depth through marine outfall to benefit from dilution and natural die-away of organisms before they are washed back to the shoreline by currents. The presence of nutrients could also promote algal growth in coastal waters, especially in bays where natural circulation patterns might keep the nutrients trapped in the water body.

d) Odour and Mosquito Nuisance

Odour and mosquito nuisance in the vicinity of STP, particularly in the downwind direction of prevailing winds, can have adverse impacts on land values, public health and environment and general utility of amenities may be threatened. These factors have to be considered in the selection of technologies and sites for location of STP and the use of treated sewage for irrigation.

e) Public Health

Public health considerations pervade through all aspects of design and operation of sewage treatment and disposal projects. Some aspects have already been referred to in earlier part of this section. Public health concepts are built into various bye-laws, regulations and codes of practice, which must be observed, such as:

i) Effluent discharge standards including the permissible microbial and helminthic quality requirements

ii) Standards for control of toxic and accumulative substances in the food chain

iii) Potential for nitrate and microbial pollution of ground waters

iv) Deterioration of drinking water resources including wells

v) Deterioration of bathing water quality

vi) Control measures for health and safety of sewage plant operators and sewage farm workers, and nearby residents, who are exposed to bio-aerosols or handle raw and/or treated sewage.

f) Landscaping

The STP structures need not be ugly and unsightly. At no real extra cost, some architectural concepts can be used and the buildings designed to suit the main climates (humid or dry) generally met within India.

Apart from the usual development of a small garden near the plants office or laboratory, some considerations need to be given to sites for disposal of screenings and grit in a harmless manner, general sanitation in the plant area and provision of a green belt around the STP. Green belt around the STP shall be preferably of plants with shallow roots in order to avoid deep and spread roots from trees accessing the water retaining structures and damaging their construction by ingress to the moist zones.

g) Status of pollution of surface waters, ground waters and coastal waters

h) Remediation needs and realistic solutions to mitigation of pollution

i) Solid wastes disposal and leachates as affecting the likely siting of STPs

j) Condition of sludge generated in STPs and potential to go in for vermicomposting

k) Clean Development Mechanism by biomethanation and energy recovery from STPs

l) Vital statistics and frequency of waterborne and vector borne diseases

2.4.4 Treatment Process

The process considerations involve factors, which affect the choice of treatment method, its design criteria and related requirements such as the following:

a) Sewage Flow and Characteristics

This constitutes the primary data required for process design. The various parameters to be determined are described in other sections of the manual.

b) Degree of Treatment Required

In case of domestic or municipal sewage, this is considered, for example, in terms of removal of BOD, nutrients (nitrogen and phosphorous), coliforms, helminths etc. Land disposal generally has to meet less stringent discharge standards than disposal to surface waters. Land disposal also has the advantage of avoiding nutrient removal in STP and is preferred wherever feasible. It is often not enough to aim only at BOD removal and let other items be left to unspecified, incidental removal, whatever may occur. The selection of a treatment process thus, depends on the extent of removal efficiency required for all the important parameters and the need to prevent nuisance conditions.

c) Performance Characteristics

The dependability of performance of a process in spite of fluctuations in influent quality and quantity are very useful attributes in ensuring a stable effluent quality. Similarly, the ability to withstand power and operational failures, also form important considerations in the choice of treatment process. The more high-rated treatment process, the more sensitive it is in operation. Other processes like digesters, lagoons and ponds may be sensitive to extreme temperature range. The choice has to match with the discharge standards to be met in a specific case. The performance characteristics for some methods of sewage treatment are indicated in Appendix A.2.1.

d) Other Process Requirements

Various other factors affecting the choice of a process include requirements in terms of -:

- Land
- Power and its dependability
- Operating (and control) equipment requirement and its indigenous availability
- Skilled staff
- Nature of maintenance problems
- Extent of sludge production and its disposal requirements
- Loss of head through plant in relation to available head (to avoid pumping as far as possible)
- Adoption of modular system

Between the land and power requirements, a trade-off is often possible, based on the actual costs. This could well be exploited to get an optimum solution for ensuring treatment requirements and giving a dependable performance.

The operating equipment and its ancillary control equipment should be easy to operate and maintain (with indigenously available spare parts) as far as possible. It is to be noted that, methane gas collection, scrubbing to remove hydrogen sulphide wherever necessary and its conversion to electricity, should be effectively done. The option of gas collection and supply to a nearby industry or area should be favoured during the site selection stage wherever possible.

The related issues are –

e) To be affordable by the ULB for its O&M

f) Trade-offs between portions to be treated for industrial uses and portions to be discharged

g) Possibility of upgrading with respect to incrementing flows over time

h) Dependency on proprietary spares to be avoided or in built into the O&M contract itself

i) Local skills to comprehend and implement monitoring

2.4.5 Financial Aspects

Finally, from among the few selected options, the overall costs (capital and operating) and financial sustainability have to be determined in order to arrive at the optimum solution.

a) Capital costs include all initial costs incurred up to plant start-up, such as:

- Civil construction, equipment supply and erection costs
- Land purchase costs including legal fees, if any
- Engineering design and supervision charges
- Interest charge on loan during construction period

b) Operating costs after start-up of plant include direct operating costs and fixed costs, such as:

• Amortisation and interest charges on capital borrowing

• Direct operation and maintenance costs on

- Salary & Wages
- Chemicals
- Energy
- Transport
- Maintenance and repairs
- Tools and Plants
- Insurance
- Overheads

c) Financial sustainability

• Levy of appropriate sewerage charges

• Capacity & Willingness to pay by the users.

• Willingness to charge

• Efficient sewerage charge collection

• Supplementary budget from alternate sources

• Revenue generation potential of the concerned local body, water boards, PHED's / Jal Nigams, Parastatal organizations, as the case may be

• Actual recovery generated

2.4.6 Legal Issues

In general, legalities do not affect sewerage projects except land acquisition issues, which require tact, patience and perseverance.

2.4.7 Community Awareness

In general, the decision-making on sewerage system management is carried out without involving the public at large and this has to change by appropriate web-based messages, hand-outs, public hearings and documenting the outcomes and taking the population along.

2.4.8 Inter- and Intra-departmental Coordination

a) Co-ordination between ULB and water boards/PHEDs/Jal Nigams/as the case may be

b) Co-ordination among water boards / PHEDs / Jal Nigams / ULB as the case may be and the elected representatives

c) Intra-departmental coordination

2.4.9 Geographical Information Systems

Geographical Information Systems (GIS) should be an integral part of sewage collection system. It allows developing city master plans (CMP), including CSP rapidly and in a precise manner and can be related precisely to its position in the ground.

The spatial modelling capabilities of GIS can be used to estimate current and future sewage flows, evaluate the capacity of the sewers and estimate the condition of the sewers.

2.4.10 City Master Plan

The CMP shall be prepared clearly indicating the various aspects as this will form a basis for the project. The CSP shall also mandatorily form part of the CMP. The various aspects to be considered are in Chapter 10. Any proposal submitted for funding shall mandatorily include the CMP and CSP. It is very important and pertinent to include and account for the mandatory provision of adequate and proper sanitation facilities in every school in the country thus complying with the directive of the GOI.

The planning period to be adopted for the preparation of the master plan shall be 30 years. In order to bring the master plan projections on the same time line for comparison and funding, the Town & Country planning authority would also be required to increase their planning period, from the present 20 years to 30 years for the reasons mentioned earlier.

2.4.11 City Sanitation Plan

The CSP should be a part of CMP and it should be prepared in accordance with the NUSP. The planning design period for on-site, decentralised and centralised systems shall be 5 years, 5 to 15 years and 30 years respectively.

2.5 DESIGN PERIOD

The project components may be designed for the periods mentioned in Table 2-1 overleaf.

Table 2.1 Design period of sewerage components

Sl. No	Component	Design Period, Years (from base year)
1	Land Acquisition	30 years or more
2	Conventional sewers (A)	30
3	Non-conventional sewers (B)	15
4	Pumping mains	30
5	Pumping Stations-Civil Work	30
6	Pumping Machinery	15
7	Sewage Treatment Plants	15
8	Effluent disposal	30
9	Effluent Utilization	15 or as the case may be
(A) Typical underground sewers with manholes laid in the roads (B) All types such as small bore, shallow sewers, pressure sewers, vacuum sewers		

Source: CPHEEO, 1993

2.6 POPULATION FORECAST

2.6.1 General Considerations

The design population should be estimated by paying attention to all the factors governing the future growth and development of the project area in the industrial, commercial, educational, social, and administration spheres. Special factors causing sudden immigration or influx of population should also be predicted as far as possible.

A judgement based on these factors would help in selecting the most suitable method of deriving the probable trend of the population growth in the area or areas of the project from the following mathematical methods, graphically interpreted where necessary:

a) Demographic method of population projection

The population change can occur in three ways: by birth (population gain), by death (population loss), or by migration (population loss or gain depending on whether movement-out or movement -in occurs in excess). Annexation of area may be considered a special form of migration. Population forecasts are frequently made by preparing and summing up separate but related projections of natural increases and of net migration, and are expressed below.

The net effect of births and deaths on population is called natural increase (natural decrease, if deaths exceed births).

The migration also affects the number of births and deaths in an area, and hence, projections of net migration are prepared before projections for natural increase.

This method thus takes into account the prevailing and anticipated birth rates and death rates of the region or city for the period under consideration.

An estimate is also made of the emigration from and immigration to the community, its area-wise growth and the net increase of population is calculated accordingly considering all these factors by arithmetical balancing.

b) Arithmetic increase method

This method is generally applicable to large and old cities. In this method, the average increase of population per decade is calculated from the past records and added to the present population to estimate population in the next decade. This method gives a low value and is suitable for well-settled and established communities.

c) Incremental increase method

In this method, the increment in arithmetical increase is determined from the past decades and the average of that increment is added to the average increase. This method gives increased values compared to the figures obtained by the arithmetical increase method.

d) Geometrical increase method

In this method, the percentage increase is assumed as the rate of growth and the average of the percentage increase is used to determine the increment in future population. This method gives a much higher value and is applicable to growing towns and cities having a vast scope of expansion.

e) Decreasing rate of growth

In this method, it is assumed that the rate of percentage increase decreases, and the average decrease in the rate of growth is calculated. The percentage increase is modified by deducting the decrease in the rate of growth. This method is applicable only to those cases where the rate of growth of population shows a downward trend.

f) Graphical method

There are two methods: in the first method, only the city in question is considered; and in the second method, other similar cities are taken into account.

i) Graphical method based on single city

In this method, the population curve of the city (i.e., the population vs. past decades) is smoothly extended for obtaining values for the future. The curve should be extended carefully; this requires vast experience and good judgement. The line of best fit may be obtained by the method of least squares.

ii) Graphical method based on cities with similar growth pattern

In this method, the city in question is compared with other cities that have already undergone the same phases of development, which the city in question is likely to undergo. Based on this comparison, a graph of populations versus decades is plotted and extrapolated.

g) Logistic method

The S shaped logistic curve for any city gives the complete trend of growth for the city right from beginning to the saturation limit of population of the city. This method is applicable to very large cities with adequate demographic data.

h) Method of density

In this approach, the trend in rate of increase in population density for each sector of a city is determined and population is forecast for each sector based on the above approach. Addition of population sector-wise, gives the population of the city.

2.6.2 Final Forecast

While the forecast of the population of a project area at any given time during the design period can be derived by any one of the foregoing methods appropriate to each case, the density and distribution of such population in several areas, zones or districts will again have to be estimated based on the relative probabilities of expansion in each zone or district, according to the nature of development and based on existing and contemplated town planning regulations. Wherever population growth forecast or master plans prepared by town planning authorities or other appropriate authorities are available, the design population should take these figures into account.

Floating population should also be considered which includes number of persons visiting the project area for tourism, pilgrimage or for working. The numbers should be decided in consultation with the tourism departments and specified for water supply and sewerage.

The worked out examples for estimation of future population by some of the methods are given in Appendix A.2.2.

2.7 PROJECT AREA

The factors that influence the determination of project area include natural topography, layout of buildings, political boundaries, economic factors, CMP, etc. For larger drainage areas, though it is desirable that the sewer capacities are designed for the total project area, sometimes the political boundaries and legal restrictions prevent construction of sewers beyond the limits of the local authority. However, when designing sewers for larger areas, there is usually an economic advantage in providing adequate capacity initially for a certain period of time and constructing additional sewers, when the pattern of growth becomes established. The need to finance projects within the available resources necessitates the design to be restricted to political boundaries. The project area under consideration should be marked on a key plan so that the area can be measured from the map.

2.8 REUSE AND DISPOSAL

The reuse of treated sewage should be given preference over disposal and the various options are discussed in Chapter 7 of this manual.

2.9 LAYOUT AND ARRANGEMENT OF SEWERAGE

The layout of collection systems shall resist the tendency to go in for underground sewerage flat out even in habitations that are only sparsely developed. The options of either time-deferred underground sewerage or incremental sewerage commensurate with the pace of development by such options as small bore, shallow sewers, twin drains, etc., to start with and eventual underground sewerage when habitations have been populated to a certain level where the revenue will be able to sustain the O&M.

The layouts by small communities shall be mandated to include the small bore sewer system / twin drain in both sides of roads, whereby the house side drain will receive the septic tank effluent and the road side drain will receive the storm water runoff. In metropolitan urban centres, decentralized sewerage shall be confined to institutional boundaries only and not culled out of habitations itself and zoning of sewerage with STPs fanning out radially outwards is to be encouraged.

A flat out choice of underground sewerage with sewers in middle of roads shall be discouraged and incremental sanitation as settled sewers, small-bore sewers, twin drain for septic tank effluents and sewers on shoulders of wide roads are to be evaluated as detailed in Chapter 3 of this manual.

2.10 LEGISLATION AND REGULATIONS

a. Water (Prevention and Control) Act, 1974

In this Act, it is necessary to obtain a consent to establish (CTE) from the Pollution Control Board (PCB) before starting the work of STP. Similarly, it is necessary to obtain the consent to operate (CTO) after completion of the construction and before actual operation. The CTE is based on whether the proposed STP design meets the discharge standards for treated sewage and the CTO is based on whether all the units originally committed are actually built and to the same size. Starting the construction without the CTE and starting the operation without CTO are punishable as an offence.

b. Environment (Protection) Act, 1986

The discharge standards for treated sewage, the noise standards governing the STP, the air emission standards governing the STP are prescribed in this act and are binding without exception. The PCB is empowered to tighten these standards wherever it is needed.

c. Municipal Bye-laws

Most municipal bye-laws stipulate that the owner of any property shall dispose of sewage in a proper manner without causing any nuisance to others. Wherever municipal sewers exist within a specified distance as per the respective bye-laws, it is obligatory that the sewage of the property be discharged into it. The bye-laws provide for action against defaulting owners.

d. Environment Impact Assessment

According to the Environment Impact Assessment (EIA) notification issued in 2006 by MoEF, this is not needed for sewerage projects. However, the concerned agencies are advised to maintain all the necessary facts and figures related to the total sanitation programme in the form of an effective and efficient Management Information Systems (MIS) which might be required in future under NUSP.

e. Indian Standards

The Bureau of Indian Standards (BIS) lays down quality levels of bought out items and construction quality and these shall not be diluted under any account. Wherever BIS are not available, internationally accepted standards may be used.

f. Town and Country Planning Act

The Town & Country Planning Act shall be mandatorily followed. Wherever there is a possibility, storm water drains on both sides of the road shall be built mandatorily.

2.11 GUIDELINES ON HOUSE SEWER CONNECTIONS

a) There is a compelling need to amend the bye-laws to make it compulsory for the population to avail house sevice sewer connection wherever public sewer is provided and if this is not forthcoming, the local authority shall effect the house sewer connection and institute revenue recovery proceedings.

b) Include house-service sewer connections as part of the sewerage project itself

c) Float Equated monthly instalments (EMI) schemes for repayment of house service sewer connection costs.

2.12 SURVEY AND INVESTIGATION

The survey and investigation are both pre-requisites for framing of the preliminary report and the preparation of a detailed project report (DPR) for any sewerage project. The engineering and policy decisions taken are dependent on the correctness and reliability of the data collected and its proper evaluation for preparing DPR to ensure success of the programme on long-term sustainable basis.

2.12.1 Basic Information

Broad knowledge of the problems likely to be faced during the various phases of the implementation of the project is essential for performing investigations effectively. Information on physical, fiscal, developmental and other aspects have to be collected.

The philosophy of survey is to rule out simple initial mistakes, which will make the entire project a blunder eventually. The entire geographical coverage of the project area relies very seriously on gravity transmission and eligible pathways, affordability by users, etc. The initial survey will chalk out the aspects to be considered and the aspects which have to be time deferred and the aspects which need to be relegated in each case.

2.12.1.1 Physical Aspects

These would necessitate the collection of information related to:

a) Topography or elevation difference needed for design of sewers and location of STP, outfall and disposal works

b) Subsoil conditions, such as types of strata likely to be encountered, depth of groundwater table and its fluctuations. In the absence of any records, preliminary data should be collected by carrying out at least 3 trial bores or 3 trial pits per hectare

c) Underground structures like storm drains and appurtenances, city survey stones, utility services like house connections for water supply & sewerage, electric & telephone cables and gas lines

d) Location of streets and adjoining areas likely to be merged or annexed

e) Contour map of the area to be superimposed on the village/town/city maps

f) Survey of India maps

g) Groundwater table and its fluctuations from local enquiries and past records

h) Underground utility services and Survey of India bench marks

i) Land use maps, density and trends of population growth and demographic studies

j) Type and number of industries for potential reuse and discharge of sewage

k) Existing drainage and sewerage facilities and data related to these facilities

l) Flow in sewers and sewers of similar areas to assess the flow characteristics

m) Historical and socio-economic data

n) Problems of maintenance of existing sewers

o) Effluent disposal sites and their availability

p) Earthquake

The possible sources of information are existing maps and plans showing streets from revenue or town surveys or the Survey of India maps.

Other sources are the topographical maps of survey of India if available with existing spot-levels, aerial photographs, photographs of complex surfaces for supplementing the existing instrumental surveys by concerned authorities like Municipalities and Roads Departments.

2.12.1.2 Survey of Natural Conditions

a) Societal preferences and local habits

b) Present status of the governmental, semi-governmental or municipal authority sponsoring the project, its capacity, adequacy, effectiveness and the desirability of its modification or necessity of a new organization to satisfactorily implement and maintain the project.

2.12.1.3 Survey on Related Plans

a) Sewerage master plan

b) Other related sewerage plans

c) Long-term comprehensive development plans for cities and towns

d) Urban planning

e) City planning area, urbanization zone, and urbanization control area

f) Land use plan

g) Road plan

h) Urban development as rezoning, residential estates, and industrial complexes

i) Design longitudinal section, transverse section

j) Design flood level and corresponding flood flow

k) Design low flood level and corresponding flow

l) Other plans.

2.12.1.4 Survey on Pollution Loads and Receiving Bodies

a) Survey on generated pollution load

b) Existing conditions and future plans related to water supply

c) Existing conditions and future plans related to industrial uses

d) Population, industrial production, agriculture, forestry and animal husbandry

e) Data on quality and quantity of sewage from large factories, offices, etc.

f) Data on sewage generated from sightseeing sources

g) Data on wells

h) Data on standard unit pollution loads from different sources

i) Survey to gather information on receiving water bodies

j) Data on existing water quality and flow in water bodies at the time of sampling

k) Data on environmental standards for water quality

l) Utilization of existing water bodies and future plans related to uses.

2.12.1.5 Survey on Existing Facilities

a) Underground installations

b) Existing sewerage and on-site sanitation facilities

c) Existing conditions of disposal of human waste

d) Existing conditions and alignment of road

e) Cultural assets and historic relics

f) Other existing facilities.

2.12.1.6 Survey on Resources of Sewerage System and its Utilization

a) Utilization of space in STP and SPS

b) The open space on top of STP structures or SPS is precious especially in highly populated cities and can be used for terrace garden, green houses.

c) Utilization of space in large sewers as conduits for optical fibre cables.

2.12.1.7 Survey on Treated Sewage, Sludge and Biogas Utilization

a) Reuse of treated sewage should be taken up after discussions between ULB, water boards, PHEDs / Jal Nigams and the public, as the case may be. Various possible reuse methods such as farm forestry, greenbelt development and lawns in road medians

b) Utilization of sludge in public areas is not possible due to issues of public acceptance and hence it is best to focus on farm forestry

c) Utilization of alternative energy, like in plant energy to be harnessed from biomethanation and to evaluate the ambient temperature suitability or heating of sludge vs. economics

d) Reuse of treated sewage to a minimum extent of 20 % by volume shall be mandatorily explored and the proposed use for achieving this 20 % target shall mandatorily form part of the CSP

e) Utilization of sludge as a construction material (as porous pavements, bricks, etc.).

2.12.1.8 Project Surveys

It should include the overall survey of the population, their historical outlook, their willingness for a change, acceptance of the concept to pay for the services, responsibility of local body under the national law of the land and above all, a public hearing on these issues.

2.12.1.9 Preliminary Project Surveys

This is concerned with the broad aspects of the project. Data on aspects such as capacity required, basic arrangement and size, physical features affecting general layout and design, availability of effluent disposal facilities, probable cost and possible methods of financing, shall be collected to prepare an engineering report describing the scope and cost of the project with reasonable accuracy. In framing such estimates, due consideration must be given to the escalation of prices of basic materials and their availability. While extreme precision and detail are not required in this phase, all the basic data obtained must be reliable.

2.12.1.10 Detailed Project Surveys

The surveys for this phase form the basis for the engineering design, as well as, for the preparation of plans and specifications for incorporation in the DPR. In contrast to preliminary survey, this survey must be precise and contain contours of all the areas to be served giving all the details that will facilitate the designer to prepare design and construction of plans suiting the field conditions. It should include, inter-alia, network of benchmarks and traverse surveys to identify the nature as well as extent of the existing underground structures requiring displacement, negotiation or clearance. Such detailed surveys are necessary to establish rights-of-way, minimize utility relocation costs, obtain better bids and prevent changing and rerouting of lines.

2.12.1.11 Construction Surveys

All control points such as base lines and benchmarks for sewer alignment and grade should be established by the engineer along the route of the proposed construction. All these points should be referred adequately to permanent objects.

a) Preliminary Layouts

Before starting the work, right-of-ways, work areas, clearing limits and pavement cuts should be laid out clearly to ensure that the work proceeds smoothly. Approach roads, detours, by-passes and protective fencing should also be laid out and constructed prior to undertaking sewer construction work. All layout work must be completed and checked before construction begins.

b) Setting Line and Grade

The transfer of line and grade from control points, established by the engineers, to the construction work should be the responsibility of the executing agency until work is completed. The methods generally used for setting the line and grade of the sewers are discussed in Chapter 3 of this manual. The procedures for establishing line and grade where tunnels are to be employed in sewer system are also discussed in Chapter 3 of this manual.

2.12.1.12 Developmental Aspects

The following should be taken into account:

a) Types of land use, such as commercial, industrial, residential and recreational uses; extent of areas to be served
b) Density of population, trends of population growth and demographic studies
c) Type and number of industries for determining quantity and nature of wastes, and locations of their discharge points
d) Existing drainage and sewerage facilities and data related to these facilities
e) Flow in existing sewers and sewers of similar areas to assess the flow characteristics
f) Historical and socio-economic data

g) Basis of design and information on the maintenance of existing sewers

h) Effluent disposal sites and their availability

Possible sources of information are census records, town and metropolitan master plans, city development plans, regional planning records, land use plan, flow gauging records, stream flow records, meteorological data and data from pollution control boards.

2.12.1.13 Fiscal Aspects

The various factors that will have an important bearing are:

a) Existing policies or commitments/obligations which may affect the financing of the project

b) Outstanding loan amounts and instalments of repayments

c) Availability of Central and State Government loans, grant-in-aid, loans from other financing bodies such as Life Insurance Corporation, Industrial Development Corporation, HUDCO, International Bank for Reconstruction and Development and other Banks and Institutions

d) Present water rates, sewer-tax and revenue realized from the service, size of property plots and land holding, the economic condition of community with respect to their tax-paying capacity

e) Factors affecting the cost of construction, operation and maintenance (O&M); some of the information can be obtained from the records related to Municipal and State Tax Levies, Acts and Rules governing loans, procedures for financing projects and registers and records of the authorities maintaining water supply and sewerage systems.

2.12.1.14 Other Aspects

The considerations that are likely to influence the planning of sewerage system are:

a) Changes in political boundaries by physical acquisition or merger of adjacent communities or by possible extension of limits

b) Feasibility of multi-regional or multi-municipal systems

c) Prevailing water pollution prevention statutes, other rules and regulations related to discharge of industrial and domestic wastes

d) Present status of the governmental, semi-governmental or municipal authority sponsoring the project, its capacity, adequacy, effectiveness and the desirability of its modification or necessity of a new organization to satisfactorily implement and maintain the project

e) Inconveniences likely to be caused to the community during execution and the feasibility of minimizing them by suitable alignment or location of the components of the sewerage system

Possible sources of information are National Acts, State and Municipal Laws and Bye-laws, minutes of the past meetings of the municipal or other governing bodies and discussions with officials, municipal councillors and other local leaders.

2.13 DETAILED PROJECT REPORT (DPR)

2.13.1 General

All projects have to follow distinct stages between the period they are conceived and completed. The various stages are:

- Pre-investment planning
  - Identification of a project
  - Survey and Investigation as described in clause 2.12
  - Preparation of project report
- Appraisal and sanction
- Construction of facilities and carrying out support activities
- Operation and maintenance
- Monitoring and feed back

2.13.1.1 Project Reports

Project reports deal with all aspects of pre-investment planning and establish the need as well as the feasibility of projects technically, financially, socially, culturally, environmentally, legally and institutionally. For big projects, economic feasibility may also have to be examined. Project reports should be prepared in three stages viz. (i) identification report (ii) pre-feasibility report and (iii) feasibility report. Projects for small towns or those forming parts of a programme may not require preparation of feasibility reports. Detailed engineering and preparation of technical specification and tender documents are not necessary for taking investment decisions, since these activities can be carried out during the implementation phase of projects. For small projects, however, it may be convenient to include detailed engineering in the project report, if standard design and drawing can be adopted.

Since project preparation is quite expensive and time consuming, all projects should normally proceed through three stages and at the end of each stage, a decision should be taken whether to proceed to the next planning stage and commit the necessary manpower and financial resources for the next stage. Report at the end of each stage should include a timetable and cost estimate for undertaking the next stage activity and a realistic schedule for all future stages of project development. It should be taken into consideration the time required for review and approval of the report, providing funding for the next stage, mobilizing personnel or fixing agency (for the next stage of project preparation) data gathering, physical surveys, site investigations, etc.

The basic design of a project is influenced by the authorities/organizations who are involved in approving, implementing, operating and maintaining the project. Therefore, the institutional arrangements, through which a project will be brought into operation, must be considered at the project preparation stage. Similarly responsibility for project preparation may change at various stages. Arrangements in this respect should be finalized for each stage of project preparation.

Sometimes, more than one organization may have a role to play in the various stages of preparation of a project. It is therefore necessary to identify a single entity to be responsible for overall management and coordination of each stage of project preparation. It is desirable that the implementing authority is identified and those responsible for operations of a project are consulted at the project preparation stage.

2.13.2 Identification Report

Identification report is basically a desk study, to be carried out relying primarily on the existing information. It can be prepared reasonably quickly by those who are familiar with the project area and needs of project components. This report is essentially meant for establishing the need for a project indicating likely alternatives, which would meet the requirements. It also provides an idea of the magnitude of cost estimates of a project to facilitate bringing the project in the planning and budgetary cycle and makes out a case for obtaining sanction to incur expenditure for carrying out the next stages of project preparation. The report should be brief and include the following information:

a) Identification of the project area and its physical environment

b) Commercial industrial, educational, cultural and religious importance and activities in and around the project area (also point out special activities or establishments like defence or others of national importance)

c) Existing population, physical distribution and socioeconomic analysis

d) Present sewage collection, treatment and disposal arrangements in the project area, pointing out deficiencies, if any, in system of collection and treatment

e) Population projection for the planning period, according to existing and future land use plans or master plans, if any

f) Establish the need for taking up a project in the light of existing and future deficiencies in sewage collection, treatment and disposal services, pointing out adverse impacts of non-implementation of the project, on a time scale

g) Bring out, how the project would fit in with the national / regional / sectoral strategies and with the general overall development in the project area

h) Identify a strategic plan for long-term development of sewage collection, treatment and disposal services in the project area, in the context of existing regional development plans and such other reports, indicating phases of development

i) State the objectives of the short-term project under consideration, in terms of population to be served and the impact of the project after completion, clearly indicating the design period

j) Identify project components, with alternatives if any; both physical facilities and supporting activities

k) Preliminary estimates of costs (component-wise) of construction of physical facilities and supporting activities, cost of operation and maintenance

l) Identify source for financing capital works and operation and maintenance, work out annual burden (debt servicing + operational expenditure)

m) Indicate institutions responsible for project approval, financing, implementation, operation and maintenance (e.g., Central Government, State Government, Zilla Parishad, Local Body, Water Supply Boards)

n) Indicate organization responsible for preparing the project report (pre-feasibility report, feasibility report), cost estimates for preparing project report and sources of funds to finance preparation of project reports

o) Indicate time table for carrying out all future stages of the project and the earliest date by which the project might be operational

p) Indicate personnel strength required and training needs for implementation of the project. Indicate if any particular/peculiar difficulties of policy or other nature that are likely to be encountered for implementing the project and how these could be resolved

q) Recommend actions to be taken to proceed further.

The following plans may be enclosed with the report:

i) An index plan to a scale of 1 cm = 2 km showing the project area, existing works, proposed works and location of community/township or institution to be served

ii) A schematic diagram showing the salient levels of project component

2.13.3 Pre-feasibility Report

After clearance is received, based on the identification report from the concerned authority and / or owner of the project and commitments are made to finance further studies, the preparation work on pre-feasibility report should be undertaken by an appropriate agency. This may be a central planning and design cell of the department dealing with the water and sewerage board, ULB, Jal Nigam or professional consultants working in the water supply, sanitation and environmental areas. In the latter case, terms of reference for the study and its scope should be carefully set out. Pre-feasibility study may be a separate and discrete stage of project preparation or it may be the first stage of a comprehensive feasibility study. In either case, it is necessary that it precedes taking up of a feasibility study because the pre-feasibility study is essentially carried out for screening and ranking of all project alternatives, and to select an appropriate alternative for carrying out the detailed feasibility study. The pre-feasibility study helps in selecting a short-term project, which will fit in the long-term strategy for improving services in the context of overall perspective plan for development of the project area.

A pre-feasibility report can be taken to be a Preliminary Project Report, the structure and component of which are as follows:

i) Executive summary

ii) Introduction

iii) The project area and the need for a project

iv) Long term plan for sewage collection, treatment and disposal

v) Proposed sewage collection, treatment and disposal project

vi) Conclusions and recommendations

vii) Tables, figures/maps and annexes

2.13.3.1 Executive Summary

It is a good practice to provide an executive summary at the beginning of the report, giving its essential features, basic strategy, approach adopted in developing the project and the salient features of financial and administrative aspects.

2.13.3.2 Introduction

This section explains the origin and concept of the project, how it was prepared and the scope and status of the report. These subsections may be detailed as under:

a) Project Genesis

i) Describe how the idea of the project originated, agency responsible for promoting the project.

ii) List and explain previous studies and reports on the project, including the project identification report and agencies which prepared them

iii) Describe how the project fits in the regional development plan, long-term sector plan, land use plan, public health care and sewage management programme, etc.

b) How was the Study Organized

i) Explain how the study was carried out, agencies responsible for carrying out the various elements of work and their role in preparing the study

ii) Time table followed for the study

c) Scope and Status of the Report

i) How the pre-feasibility report fits in the overall process of project preparation

ii) Describe data limitation

iii) List interim reports prepared during the study

iv) Explain the pre-feasibility report is intended to be used for obtaining approval for the proposed project

2.13.3.3 Project Area and the Need for the Project

This section establishes the need for the project. It should cover the following main items.

2.13.3.3.1 Project Area

i) Give geographical description of the project area with reference to maps

ii) Describe special features such as topography, climate, culture, religion, migration, etc., which may affect project design, implementation and operation

iii) Map showing administrative and political jurisdiction

iv) Describe any ethnic, cultural or religious aspects of the communities which may have a bearing on the project proposal.

2.13.3.3.2 Population Pattern

i) Estimate population in the project area, indicating the sources of data or the basis for the estimate

ii) Review previous population data, historic growth rates and causes

iii) Estimate future population growth with different methods and indicate the most probable growth rates and compare with past population growth trends

iv) Compare growth trends within the project area, with those for the region, state and the entire country

v) Discuss factors likely to affect population growth rate

vi) Estimate probable densities of population in different parts of the project area at future intervals of time e.g. five, ten and twenty years ahead

vii) Discuss patterns of seasonal migration, if any, within the area

viii) Indicate implication of the estimated growth pattern on housing and other local infrastructure.

2.13.3.3.3 Socio-Economic Aspects

i) Describe present living conditions of the people of different socio-economic and ethnic groups

ii) Identify locations according to income levels or other indications of socio-economic studies

iii) Show on the project area map, location-wise density of population, religion, poverty groups and ethnic concentrations and the present and future land uses (as per development plan)

iv) Information on housing conditions and relative proportions of owners and tenants

v) Provide data and make projection on housing standards and average household occupancy in various parts of the project area

vi) Provide data on education, literacy and unemployment by age and sex

vii) Describe public health status within the project area with particular attention to diseases related to water and sanitary conditions

viii) Provide data on maternal and infant mortality rates and life expectancy

ix) Discuss the status of health care programmes in the area, as well as other projects, which have bearing on improvements in environmental sanitation.

2.13.3.3.4 Sector Institutions

i) Identify the institutions (Government, Semi-Government and Non-Government) which are involved in any of the stages of water supply and sanitation project development in the area (Planning, preparing projects, financing, implementation, O&M and evaluation)

ii) Comment on roles, responsibilities and limitation (territorial or others) of all the identified institutions, in relation to water supply and sanitation (This may be indicated on a diagram).

2.13.3.3.5 Existing Sewage Collection, Treatment and Disposal Systems and Population Served

Describe each of the existing sewage collection, treatment and disposal systems (including conventional, decentralized, and on-site systems) in the project area, indicating the following details mentioned hereunder:

i) Area served, quantity and quality of sewage collected, components of the system such as collection network, SPS, STP, sewage reuse and disposal methods, etc.

ii) Private sewage disposal methods such as septic tanks, on-site toilets, etc.

2.13.3.3.6 Urban Drainage and Solid Wastes

Briefly describe existing systems of storm water drainage and solid waste collection and disposal. This discussion should be focused in terms of their impact on sewerage management and the environment.

2.13.3.3.7 Need for the Project

i) Comment as to why the existing system cannot satisfy the existing and projected demands for services with reference to population to be served

ii) Describe benefits of system improvements (which may include rehabilitation or developing a new system)

iii) Indicate priorities to improvement of existing system, expansion of systems, construction of new system, assessment of the need for consumer education in hygiene and comments on urgency of project preparation and implementation.

2.13.3.4 Long Term Plan for Sewage Collection, Treatment and Disposal

a) Sewage collection, treatment and disposal services have to be planned, as a phased development programme and any short-term project should be such that it would fit in the long-term strategy. Such a long-term plan or the strategic plan should be consistent with the future overall development plans for the areas. A long-term plan may be prepared for a period of 30 years and alternative development sequences may be identified to provide target service coverage at affordable costs. From these alternative development sequences, a priority project to be implemented in short term can be selected. It is this project, which then becomes the subject of a comprehensive feasibility study.

b) Alternative development sequences should be identified in the light of the coverage to be achieved during the planning period in phases. This calls for definition of the following:

i) Population to be covered with improved sewage management facility

ii) Target dates by which the above mentioned coverage would be extended within the planning period, in suitable phases

iii) Consistency and coordination to be maintained between projections for both water supply and sanitation services.

c) It must be noted that availability of funds is one of the prime factors which will ultimately decide the scope and scale of a feasible project

d) Selection of a Strategic Plan

Each of the alternative development sequences, which can overcome the existing deficiencies and meet the present and future needs, consist of a series of improvements and expansions to be implemented over the planned period. Since all the needs cannot be satisfied in the immediate future, it is necessary to carefully determine priorities of target groups for improvement in services and stages of development and thus restrict the number of alternatives.

e) Planning for system requirement includes consideration of the following:

i) Possibilities of rehabilitating and/or de-bottlenecking the existing systems

ii) Alternative treatment systems and pumping schemes

f) It may also be necessary to ascertain if supporting activities like health education, staff training and institutional improvements etc., are necessary to be included as essential components of the project. All the physical and supporting input need to be carefully budgeted (capital and operating) after preparing preliminary designs of all facilities identified for each of the development sequences. These may then be evaluated for least cost solution by ‘net present worth’ method, which involves expressing all costs (capital and operating) for each year in economic terms, discounting future costs to present value, selecting the sequence with the lowest present value.

g) As stated earlier, costs are to be expressed in economic terms and not in terms of their financial costs. This is because the various alternatives should reflect resource cost to the economy as a whole at different future dates. Costing of the selected project may however be done in terms of financial costs, duly considering inflation during project implementation.

2.13.3.5 Proposed Sewerage Project

a) Details of the Project

The project to be selected may consist of those components of the least cost alternative of development sequence, which can be implemented during the next 3 to 4 years. Components of the selected project may be as follows:

i) Rehabilitation and de-bottlenecking of the existing facilities

ii) Construction of new facilities for improvement and expansion of existing systems

iii) Support activities like training, consumer education, public motivation, etc.

iv) Equipment and other measures necessary for operation and maintenance of the existing and expanded systems

v) Consultancy services needed (if any) for conducting feasibility study, detailed engineering, construction supervision, socio-economic studies and support activities.

b) Project Components

All project components should be thoroughly described, duly supported by documents such as:

i) Location maps

ii) Technical information for each physical component and economic analysis where necessary

iii) Preliminary engineering designs and drawings in respect of each physical component, such as collection network, SPS, STP and disposal system.

c) Implementation Schedule

A realistic implementation schedule should be presented, taking into consideration time required for all further steps to be taken, such as conducting feasibility study, appraisal of the project, sanction to the project, fund mobilization, implementation, trial and commissioning. In preparing this schedule due consideration should be given to all authorities/groups whose inputs and decisions can affect the project and its timing.

d) Cost Estimates

Cost estimates of each component of the project should be prepared and annual requirement of funds for each year should be worked out, taking into consideration the likely annual progress of each component. Due allowance should be made for physical contingencies and annual inflation. This exercise will result in arriving at total funds required annually for implementation of the project.

e) Pre-feasibility Report

The pre-feasibility report should bring out any major environmental and social impact the project is likely to cause and if these aspects will affect its feasibility (Refer to section 2.13.3 of this manual).

f) Institutional Responsibilities

The pre-feasibility report should identify the various organizations/departments/agencies that would be responsible for further planning and project preparation, approval, sanction, funding, implementation, O&M of the project and indicate the manpower needed to implement and later operate and maintain the project. It should also discuss special problems likely to be encountered during O&M, in respect of availability of skilled and technical staff, funds, transport, chemicals, communication, power, spare parts, etc. Quantitative estimates of all these resources should be made and included in the project report.

g) Financial Aspects

The capital cost of a project is the sum of all expenditure required to be incurred to complete design and detailed engineering of the project, construction of all its components including support activities and conducting special studies. After estimating component-wise costs, they may also be worked out on annual basis throughout the implementation period, taking into consideration construction schedule and allowances for physical contingencies and inflation. Basic item costs to be adopted should be of the current year. Annual cost should be suitably increased to cover escalation during the construction period. Total of such escalated annual costs determines the final cost estimate of the project. Financing plan for the project should then be prepared, identifying all the sources from which funds can be obtained and likely annual contribution from each source, until the project is completed. The possible sources of funds include:

i) Cash reserves available with the project authority
ii) Grant-in-aid from government
iii) Loans from government
iv) Loans from financing institutions like Life Insurance Corporation, Banks, HUDCO, etc.,
v) Open market borrowings
vi) Loans/grants from bilateral/international agencies
vii) Capital contribution from voluntary organization or from consumers

h) Interest on Loan

If the lending authority agrees, interest payable during implementation period can be capitalized and loan amount increased accordingly.

i) Recurring Expenditure

The next step is to prepare recurrent annual costs of the project for the next few years (approximately 10 years) covering O&M expenditure of the entire system (existing and proposed).

This would include expenditure on staff, chemicals, energy, spare parts and other materials for system operation, transportation, up-keep of the systems and administration. The annual financial burden imposed by a project comprises the annual recurring cost and payment towards loan and interest (debt servicing) less the revenue derived from taxes, tariffs, etc.

j) Financing Plan

Every State Government and the GOI have schemes for financing water supply and sewage collection, treatment and disposal schemes in the urban and rural areas and definite allocations are made for the national plan periods. It will be necessary at this stage to ascertain if and how much finance can be made available for the project under consideration and to estimate the annual availability of funds for the project until its completion. This exercise has to be done in consultation with the concerned department of the Government and the lending institutions, which would see whether the project fits in the sector policies and strategies, and can be brought in an annual planning and budgetary cycle, taking into consideration the commitments already made in the sector and the overall financial resource position. The project may be finally sanctioned for implementation if the financing plan is firmed up.

2.13.3.6 Conclusion and Recommendations

a) Conclusions

This section should present the essential findings and results of the pre-feasibility report. It should include a summary of the following main items:

i) Existing coverage

ii) Review of the need for the project

iii) Long-term development plans considered

iv) Recommended project, and its scope in terms of coverage and components

v) Priorities concerning target-groups and areas to be served by the project

vi) Capital costs and tentative financing plan

vii) Annual recurring costs and debt servicing and projection of operating revenue

viii) Urgency for implementation of the project

ix) Limitation of the data/information used and assumption and acknowledgements made and need for in-depth investigation, survey and revalidation of assumptions and judgments, while carrying out the feasibility study.

The administrative difficulties likely to be met with and risks involved during implementation of the project should also be commented on. These may pertain to boundary of the project area, availability of land for constructing project facilities, coordination with the various agencies, acceptance of service by the beneficiaries, shortage of construction materials, implementation of support activities involving peoples' participation, supply of power, timely availability of funds for implementation of the project and problems of O&M of the facilities.

b) Recommendations

i) This should include all actions required to be taken to complete project preparation and implementation, identifying the agencies responsible for taking these actions. A detailed timetable for actions to be taken should be presented. If found necessary and feasible, taking up works for rehabilitating and/or de-bottlenecking the existing system should be recommended as an immediate action. Such works may be identified and cost be estimated so that detailed proposals can be developed for implementation.

ii) It may also be indicated whether the project authority can go ahead with taking up detailed investigations, data collection, operational studies, without undertaking feasibility study formally.

iii) In respect of small and medium size projects, the pre-feasibility report can be considered sufficient for obtaining investment decision for the project if:

- The results of the pre-feasibility study are based on adequate and reliable data / information. The analysis of the data and situation is carried out fairly intensively,
- No major environmental and social problems are likely to crop up that might jeopardize project implementation, and
- No major technical and engineering problems are envisaged during construction and operation of the facilities.

iv) In that case, the pre-feasibility study with suitable concluding report should be processed for obtaining investment decision for the project. The feasibility study can then be taken up at the beginning of the implementation phase and if results of the study are noticed to be at variance with the earlier ones, suitable modification may be introduced during implementation.

v) In respect of major projects however and particularly those for which assistance from bilateral or international funding agencies is sought for, comprehensive feasibility study may have to be taken up before an investment decision can be taken.

2.13.4 Feasibility Report

Feasibility study examines the project selected in the pre-feasibility study as a short-term project in much detail, to check if it is feasible technically, financially, economically, socially, legally, environmentally and institutionally.

Enough additional data/information may have to be collected to examine the above mentioned aspects, though the details necessary for construction of project components may be collected during execution of works.

It is a good practice to keep the authority responsible for taking investment decision, informed of the stage and salient features of the project. If there are good prospects of the project being funded immediately after the feasibility study is completed, detailed engineering of priority components may be planned simultaneously.

The feasibility report may have the following sections:

a) Background

b) Proposed project

c) Institutional and financial aspects

d) Techno Economic Appraisal Procedure

e) Conclusion and recommendations

2.13.4.1 Background

This section describes the history of project preparation, how this report is related to other reports and studies carried out earlier, and in particular it's setting in the context of a pre-feasibility report.

It should also bring out if the data/information and assumptions made in the prefeasibility report are valid, and if not, changes in this respect should be highlighted. References to all previous reports and studies should be made.

In respect of the project area, need for a project and strategic plan for the same, only a brief summary of the information covered in pre-feasibility report should be presented, highlighting such additional data/information if any collected for this report.

The summary information should include planning period, project objectives, service coverage, service standards considered and selected for long-term planning and for the project, community preferences and affordability, quantification of future demands for services, alternative strategic plans, their screening and ranking, recommended strategic plan and cost of its implementation.

2.13.4.2 Proposed Project

This section describes details of the project recommended for implementation. Information presented here is based on extensive analysis and preliminary engineering designs of all components of the project. The detailing of this section may be done in the following subsections.

a) Objectives

Project objectives may be described in terms of general development objectives such as health improvements, ease in sewerage management, improved environmental conditions, human resources development, institutional improvements and terms of specific objectives such as coverage of various target groups.

b) Project Users

Define number of people by location and institutions who will benefit and/or not benefit from the project area and reasons for the same, users involvement during preparation, implementation and operation of the project.

c) Rehabilitation and De-bottlenecking of the Existing Sewerage System

In fact, rehabilitation, improvements and de-bottlenecking works, if necessary, should be planned for execution prior to that of the proposed project. If so these activities should be mentioned in the feasibility report, if however these works are proposed as components of the proposed project, the necessity of undertaking the rehabilitation / improvement de-bottlenecking works should be explained.

d) Project Description

This may cover the following items in brief:

i) Definition of the project in the context of the recommended development alternative (strategic plan) and explanation for the priority of the project

ii) Brief description of each component of the project, with maps and drawings

iii) Functions, location, design criteria and capacity of each component

iv) Technical specification (dimension, material) and performance specifications

v) Stage of preparation of designs and drawings of each component

vi) Constructing in-house facilities

vii) Method of financing

viii) Existing benchmarks (for relevant indicators mentioned in the "Handbook on Service Level Benchmarking", MoUD) and benchmarks expected to be achieved after implementation of the project should be mentioned in the report. The indicators included in above reference are given in Table 2-2.

Table 2-2 Service level benchmarks for sewage management

Sl. No.	Proposed Indicator	Benchmark	Sl. No.	Proposed Indicator	Benchmark
1	Coverage of toilets	100%	6	Extent of reuse and recycling of sewage	20%
2	Coverage of sewage network services	100%	7	Efficiency in redressal of customer complaints	80%
3	Collection efficiency of the sewage network	100%	8	Extent of cost recovery in sewage management	100%
4	Adequacy of sewage treatment capacity	100%	9	Efficiency in collection of sewage charges	90%
5	Quality of sewage treatment	100%			

Source:MoUD, 2011

e) Support Activities

Need for and description of components such as staff training, improving billing and accounting, consumer education, health education, community participation, etc., and timing of undertaking these components and the agencies involved should be included.

f) Integration of the Proposed Project with the Existing and Future Systems

Describe how various components of the proposed project would be integrated with the existing and future works.

g) Agencies Involved in Project Implementation and Relevant Aspects

i. Designate the lead agency

ii. Identify other agencies including government agencies, who would be involved in project implementation, describing their role, such as granting administrative approval, technical sanction, approval to annual budget provision, sanction of loans, construction of facilities, procurement of materials and equipment, etc.,

iii. Outline arrangements to coordinate the working of all agencies

iv. Designate the operating agency and its role during implementation stage

v. Role of consultants, if necessary, scope of their work, and terms of reference

vi. Regulations and procedures for procuring key materials and equipment, power, and transport problems, if any

vii. Estimate number and type of workers and their availability

viii. Procedures for fixing agencies for works and supplies and the normal time it takes to award contracts

ix. List of imported materials, if required, procedure to be followed for importing them and estimation of delivery period

x. Outline any legislative and administrative approvals required to implement the project, such as those pertaining to environmental clearance, prescribed effluent standards, acquisition of lands, permission to construct across or along roads and railways, high-tension power lines, in forest area and defence or other such restricted areas

xi. Comment on the capabilities of contractors and quality of material and equipment available indigenously

h. Cost Estimates

i. Outline basic assumptions made for unit prices, physical contingencies, price contingencies and escalation

ii. Summary of estimated cost of each component for each year till its completion and work out total annual costs to know annual cash flow requirements

iii. Estimate foreign exchange cost if required to be incurred

v. Work out per capita cost of the project on the basis of design population, cost per unit of sewage treated and disposed and compare these with norms, if any, laid down by government or with those for similar projects

i) Implementation Schedule

Prepare a detailed and realistic implementation schedule for all the project components, taking into consideration stage of preparation of detailed design and drawings; additional field investigations required, if any; time required for preparing tender documents; notice period; processing of tenders; award of works / supply contract; actual construction period; period required for procurement of material and equipment; testing; trials of individual components; and commissioning of the facilities, etc.

If consultant's services are required, the period required for completion of their work should also be estimated. A detailed PERT/CPM network showing implementation schedule for the whole project, as well as those for each component should be prepared, showing linkages and inter-dependence of various activities.

Implementation schedule should also be prepared for support-activities such as training, consumer education, etc., and their linkages with completion of physical components and commissioning of the project should be established.

j) Operation and Maintenance of the Project

Estimate annual operating costs considering staff, chemicals, energy, transport, routine maintenance of civil works, maintenance of electrical/mechanical equipment, including normal cost of replacement of parts and supervision charges. Annual cost estimates should be prepared for a period of 10 years from the probable year of commissioning the project, taking into consideration expected coverage and escalation.

Procedure for monitoring and evaluating the project performance with reference to project objectives should be indicated.

2.13.4.3 Institutional and Financial Aspects

a) Institutional Aspects

It is necessary to examine capabilities of the organizations that would be entrusted with the responsibility of implementing the project and operating the same after it is commissioned. The designated organization(s) must fulfill the requirements in respect of organizational structure, personnel, financial, health and management procedures, so that effective and efficient performance is expected. This can be done by describing the following aspects:

i) History of the organization, its functions, duties and powers, legal basis, organization chart (present and proposed), relationship between different functional groups of the organization and with its regional offices, its relation with government agencies and other organizations involved in sector development

ii) Public relations in general and consumer relations in particular, extension services available to sell new services, facilities for conducting consumer education programme and settling the complaints

iii) Systems for budgeting capital & recurring expenditure, revenue, accounting expenditure & revenue, internal & external audit arrangements and inventory management

iv) Present positions and actual staff, comments on number and quality of staff in each category, ratio of staff proposed for maintenance and operation of the project to the population served, salary ranges of the staff and their comparison with those of other public sector employees

v) Staff requirement (category wise) for operating the project immediately after commissioning, future requirements, policies regarding staff training, facilities available for training

vi) Actual tariffs for the last 5 years, present tariff, tariff proposed after the project is commissioned, its structures, internal and external subsidies, procedure required to be followed to adopt new tariff, expected tariff and revenues in future years, proposal to meet the shortage in revenues

vii) Prepare annual financial statements (income statements, balance sheets and cash flows) for the project operating agency for five years after the project is commissioned; explain all basic assumptions for the financial forecast and the terms and conditions of tapping financial sources; demonstrate ability to cover all operating and maintenance expenditure and loan repayment, workout rate of return on net fixed assets and the internal financial rate of return of the project

b) Financing Plan

Identify all sources of funds for implementation of the project, indicating year-by-year requirements from these sources, to meet expenditure as planned for completing the project as per schedule, state how interest during construction will be paid, or whether it will be capitalized and provided for in the loan, explain the procedures involved in obtaining funds from the various sources.

2.13.4.4 Techno Economic Appraisal Procedure

The decision between technologies of sewerage as well as sewage treatment should be carried out on life cycle analysis of major components. In general, the life cycle of civil works can be taken as 30 years and that of equipment can be taken as 15 years in non-sewage treatment locations and 10 years in sewage treatment locations. The analysis should include:

a) Net Present Value (NPV) of capital costs

b) Equivalent cost of annuity and O&M costs

c) Revenue recoverable if any by way of by-products

d) Land Cost

e) Dependency on Imports for day to day spares

f) Import substitution

g) Time required to achieve the desired project objectives

h) Mitigation of any adverse environmental impacts

i) Long term sustainability by the finances of the ULB

While aspects of a) through d) can be attributed to numerical values, the aspects e) through i) will be subjective and has to be appraised based on higher weightage for most preferred technologies.

Thus, the exercise of techno-economic appraisal is not a fully mathematical approach and has to be tempered as two interdependent aspects, both kept up and reasoned out interactively. The tendency to overly complicate the exercise with undue mathematics shall be resisted.

2.13.4.5 Conclusion and Recommendations

This section should discuss justification of the project, in terms of its objectives, cost effectiveness, affordability, willingness of the beneficiaries to accept the services and effect of not proceeding with the project.

Issues that are likely to adversely affect project implementation and operation should be outlined and ways of tackling the same should be suggested. Effect of changes in the assumptions made for developing the project on project implementation period, benefits, tariff, costs and demand, etc., should be mentioned.

Definite recommendations should be made regarding time-bound actions to be taken by the various agencies, including advance action that may be taken by the lead agency pending approval and financing of the project.

2.14 PLANNING OF SEWERAGE SYSTEM

2.14.1 Approach

The approach to planning of sewerage shall be governed by optimum utilization of the funds available such that the sewerage system when completed does not become unused for long and at the same time does not become inadequate very soon.

2.14.2 Design Population Forecast

This shall be as per the methods in Chapter 3 of this manual and its validation with respect to known growths in recent decades and evolving a reasonable basis by comparing with other similar habitations. There is no hard and fast mathematical basis for this and the methods in Chapter 3 are only a guideline.

2.14.3 Estimation of Sewage Flow

This shall be as per the methods in Chapter 3 of this manual and its validation with respect to known growths in recent decades and evolving a reasonable basis. The design population having been established, the judgement of per capita water supply is the key.

2.14.4 Sewage Characteristics and Pollution Load

The raw sewage characteristics are a function of the level of water supply and per capita pollution load as shown in Chapter 5 of this manual. Thus, the level of water supply decides the concentration of pollutants.

The pollutant load from a given habitation expressed as kg/day will remain the same but the concentration will vary depending on the level of water supply. Where the actual level of water supply is not foreseeable, the desirable level as in Chapter 3 shall be followed.

2.14.5 Planning of Sewer System

The design principles in Chapter 3 of this manual shall be followed. In essence, it stipulates that the options of small bore sewers, shallow sewers, twin drains and underground sewers all have to be relatively evaluated to sub regions of the project site instead of blindly going in for total underground sewer flat out. Incremental sewer shall also be considered based on the phased development of the region instead of directly opting for total underground sewer system.

2.14.6 Planning of Pumping Station

The design principles in Chapter 4 of this manual shall be followed. In essence, it stipulates that the options of horizontal foot mounted centrifugal pump sets in a dry well adjacent to wet well has its importance in shallow lift smaller capacity pump stations and submersible pump sets are not a panacea for all applications. In addition, the twin wet well concept for degritting shall be considered.

2.14.7 Planning of Sewage Treatment Facilities

The design principles in Chapter 5 of this manual shall be followed. In essence, it stipulates that the choice of conventional systems as also recent emerging trends can also be considered provided the costs of the latter are ascertained from recent contracts in the country and not arbitrarily based on quotes from vendors of these technologies.

2.15 PLANNING OF SLUDGE TREATMENT AND UTILIZATION

2.15.1 Basic Philosophy of Sludge Treatment

Sludge in STPs generally refers to the biological organisms, which have a tendency to decay and putrefy and as otherwise has its value as soil fillers in agriculture and biomethanation. The philosophy shall be to opt for the biomethanation route and derive electricity by igniting the methane gas in specially designed gas engines.

2.15.2 Design Sludge Generation

The design principles in Chapter 6 of this manual shall be followed. In essence, it stipulates the quantity and volatile portion of the sludge solids given by the BOD load. The numerical design guidelines are more easily followed than the theoretical derivations.

2.15.3 Planning for Sludge Reuse

Sludge reuse is to be considered for biomethanation and using the methane gas to produce electricity and the digested sludge as soil filler in agriculture or farm forestry. The latter use as soil filler may not be easily possible in metropolitan urban centres for want of the land. Transportation of the sludge outside the limits of the metropolitan area is never easy, as the public there will object to this. Hence, methods such as pellets to marketable soil fillers or composted organic fertilizers shall have to be explored, though this is new to India.

However, the use of treated sewage sludge for land application shall be subject to its compliance to section 6.10.2.1 and section 6.10.2.1.1 of chapter 6 of this manual

2.15.4 Common Sludge Treatment Facilities

Common sludge treatment implies that sewage sludge generated in two or more STPs are collected in one STP and treated there. It is an effective method of sludge treatment for urban areas, where the land acquisition for STPs is difficult. However, while planning it is important to consider that transportation / collection of sludge is difficult.

2.15.5 Transportation and Disposal of Sludge

The practice of transporting of wet sludge in tankers and spraying onto agriculture fields are reported to be in vogue in developed western countries where such lands are in plenty. However, this practice is not recommended for India because of the fact that in the arid temperatures in most parts of the country this may set off an unintentional cycle of airborne aerosol infection. Thus transportation, if ever to be carried out, shall have to be only in the form of dewatered sludge cake to at least a solid content of about 25%. The disposal shall have to be for eco-friendly purposes as agriculture or farm forestry or pellets for marketing as supplemented organic fertilizers.

However, the use of treated sewage sludge for land application shall be subject to its compliance to section 6.10.2.1 and section 6.10.2.1.1 of chapter 6 of this manual.

2.16 PLANNING OF UTILIZATION OF RESOURCES AND SPACE

2.16.1 Planning of Utilization of Space in Sewage Pumping Stations and Treatment Plant

The open spaces in STPs and SPS, especially roof-tops, shall be used for horticulture, sports facilities/playgrounds, parks, etc. This will help utilization of such space in densely populated cities.

2.17 PLANNING FOR RECONSTRUCTION

The facilities get older with the passage of time and at some stage, they are not able to function at the desired level of performance.

It becomes necessary to carry out rehabilitation or reconstruction work to make them work properly. For this purpose, a reconstruction plan is to be anticipated and developed in the planning stage itself.

2.17.1 General Aspects of Reconstruction Planning

By definition, reconstruction arises when the original construction has become either useless or is damaged due to earthquakes or floods. In such situations, the single most important requirement is the records of “as constructed” drawings, which show the approved drawings with endorsements of whatever changes have been carried out in construction. In the absence of these drawings, it is impossible to understand why and how the construction failed. This is most important as the original drawing has been approved based on a set of standards but still it has failed and hence, there are important issues to be understood. Thus, the most important aspect of reconstruction planning is the documentation of “as constructed” drawings and the original design. The next important need is to build a ready reference of past construction failures and the reasons and reconstruction history. Perhaps the more important aspect is to encourage the engineers to be frank of their unintentional lapses and treat this as human error and not to flog even trivial lapses as major flaws.

2.17.2 Reconstruction Planning of Sewers

Almost everything written in Section 2.17.1 applies here also. In addition, the following are relevant:

a) A mandatory record of the fate of gases inside major sewers monitored and chronicled for ready reference

b) An ultra-sonic survey of major sewers once a year to maintain a record of the integrity of the sewers and the weakness that may be occurring in some sections

c) A procedure for alternative diversion of sewage flow by temporary submersible pump sets in the upstream manhole of a damaged portion to the downstream manhole, thus permitting the repairs to the damaged section

d) A procedure to plug the manholes on both ends of the damaged sewer using pneumatic plugs similar to football or automobile tubes

e) A standardised schedule of rates for such emergency work

f) A stock of well point system for dewatering the damaged portion

g) The most important plan in facing a failure of a trunk sewer is to realize that ground water may be polluted by seepage of raw sewage. Thus, priority is to route the sewage in another trunk sewer to shut off the incoming raw sewage immediately and divert the same to another destination even if it means overloading the trunk sewer where it is diverted.

h) While designing the sewer system itself, trunk sewers shall be designed to be possible to be used for such diversions by temporarily using the sewer as a pumping line under low pressure. After all, these sewers are laid using long sewer pipes of 6 m length, and the load carrying capacity needs a rating of at least about 4 kg/sq. cm and this is adequate for such low head pumped diversions. Temporary pumping lines of low pressure can be laid above ground along property boundary compound walls by using double-flanged DI pipes which are easy to lay and dismantle.

2.17.3 Reconstruction Planning of SPS and STP

The reconstruction plan for SPS and STP has to address the following issues:

a) In both these installations, the reconstruction applies largely to sewage retaining civil works only, because in the case of mechanical and electrical equipment, it is replacement and not reconstruction. Replacement does not require great skill. Reconstruction of sewage holding structures requires very great skills and experience and this includes piping and valves.

b) The importance of record keeping of "as constructed" drawings as stated earlier in Section 2.17.1 is very much important in this case also.

c) The most important plan in facing a failure of a sewage holding civil work as tank is to realize that ground water may be polluted by seepage of raw sewage and thus shut off the incoming raw sewage immediately and divert the same to another destination even if it means overloading the new destination.

d) While designing the sewer system itself, the pumping mains shall be designed to be possible to use for such diversions by temporarily overloading another trunk sewer. After all, these sewers are laid using long sewer pipes of 6 m length, and the load varying capacity needs a rating of at least about 4 kg/sq. cm and this is adequate for such low head pumped diversions. This may not be possible in large metropolitan centres but must be possible in class II and class III cities.

e) In the case of reconstruction of sewage holding structures, the best is to abandon the damaged structure, strengthen its foundation and inscribe a new structure. This may result in a loss of volume by about 10% but that is nothing to be taken seriously.

2.18 ENVIRONMENTAL PRESERVATION AND BEAUTIFICATION

2.18.1 In Sewer Systems

Most often, the slimy matter taken out of the manholes is left on the road edges itself and this creates a health hazard. In planning stage itself, solutions by way of driving trucks to collect all these to a central facility close to the municipal solid wastes dump-site has to be recognized. Accordingly, in the planning stage itself provisions shall be made in the estimates for procuring a set of mobile trucks that can be deployed in such situations, as no commercial truck will come forward to remove such muck from sewage manholes.

2.18.2 In Sewage Treatment Plant and Pumping Stations

Suitable provisions for greening of the premises shall be made in the cost estimation stage itself.

2.18.3 Environmental Preservation Measures of Surrounding Area

The fuel and energy available in the treated sewage and sludge in sewerage system can be utilized to contribute to energy conservation in the area. The reduction in energy consumed by the sewerage facilities can indirectly contribute to the prevention of global warming.

In order to preserve the environment of a city and to have positive impact on global environment, it is necessary to use various functions of the sewerage system as described below.

a. Preservation of water quality

In order to plan water quality conservation of a close natural water area, it is necessary to promote introduction of advanced treatment process. It is necessary to promote introduction of efficient advanced processing technology at sustainable cost. Moreover, it is important to plan the public awareness such that the ratio of pollution discharged without treatment is reduced gradually.

b. Use of resurgent water, rain water

Cooling the road and building by resurgent water, rain water, etc., can be planned.

c. Utilization of resources and energy

The practically feasible utilization of resource including treated sewage and sludge can be planned to avoid draining of water and nutrient.

d. Energy conservation measures

The introduction of energy-saving equipment in sewerage facilities can be thought of as the first energy conservation measures. This can be done while updating of apparatus and equipment. It is also important to aim at energy saving by improving the operating method of existing facilities.

e. Reduction of greenhouse gas

A lot of greenhouse gases (e.g., methane, CO2, etc.,) is discharged in sewerage systems. Measures to reduce such emission can be planned.

2.19 ENGINEERING PLANS

2.19.1 Plans

All plans for sewerage facilities should be in a well-organized format and bear a suitable title showing the name of the municipality, sewer district and organization.

They should show the scale in metric measure, a graphical scale, the north point, date, and the name and signature of the engineer. A space should be provided for signature and / or approval stamp of the appropriate reviewing authority.

The plans should be clear and legible. They should be drawn to a scale, which will permit all necessary information to be plainly shown. Datum used should be indicated. Locations and logs of test borings, when required, should be shown on the plans.

Detail plans should consist of plan views, elevations, sections, and supplementary views, which together with the specifications and general layouts, provide the working information for the contract and construction of the facilities. They should also include dimensions and relative elevations of structures, location, equipment, size of piping, water levels and ground elevations.

2.19.2 Specifications

Complete signed technical specifications should be prepared and submitted for the construction of sewers, SPS, STP, and all other appurtenances, and should accompany the plans. The detailed specifications accompanying construction drawings should include, but not be limited to, detailed specifications for the approved procedures for operation during construction, related construction information not shown on the drawings, which is necessary to inform the builder in detail of the design requirements for the quality of materials, workmanship, and fabrication of the project. The specifications should also include: the type, size, strength, operating characteristics, and rating of equipment; allowable infiltration; the complete requirements for all mechanical and electrical equipment, including machinery, valves, piping, and jointing of pipe; electrical apparatus, wiring, instrumentation, and meters; laboratory fixtures and equipment; operating tools, construction materials; special filter materials, such as, stone, sand, gravel, or slag; miscellaneous appurtenances; chemicals when used; instructions for testing materials and equipment as necessary to meet design standards; and performance tests for the completed facilities and component units. It is suggested that these performance tests be conducted at design load conditions wherever practical.

2.19.3 Revisions to Approved Plans

In case if the project is prepared and approved and due to some reason, the implementation is not started for a long period (say 5 - 10 years), some of the important factors affecting generated amount of sewage, such as population, water supply coverage, etc. will change. In such cases, revision of the approved plan will be required and approval shall be required again. Moreover, any deviations from approved plans or specifications affecting capacity, flow, operation of units, or point of discharge shall be approved, in writing, before such changes are made. Revised plans or specifications should be submitted well in advance of any construction work, which will be affected by such changes to allow sufficient time for review and approval. Structural revisions or other minor changes not affecting capacities, flows or operation can be permitted during construction without approval. “As built” plans clearly showing such alterations shall be submitted to the reviewing authority at the completion of the work.

2.20 CHECKLIST

The MoUD website http://urbanindia.nic.in/programme/uwss/dprs-checklists/sews.pdf contains the checklist for the preparation of DPR for sewerage schemes which require financial assistance. This checklist can be referred to and shall be complied with.

2.21 MANDATORY REQUIREMENTS IN SANITATION SECTOR

These shall be as follows

1. Each state government shall mandatorily pass a “Sewerage & Sanitation act” and notify the rules thereunder. The reason for this is to empower the ULB’s to prevail on the property owners/ occupiers to avail house service sewer connections once the sewerage system is developed by the ULB, within 30 m of the premises irrespective of whatever be the mode of existing sewage disposal system.

In case the owner/occupier fails to do so the ULB by virtue of its powers can disconnect essential services like, water supply and electricity after the expiry of the notice period. An example of such a provision can be seen under rule 10(5) of the Goa Sewerage System and sanitation services management rules 2010 enacted under the Goa Sewerage system and services management act 2008 as contained in Appendix C. 2-2 of Part C Management.

2. Similarly, each state government shall mandatorily formulate and notify appropriate act and rules for septage management.

CHAPTER 3: DESIGN AND CONSTRUCTION OF SEWERS

3.1 GENERAL

The major roles of a sewer system can be listed as follows:

– Improvement in the environment by removing the sewage as it originates

– Preventing inundation of low lying areas that may be otherwise caused by not providing sewers

– Prevention of vector propagation by sewage stagnations

– Avoiding cross connections with freshwater sources by seepage

In addition, there is a strong emphasis on:

a) Avoiding sewer impacts on groundwater quality by infiltration of soil water into sewers and exfiltration of sewage into soil water, occurring rather as a cycle depending on the flow conditions in leaky sewers, and

b) Moving away from the mind-set that a sewer system shall necessarily be an underground sewer right in the middle of the road with costly construction, upkeep and remediation and making the objective realizable if necessary in an incremental sewerage commensurate with optimizing the area coverage in the available financial and human resources to create and sustain the system.

This chapter presents the following:

Part - 1 Estimation of Design Flows

Part - 2 Types and Hydraulics of Sewers

Part - 3 Design of Sewer Networks

Part - 4 Types and Construction of Manholes

Part - 5 Laying, Jointing and Construction of Sewers

PART - 1 ESTIMATION OF DESIGN FLOWS

3.2 DESIGN PERIOD

The length of time up to which the capacity of a sewer will be adequate is referred to as the design period. In fixing a design period, consideration must be given for the useful life of structures and equipment employed, taking into account obsolescence as well as wear and tear. The flow is largely a function of the population served, population density, water consumption, lateral and sub main sewers are usually designed for peak flows of the population at saturation density as set forth in the master plan. Trunk sewers, interceptors, and outfalls are difficult and uneconomical to be enlarged or duplicated and hence are designed for longer design periods. In the case of trunk sewers serving relatively undeveloped areas adjacent to metropolitan areas, it is advisable to construct initial facilities for more than a limited period. Nevertheless, right of way for future larger trunk sewers can be acquired or reserved. The recommended design period for various components shall be as in Table 2-1.

3.3 POPULATION FORECAST

Methods of estimation of population for arriving at the design population have been discussed in Section 2.6. When a master plan containing land use pattern and zoning regulations is available for the town, the anticipated population can be based on the ultimate densities and permitted floor space index provided for in the master plan.

In the absence of such information on population, the following densities are suggested for adoption as in Table 3.1.

Table 3.1 Densities of Population vs. Populated areas

Size of town (Population)	Density of population per hectare
Up to 5,000	75-150
Above 5,001 to 20,000	150-250
Above 20,001 to 50,000	250-300
Above 50,001 to 1,00,000	300-350
Above 1,00,001	350-1,000

Source: CPHEEO, 1993

In cities where Floor Space Index (FSI) or Floor Area Ratio (FAR) limits are fixed by the local authority this approach may be used for working out the population density. The FSI or FAR is the ratio of total floor area (of all the floors) to the plot area.

The densities of population on this concept may be worked out as in the following example for an area of one hectare (ha)

Roads	20%
Gardens	15%
Schools (including playgrounds)	5%
Markets	2%
Hospital and Dispensary	2%
Total	44%

Area available for Residential Development = 100 - 44 = 56% or 0.56

Actual total floor area = Area for residential development x FSI

Assuming an FSI of 0.5 and floor area of 9 m^2/person

Number of persons or density per hectare $= \dfrac{0.56 \times 10{,}000 \times 0.5}{9} = 311$

3.4 TRIBUTARY AREA

The natural topography, layout of buildings, political boundaries, economic factors etc., determine the tributary area. For larger drainage areas, though it is desirable that the sewer capacities be designed for the total tributary area, sometimes, political boundaries and legal restrictions prevent the sewers to be constructed beyond the limits of the local authority. However, in designing sewers for larger areas, there is usually an economic advantage in providing adequate capacity initially for a certain period of time and adding additional sewers, when the pattern of growth becomes established. The need to finance projects within the available resources necessitates the design to be restricted to political boundaries. The tributary area for any section under consideration has to be marked on a key plan and the area can be measured from the map.

3.5 PER CAPITA SEWAGE FLOW

The entire spent water of a community should normally contribute to the total flow in a sanitary sewer. However, the observed dry weather flow quantities usually are slightly less than the per capita water consumption, since some water is lost in evaporation, seepage into ground, leakage etc. In arid regions, mean sewage flows may be as little as 40% of water consumption and in well developed areas; flows may be as high as 90%. However, the conventional sewers shall be designed for a minimum sewage flow of 100 litres per capita per day or higher as the case may be. Non-conventional sewers shall be designed as the case may be.

For some areas, it is safe to assume that the future density of population for design as equal to the saturation density. It is desirable that sewers serving a small area be designed accordingly on saturation density.

For new communities, design flows can be calculated based on the design population and projected water consumption for domestic use, commercial use and industrial activity. In case a master plan containing land use pattern and zoning regulation is available, the anticipated population can be based on the ultimate densities as in Table 3.1.

The flow in sewers varies from hour to hour and seasonally. However, for the purpose of hydraulic design estimated peak flows are adopted. The peak factor or the ratio of maximum to average flows depends upon contributory population as given in Table 3.2.

Table 3.2 Peak factor for Contributory Population

Contributory Population	Peak Factor
up to 20,000	3.00
Above 20,001 to 50,000	2.50
Above 50,001 to 7,50,000	2.25
above 7,50,001	2.00

Source: CPHEEO, 1993

The peak factor also depends upon the density of population, topography of the site, hours of water supply and hence, individual cases may be further analyzed if required. The minimum flow may vary from 1/3 to 1/2 of average flow.

3.6 INFILTRATION

Estimate of flow in sanitary sewers may include certain flows due to infiltration of groundwater through joints. Since sewers are designed for peak discharges, allowances for groundwater infiltration for the worst condition in the area should be made as in Table 3.3

Table 3.3 Ground water infiltration

	Minimum	Maximum
Litres/ha/day	5,000	50,000
Litres/km/day	500	5,000
Litres/day/manhole	250	500

Source: CPHEEO, 1993

Once the flow is estimated as per Table 3.3, the design infiltration value shall be limited to a maximum of 10% of the design value of sewage flow.

Care shall be taken that in high ground water locations and coastal locations, the sewer pipes shall not be stoneware or vitrified clay pipes and instead shall be cast iron / ductile iron pipes or other non-metallic pipes with safeguards against floatation as discussed later in the section on laying of sewers.

3.7 SEWAGE FROM COMMERCIAL INSTITUTIONS

The industries and commercial buildings often use water other than the municipal supply and may discharge their liquid wastes into the sanitary sewers. Estimates of such flows have to be made separately as in Table 3.4 (overleaf) for their potable water supply.

3.8 INDUSTRIAL EFFLUENTS TO BE DISCOURAGED

The mixing of industrial effluents through discharge into public sewers is undesirable due to the possible detrimental effects of such effluent on the operation of biological sewage treatment process. This aspect has been well recognized in recent times and industrial areas having polluting industries are generally located such as to avoid mixing with sewage.

However, in cities that have undergone unregulated growth in the past, polluting industries may exist in pockets of mixed land use. In such cases, those industries are required to implement zero liquid discharge (ZLD) by reusing the effluents after appropriate treatment in house.

Of all the industries, this shall strictly apply to the automobile service stations and machine shops from where the spent metal plating baths and oil & grease shall be prevented from entering the sewers.

Table 3.4 Institutional needs for potable water

No.	Institutions	Water Supply (litres)
1	Hospital including laundry and beds exceeding 100	450 per bed
2	Hospital including laundry and beds not exceeding 100	340 per bed
3	Lodging houses / hotels	180 per bed
4	Hostels	135 lpcd
5	Nurses homes and medical quarters	135 lpcd
6	Boarding schools/colleges	135 lpcd
7	Restaurants	70 per seat
8	Airports and Seaports, duty staff	70 lpcd
9	Airports and Seaports, alighting and boarding persons	15 lpcd
10	Train and Bus stations, duty staff	70 lpcd
11	Train and Bus stations, alighting and boarding persons	15 lpcd
12	Day schools/colleges	45 lpcd
13	Offices	45 lpcd
14	Factories, duty staff	45 lpcd
15	Cinema, concert halls and theatres	15 lpcd

3.9 STORM RUNOFF

The sanitary sewers are not expected to receive storm water. Strict inspection, vigilance, and proper design and construction of sewers and manholes should eliminate this flow or bring it down to a very insignificant quantity.

However, in small habitations where rainfall is almost a continuous affair, it may be necessary to include storm water in the design of sewers as under.

3.9.1 Estimation of Storm Runoff

The storm runoff is that portion of the precipitation, which drains over the ground. Estimation of such runoff reaching the storm sewers therefore is dependent on the intensity, duration of precipitation, characteristics of the tributary area, and the time required for such flow to reach the sewer.

The design of storm water sewers begins with an estimate of the rate and volume of surface runoff. When rain falls on a given catchment, a portion of the precipitation is intercepted by the vegetation cover that mostly evaporates, a portion hits the soil and some of it percolates down below and the rest flows over the ground. The higher the intensity of rain, the higher will be the peak runoff.

The characteristics of the drainage area such as imperviousness, topography including depressions, water pockets, shape of the drainage basin and duration of the precipitation determine the fraction of the total precipitation, which will reach the sewer. This fraction is known as the coefficient of runoff.

The time-period after which the entire area begins contributing to the total runoff, at a given monitoring point, is known as the time of concentration. It is also defined as the time it takes for a drop of water to flow from the most distant point to the outlet of the basin. The duration of rainfall that is equal to the time of concentration is known as the critical rainfall duration. The rational formula for the relationship between peak runoff and the rainfall is given below.

$$Q = 10\,C\,i\,A \tag{3.1}$$

where,

Q : Runoff in m^3/hr

C : Dimensionless runoff coefficient

i : Intensity of rainfall in mm/hr

A : Area of drainage district in hectares

The storm water flow for this purpose may be determined by using the rational method, hydrograph method, rainfall-runoff correlation studies, digital computer models, inlet method or empirical formulae. The empirical formulae that are available for estimating the storm water runoff can be used only when comparable conditions to those for which the equations were derived initially exist.

A rational approach, therefore, demands a study of the existing precipitation data of the area concerned to permit a suitable forecast. Storm sewers are not designed for the peak flow of rare occurrence such as once in 10 years or more, but it is necessary to provide sufficient capacity to avoid too frequent flooding of the drainage area. There may be some flooding when the precipitation exceeds the design value, which has to be permitted. The frequency of such permissible flooding may vary from place to place, depending on the importance of the area. Though such flooding causes inconvenience, it may have to be accepted occasionally, considering the economy effected in the sizes of the drains and the costs.

The maximum runoff, which has to be carried in a sewer section should be computed for a condition when the entire basin draining at that point becomes contributory to the flow and the time needed for this is known as the time of concentration (with reference to the concerned section). Thus, for estimating the flow to be carried in the storm sewer, the intensity of rainfall which lasts for the period of time of concentration is the one to be considered contributing to the flow of storm water in the sewer. Of the different methods, the rational method is more commonly used as herein.

It may be reiterated that Q represents only the maximum discharge caused by a particular storm. The portion of rainfall, which finds its way to the sewer, is dependent on the imperviousness and the shape of the drainage area apart from the duration of storm.

The percent imperviousness of the drainage area can be obtained from the records of a particular district. In the absence of such data, Table 3.5 (overleaf) may serve as a guide.

Table 3.5 Percentage of Imperviousness of Areas

S. No.	Type of Area	Percentage of Imperviousness
1.	Commercial and Industrial Area	70-90
2.	Residential Area - High Density - Low Density	 61-75 35-60
3.	Parks and undeveloped areas	10-20

Source: CPHEEO, 1993

When several different surface types or land use comprise the drainage area, a composite or weighted average value of the imperviousness runoff coefficient can be computed, such as:

$$I = [(A_1 I_1) + (A_2 I_2) + \ldots\ldots + (A_n I_n)] / [(A_1 + A_2 + \ldots\ldots + A_n)] \quad (3.2)$$

where,

I :	Weighted average imperviousness of the total drainage basin
A_1, A_2, A_n :	Sub drainage areas
I_1, I_2, I_n :	Imperviousness of the respective sub-areas.

The weighted average runoff coefficients for rectangular areas, of length four times the width as well as for sector shaped areas with varying percentages of impervious surface for different time of concentration are given in Table 3.6 (overleaf).

Although these are applicable to particular shape areas, they also apply in a general way to the areas, which are usually encountered in practice. Errors due to difference in shape of drainage are within the limits of accuracy of the rational method and of the assumptions on which it is based.

3.9.2 Rational Method

3.9.2.1 Runoff-Rainfall Intensity Relationship

The entire precipitation over the drainage district does not reach the sewer. The characteristics of the drainage district, such as, imperviousness, topography including depressions and water pockets, shape of the drainage basin and duration of the precipitation determine the fraction of the total precipitation, which will reach the sewer.

This fraction known as the coefficient of runoff needs to be determined for each drainage district. The runoff reaching the sewer is given by Equation (3.1).

3.9.2.2 Storm Frequency

The frequency of storm for which the sewers are to be designed depends on the importance of the area to be drained. Commercial and industrial areas have to be subjected to less frequent flooding. The suggested frequency of flooding in the different areas is as follows -:

Table 3.6 Runoff Coefficients for Times of Concentration

Duration, t, minutes	10	20	30	45	60	75	90	100	120	135	150	180
Weighted Average Coefficient												
1. Sector concentrating in stated time												
a. Impervious	0.525	0.588	0.642	0.700	0.740	0.771	0.795	0.813	0.828	0.840	0.850	0.865
b. 60% Impervious	0.365	0.427	0.477	0.531	0.569	0.598	0.622	0.641	0.656	0.670	0.682	0.701
c. 40% Impervious	0.285	0.346	0.395	0.446	0.482	0.512	0.535	0.554	0.571	0.585	0.597	0.618
d. Pervious	0.125	0.185	0.230	0.277	0.312	0.330	0.362	0.382	0.399	0.414	0.429	0.454
2. Rectangle (length = 4 × width) concentrating in stated time												
a. Impervious	0.550	0.648	0.711	0.768	0.808	0.837	0.856	0.869	0.879	0.887	0.892	0.903
b. 50% Impervious	0.350	0.442	0.499	0.551	0.590	0.618	0.639	0.657	0.671	0.683	0.694	0.713
c. 30% Impervious	0.269	0.360	0.414	0.464	0.502	0.530	0.552	0.572	0.588	0.601	0.614	0.636
d. Pervious	0.149	0.236	0.287	0.334	0.371	0.398	0.422	0.445	0.463	0.479	0.495	0.522

Source: CPHEEO, 1993

a) Residential areas

  i) Peripheral areas — twice a year

  ii) Central and comparatively high priced areas — once a year

b) Commercial and high priced areas — once in 2 years

3.9.2.3 Intensity of Precipitation

The intensity of rainfall decreases with duration. Analysis of the observed data on intensity and duration of rainfall of past records over a period of years in the area is necessary to arrive at a fair estimate of intensity-duration for given frequencies. The longer the record available, the more dependable is the forecast. In Indian conditions, intensity of rainfall adopted in design is usually in the range of 12 mm/hr to 20 mm/hr or based on the actual record.

Table 3.7 gives the analysis of the frequency of storms of stated intensities and durations during 26 years for which rainfall data were available for a given town.

Table 3-7 Duration vs. intensity of storms

Duration in Minutes	Intensity mm/hr	30	35	40	45	50	60	75	100	125
		No. of storms of stated intensity or more for a period of 26 years								
5						100	40	18	10	2
10				90	72	41	25	10	5	1
15			82	75	45	20	12	5	1	
20		83	62	51	31	10	9	4	2	
30		73	40	22	10	8	4	2		
40		34	16	8	4	2	1			
50		14	8	4	3	1				
60		8	4	2	1					
90		4	2							

Source: CPHEEO, 1993

The stepped line indicates the location of the storm occurring once in 2 years, i.e., 13 times in 26 years. The time-intensity values for this frequency are obtained by interpolation from Table 3.8.

Table 3.8 Time intensity values of storms

i (mm/hr)	t (min)	i (mm/hr)	t (min)
30	51.67	50	18.50
35	43.75	60	14.62
40	36.48	75	8.12
45	28.57		

Source: CPHEEO, 1993

The relationship may be expressed by a suitable mathematical formula, several forms of which are available. The following two equations are commonly used:

i) $$i = \frac{a}{(t^n)} \qquad (3.3)$$

ii) $$i = \frac{a}{t + b} \qquad (3.4)$$

where,

i :	Intensity of rainfall (mm/hr)
t :	Duration of storm (minutes)
a, b and n :	Constants

The available data on i and t are plotted and the values of the intensity (i) can then be determined for any given time of concentration, (tc).

3.9.2.4 Time of Concentration

It is the time required for the rain-water to flow over the ground surface from the extreme point of the drainage basin and reach the point under consideration. It is equal to inlet time (t) plus the time of flow in the sewer (tf).

The inlet time is dependent on the distance of the farthest point in the drainage basin to the inlet manhole, the shape, characteristics and topography of the basin and may generally vary from 5 to 30 minutes. In highly developed sections, the inlet time may be as low as 3 minutes. The time of flow is determined by the length of the sewer and the velocity of flow in the sewer. It is to be computed for each length of sewer to be designed.

a) Tributary Area

For each length of storm sewer, the drainage area should be indicated clearly on the map and measured. The boundaries of each tributary are dependent on topography, land use, nature of development and shape of the drainage basins. The incremental area may be indicated separately on the compilation sheet and the total area computed.

b) Duration of Storm

Continuously long, light rain saturates the soil and produces higher coefficient than that due to, heavy but intermittent, rain in the same area because of the lesser saturation in the latter case. The runoff from an area is significantly influenced by the saturation of the surface to nearest the point of concentration, rather than the flow from the distant area. The runoff coefficient of a larger area has to be adjusted by dividing the area into zones of concentration and by suitably decreasing the coefficient with the distance of the zones.

A typical example of the computation of storm runoff is given in Appendix A.3.1.

3.10 MEASUREMENT OF FLOWS IN EXISTING DRAINS/SEWERS

Quite often, the measurement of flows in existing drains or sewers will provide valuable data for a more realistic assessment of the design flows. In general, non-sewered areas will most certainly be having a set of drains where the generated sewage will be flowing out. The assessment of the flows in drains can be done by a variety of methods right from the rudimentary crude method to the most sophisticated dye tracer method. The choice of methods presented hereunder is considered to be appropriate to the conditions in the country particularly, away from metropolitan centres.

a) The Float Method

At the very outset, a non-intrusive method is called for. This can be done by finding out the time taken for a float like an empty match-box or a plastic box to travel for about 3 m in a straight reach and measuring the width and depth of flow in the drain. If we assume the respective values as 20 seconds, the width as 0.9 m and depth of flow as 0.6 m, the flow can be assessed as 0.8 × 0.9 × 0.6 × (3/20) × 1000 = 65 lps. The factor of 0.8 is the average velocity in such drains for the depth of flow.

b) The V notch method

This requires the insertion of a V notch plate in the drain at a location where the downstream discharge can be a free fall. These plates can be cut out from stainless steel (SS) or Teflon sheets of nominal thickness of about 2 mm and inserted tightly into the drain and the gaps can be closed by a mixture of clay and cement in equal proportion mixed to a thick consistency and smeared on the downstream side. The V notch is best chosen such that the angle subtended is 90 degrees. The clearances to be ensured are shown in Figure 3.1

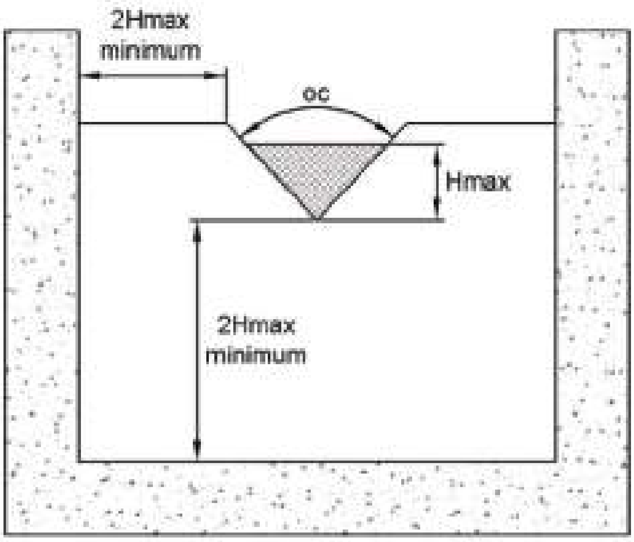


Figure 3.1 Typical mounting of a V Notch in a drain

The depth of flow is measured over the lower tip of the V bottom and the discharge is

Q = 1.42 × tan of angle of V notch × H power 2.5 (3.5)

As the angle is 90 degrees, the tangent is equal to 1 and hence, the equation simplifies to

$$Q=1.42\times H \text{ power } 2.5 \qquad (3.6)$$

Where Q is cum/sec and H is in m

c) The rectangular weir method

This can be used if there is already an existing levelled overflow weir like the overflow culverts in irrigation canals. In smaller drains and in places where workmanship of V notch cuts are difficult, these can be used easily by cutting a mild steel or wood sheet as shown in Figure 3.2.

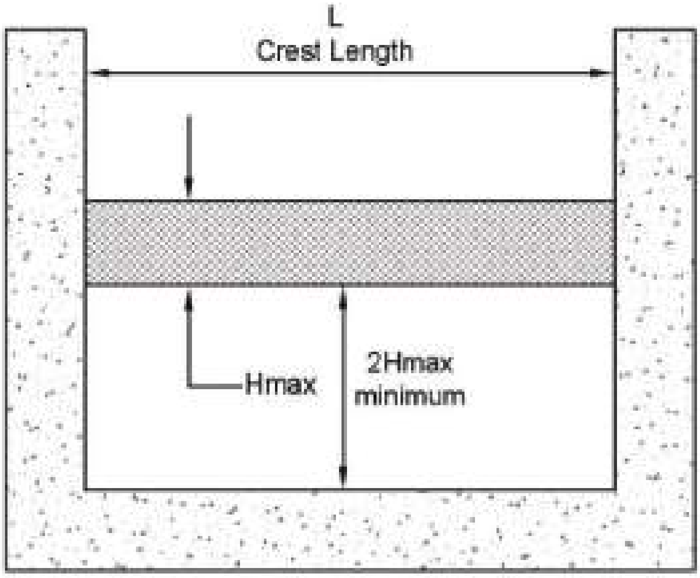


Figure 3.2 Typical mounting of a rectangular weir in a drain

The depth of flow is measured over the overflow edge of the notch and the discharge is

$$Q = 1.85 \times L \times H \text{ power } 1.5 \qquad (3.7)$$

Where,

Q is cum/sec,
H is in m,
L is the length of weir

d) The rectangular weir with end contractions method

These are similar to the rectangular weir except that the length of the weir is smaller than the width of the drain as in Figure 3-3 overleaf.

The depth of flow is measured over the overflow edge of the notch and the discharge is

$$Q = 1.85 \times (L - 0.2H) \times H \text{ power } 1.5 \qquad (3.8)$$

Where

Q is cum/sec,
H is in m,
L is the length of weir

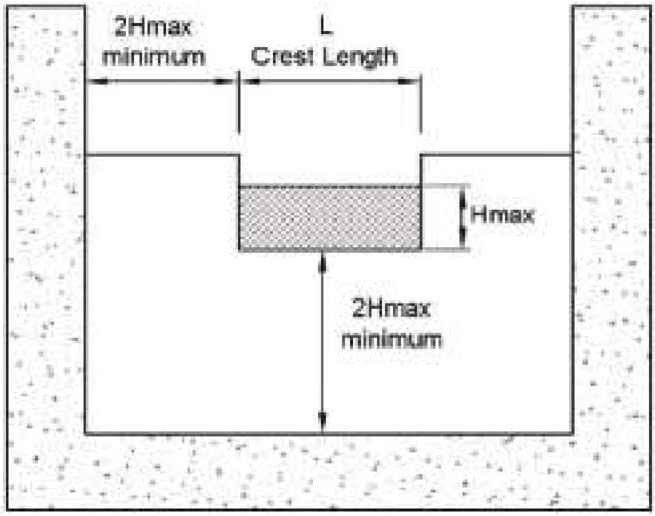


Figure 3.3 Typical mounting of a rectangular weir with end constrictions in a drain

e) The Palmer-Bowlus Flume

This can be used in case of both the drains and pipes flowing under gravity. Its major advantages are (i) less energy loss; (ii) minimal restriction to flow and (iii) Easy installation in existing conduits. It is a readymade piece for various widths and diameters. The placement in a drain will be as in Figure 3.4 and that in a sewer pipe will be as in Figure 3.5 overleaf.

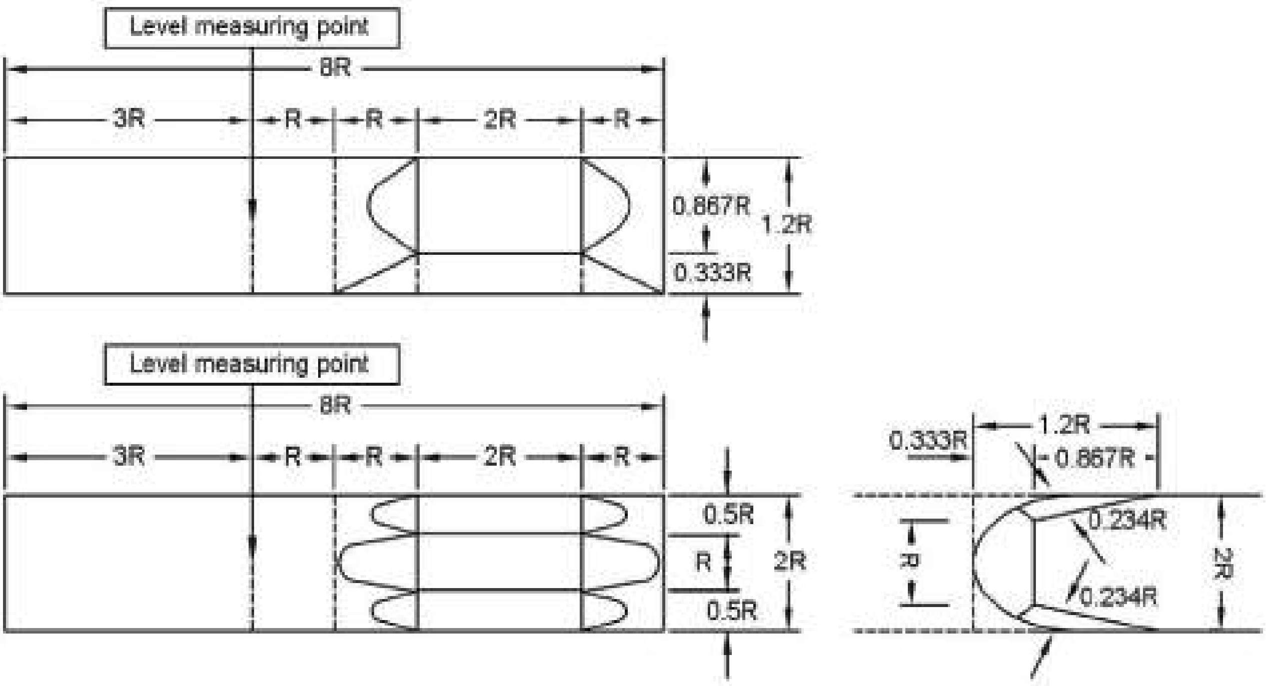


Figure 3.4 Palmer-Bowlus flume installation in drains

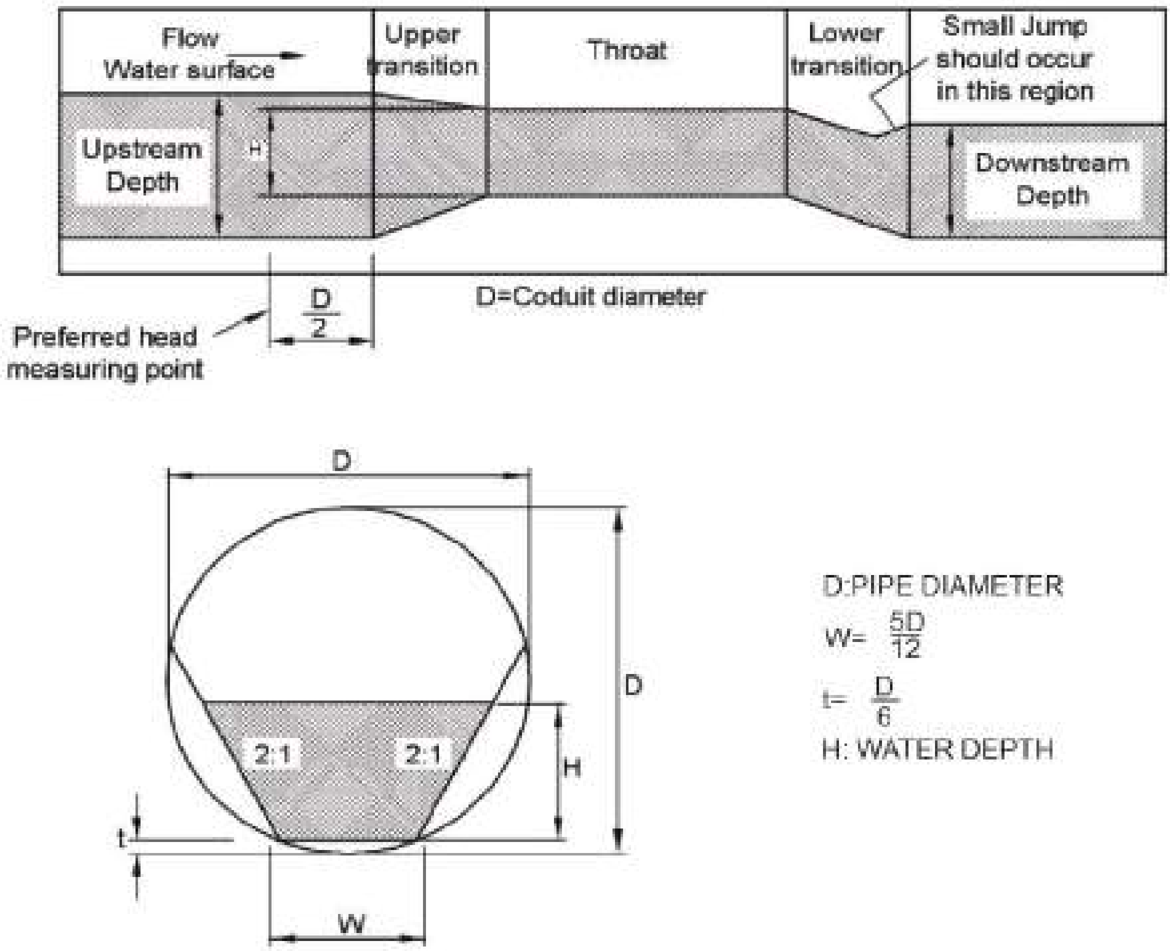


Figure 3.5 Palmer-Bowlus flume installation in circular sewer

This has the specific advantage of its ability to be placed in a manhole to measure the sewage flow in the gravity sewer as long as the flow is not exceeding the diameter of the sewer. Typical installation details are seen in Figure 3.6 overleaf.

The depth of flow needs to be measured in only one location and thus it is a lot easier. In addition, it can be easily removed after measurement. The only disadvantage is it cannot be used when the depth of flow exceeds the diameter of the sewer and to this extent, it may have limitations in the surcharged condition of sewers in historical cities. This also has the advantage of facilitating a flow measurement in large diameter sewers, which flow under gravity and the flume itself is much simpler as in Figure 3.7.

The chart for getting at the flow once the depth is measured is obtained by relating to a standard curve supplied by the plume manufacturer depending on the shape of the plume. This is also available as software linked to a personal computer.

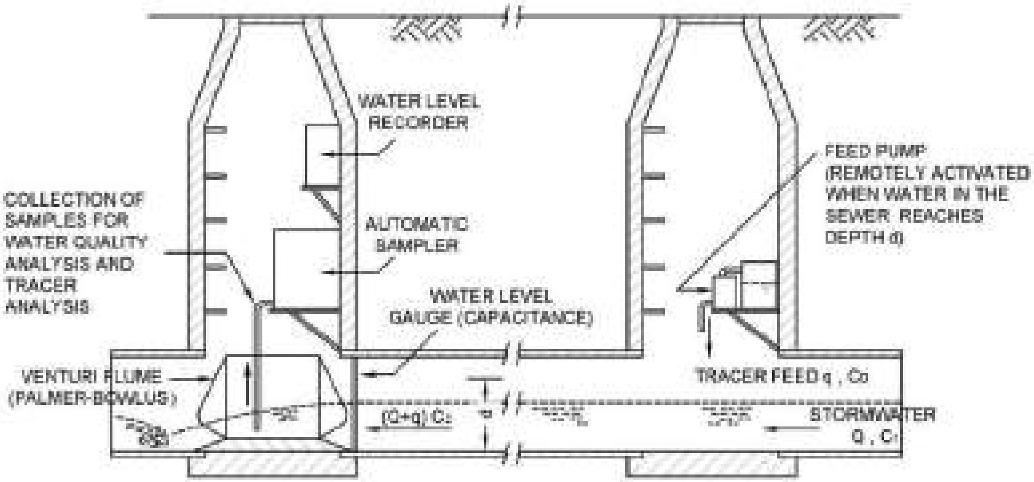
The combination of the Palmer-Bowlus and Tracer dye techniques have been reported as early as 1974 as illustrated in Figure 3.7. It is a system worth inducting in large trunk sewers near the outfalls to have an integrated measurement of the flows and key quality parameters or at least for the flow details and variation patterns.

Top Left and Right- The installation in manholes by inserting the pipe ends into the sewer and measuring the depth of flow by ultrasonic sensor to integrate to a computer as needed.

Bottom left- The Flume, originally invented by Palmer & Bowlus for the Los Angeles County Sanitation District and in use for over three decades, is made by many manufacturers.

Bottom Right- The installation, in a large circular sewer by merely placing the readymade flume at the invert and measurement of the depth, which can be done by ultrasonic sensor.

Figure 3.6 Configurations and use of Palmer-Bowlus flume



Source: J. Marsalek, 1974

Figure 3.7 Instrumentation for flow measurement and sampling in urban large conduits

f) The Venturi Pipe or the Dall Tube

While dealing with old pumping mains, there is a chance of detecting a venturi pipe fitting in the pipeline, as was the standard practice in those years. The flow through it is a function of the difference in head of the fluid at the mouth and the throat and the formula for a given venturi metre is very simple as

$$Q = K \times (a1 \text{ x } a2) \text{ (factor)} \tag{3.9}$$

$$\text{factor} = \text{SQRT } (2gh/(a1^2 - a2^2)) \tag{3.10}$$

Where
K = 0.95 to 0.98
a1 = area in sqm at mouth
a2 = area in sqm at throat
h = h1-h2
h1 = piezometric water level in m at mouth
h2 = piezometric water level in m at throat

It is thus clear that once the difference in head is measured between sewage pressure head at mouth and at throat, the square root of the same is directly proportional to the flow. It is possible to connect a differential Mercury manometer to the sampling ports in the metre and open the quarter turn-cock when flow needs to be measured and to note the reading. A simple wall chart relating the difference to the flow will be more than needed. Of course, instrumentation is possible by connecting the two pressures to a differential pressure transmitter and taking its output to a square root extractor and then to a multiplier for the constant for the metre and thereby get a continuous reading of the flow without any interventional systems.

Suffice to say that so far as estimation of flows for design of sewer systems or augmentation of sewer systems are concerned, where an existing pumping station with a venturi meter in the delivery main is available, a simple mercury manometer U tube, connected to the ports of the venturi meter may help in ascertaining the variation of the flow pattern and arrive at peak flow factors etc. more realistically.

A Dall tube is nothing but a venturi pipe-fitting of a reduced length and as otherwise all other properties of flow measurements are the same.

In fact, if possible this can be inserted into an existing pumping main for the evaluation of the above flow patterns.

PART - 2 TYPES AND HYDRAULICS OF SEWERS

3.11 TYPES OF COLLECTION SYSTEM

These are separate sewers, combined sewers, pressurized sewers and vacuum sewers.

3.11.1 Separate Sewers

These sewers receive domestic sewage and such industrial wastes pre-treated to the discharge standards as per the Environment Protection Act 1986. The consent to discharge into sewers are given by the local pollution control administration.

3.11.2 Combined Sewers

These sewers receive storm water in addition and have some advantages in locations of intermittent rainfall almost throughout the year and with a terrain permitting gravitated collection and obviously being confined to a very small region as a whole. As otherwise, in regions of seasonal rainfall like in monsoons, the combined system will have serious problems in achieving self cleansing velocities during dry seasons and necessitating complicated egg shaped sewers etc. to sustain velocities at such times, plus the treatment plant to be designed to manage strong sewage in dry season and dilute sewage in monsoon season as also the hydraulics. These sewers are also ideally suited for resorts and private development.

3.11.3 Pressurized Sewers

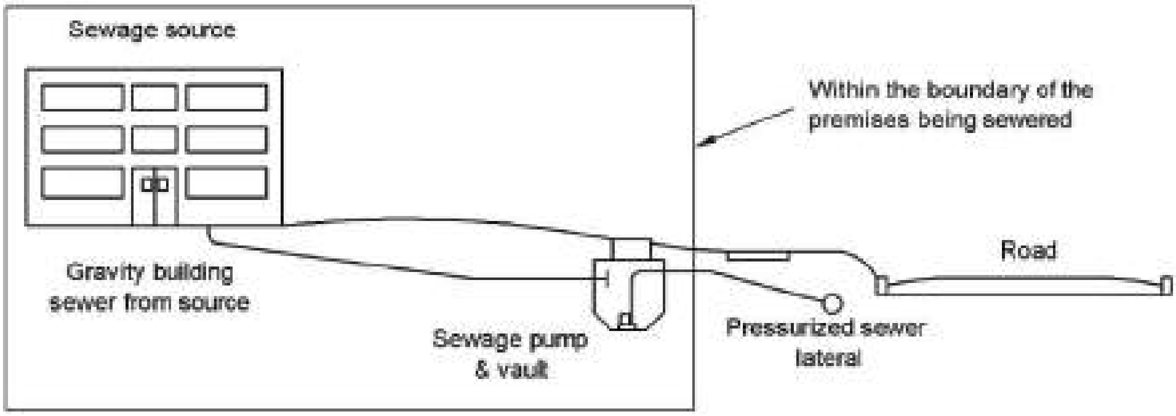
Pressurized sewers are for collecting sewage from multiple sources to deliver to an existing collection sewer, and/or to the STP and are not dependent on gravity and thus topography is not a challenge. Typically, sewage from establishments in the vicinity is collected in a basin fitted with submersible pump to lift and inject the sewage to a sewer on the shoulder of the roadway, thus sparing the riding surface from the infamous digging for initial repairs and often for repairs.

The principle advantages are the ability to sewer areas with undulating terrain, rocky soil conditions and high groundwater tables as pressurized sewers can be laid close to the ground and anchored well besides there cannot be infiltration, and exfiltration is quickly detected and set right and essentially smaller diameter pipes and, above all, obviating the cumbersome deep manholes as also road crossings by CI or DI pipes with trenchless technology laid inside a casing pipe and installation without disrupting traffic, opening trenches across paved roadways, or moving existing utilities etc.

An important issue is for each plot to have a grinder pump set and each commercial plot to have its own grease interceptors to remove excessive fats, oils & grease before the grinder pump. Obviously, this system is not suitable for continuous building area.

A disadvantage is the need to ensure unfailing power supply to the grinder pump and hence this is perhaps limited to high profile condominiums and not the public sewer systems in India.

A typical profile is shown in Figure 3.8 overleaf.

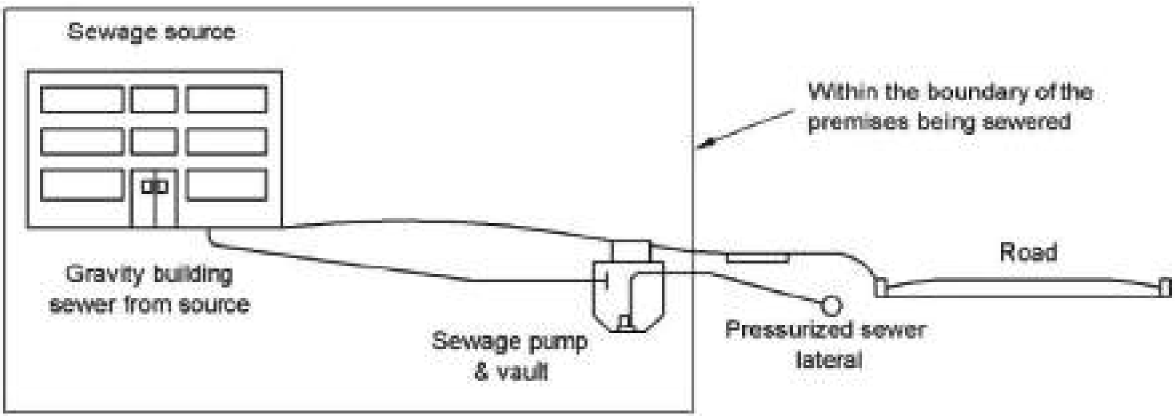


Source: WERF, Fact sheet 2

Figure 3.8 Profile of Pressurized Sewer system

3.11.4 Vacuum Sewer System

The vacuum sewer collects sewage from multiple sources and conveys it to the STP. As the name suggests, a vacuum is maintained in the collection system and when a house sewer is opened to atmospheric pressure, sewage and air are pulled into the sewer, whereby the air forms a “plug” in the line, and air pressure pushes the sewage toward the vacuum station. This differential pressure comes from a central vacuum station. These sewers can take advantage of available slope in the terrain, but have a limited capacity to pull water uphill may be to some 9 m. Each valve pit is fitted with a pneumatic pressure-controlled vacuum valve, which automatically opens after a predetermined volume of sewage has entered the sump. The difference in pressure between the valve pit (at atmospheric pressure) and the main vacuum line (under negative pressure) pulls sewage and air through the service line. The amount of air that enters with the sewage is controlled by the length of time that the valve remains open. When the vacuum valves closes, atmospheric pressure is restored inside the valve pit. Overall, the lines are installed in a saw-tooth or vertical zig-zag configuration so that the vacuum created at the central station is maintained throughout the network. A disadvantage is the need to ensure unfailing power supply to the vacuum pump and hence this is perhaps limited to high profile condominiums and not the public sewer systems in India. A typical profile is shown in Figure 3.9.



WERF, Fact sheet 4

Figure 3.9 Profile of Vacuum sewer system

3.12 MATERIALS, SHAPES AND SIZES OF SEWERS

3.12.1 Introduction

Factors influencing the selection of materials for sewers are flow characteristics, availability in the sizes required including fittings and ease of handling and installation, water tightness and simplicity of assembly, physical strength, resistance to acids, alkalies, gases, solvents, etc., resistance to scour, durability and cost including handling and installation. No single material will meet all the conditions that may be encountered in sewer design. Selection should be made for the particular application and different materials may be selected for parts of a single project. The determination of the suitability in all respects of the pipes and specials for any work is a matter of decision by the engineer concerned on the basis of requirements for the scheme and guided by Appendix A.3-10 on relative limitations on use of pipe materials in specific locations.

3.12.2 Brick

Brickwork is used for construction of sewers, particularly in larger diameters. Many old brick sewers are still in use and the failures are mainly due to the disintegration of the bricks or the mortar joints. Because of the comparatively higher cost, larger space requirement, slower progress of work and other factors, brick is now used for sewer construction only in special cases. The advantage of brick sewers is that these could be constructed to any required shape and size. Brick sewers shall have cement concrete or stone for invert and 12.5 mm thick cement plaster with neat finish for the remaining surface. To prevent ground water infiltration, it is desirable to plaster the outside surface. Inside plaster can be with mortar using high alumina cement conforming to IS 6452 or polyurea coating and the outer surface shall be plastered with mortar using sulphate resistant cement.

3.12.3 Concrete

The advantages of concrete pipes are the relative ease with which the required strength may be provided, feasibility of adopting a wide range of pipe sizes and the rapidity with which the trench may be backfilled. However, these pipes are subject to crown corrosion by sulphide gas, mid depth water line corrosion by sulphate and outside deterioration by sulphate from soil water. These shall be manufactured with sulphate resistant cement and with high alumina coating on the inside at the manufacturers works itself. Protective measures as outlined in corrosion protection in sewers shall be provided where excessive corrosion is likely to occur.

3.12.3.1 Precast concrete

Plain cement concrete pipes are used in sewer systems on a limited scale only and generally, reinforced concrete pipes are used. Non-pressure pipes are used for gravity flow and pressure pipes are used for force mains, submerged outfalls, inverted siphons and for gravity sewers where absolute water-tight joints are required. Non-pressure pipes used for construction of sewers and culverts shall confirm to the IS 458. Certain heavy-duty pipes that are not specified in IS 458 should conform to other approved standards.

3.12.3.2 Cast-in-situ Reinforced Concrete

Cast-in-situ reinforced concrete sewers are constructed where they are more economical, or when non-standard sections are required, or when a special shape is required or when the headroom and working space are limited. The sewer shape shall be of an economic design, easy to construct and maintain and shall have good hydraulic characteristics. Wide flat culvert bottoms shall be provided with “Vee” of at least 15 cm cuvettes in the centre. All formwork for concrete sewers shall be unyielding and tight and shall produce a smooth sewer interior. Collapsible steel forms will produce the desirable sewer surface, and may be used when the sewer size and length justify the expense. It is desirable to specify a minimum clear cover of 50 mm over reinforcement steel and a minimum slump consistent with workability shall be used for obtaining a dense concrete structure free of voids. The distance for cutting concrete shall be kept to a minimum to avoid segregation and the vibrating of concrete done by approved mechanical vibrators. Air entraining cement or plasticizing agents may be used to improve workability and ensure a denser concrete. Concrete shall conform to IS 456.

3.12.4 Stoneware or Vitrified Clay

These pipes are normally available in lengths of 90 cm and the joints need caulking with yarn soaked in cement mortar and packing in the spigot and socket joints, which requires skilled labour. Specifications for the AA class and A class are identical except that in the case of class AA pipes, one hundred percent hydraulic testing has to be carried out at the manufacturing stage, whereas in the case of Class A only five percent of the pipes are tested hydraulically by following IS 651. The resistance of vitrified clay pipes to corrosion from most acids and to erosion due to grit and high velocities gives it an advantage over other pipe materials in handling acid concentrations. A minimum crushing strength of 1,600 kg/m is usually adopted for all sizes manufactured presently. The strength of vitrified clay pipes often necessitates special bedding or concrete cradling to improve field supportive strength.

3.12.5 Asbestos Cement

For sewerage works, asbestos cement pipes are usually used in sizes ranging from 80 mm to 1000 mm in diameter. Standard specifications have been framed by the BIS in IS 6908. Non corrosiveness to most natural soil conditions, freedom from electrolytic corrosion, good flow characteristics, light weight, ease in cutting, drilling, threading and fitting with specials, allowance of greater deflection up to 12 degrees with mechanical joints, ease of handling, tight joints and quick laying and backfilling are to be considered. These pipes cannot however stand high super imposed loads and may be broken easily. They are subject to corrosion by acids, highly septic sewage and by highly acidic or high sulphate soils. Protective measures as outlined in corrosion protection in sewers shall be provided in such cases. While using AC pipes strict enforcement of approved bedding-practices will reduce possibility of flexible failure.

Where grit is present, high velocities such as those encountered on steep grades may cause erosion. It is stated that in a recent process of manufacture titled Maaza, high forming pressures of up to 80 kg / sqcm, leading to very smooth surface and very few air pores are possible. However, the relevant BIS standard or code of practice is awaited.

3.12.6 Cast Iron

Cast Iron pipes and fittings are being manufactured in the country for several years. These pipes are available in diameters from 80 mm to 1050 mm and are covered with protective coatings. Pipes are supplied in 3.66 m and 5.5 m lengths and a variety of joints are available including socket, spigot, and flanged joints. These pipes have been classified as LA, A and B according to their thickness. Class LA pipes have been taken as the basis for evolving the series of pipes. Class A pipes allow 10 % increase in thickness over class LA. Class B pipes allows 20 % increase in thickness over class LA. Cast iron pipes with a variety of jointing methods are used for pressure sewers, sewers above ground surface, submerged outfalls, piping in sewage treatment plants and occasionally on gravity sewers where absolutely water-tight joints are essential or where special considerations require their use. IS 1536 and IS 1537 give the specifications for spun, and vertically cast pipes respectively. The advantage of cast iron pipes are long laying lengths with tight joints, ability when properly designed to withstand relatively high internal pressure and external loads and corrosion resistance in most natural soils. They are however subject to corrosion by acids or highly septic sewage and acidic soils. Whenever it is necessary to deflect pipes from a straight line either in the horizontal or in the vertical plane, the amount of deflection allowed should not normally exceed 2.5 degrees for lead caulked joints. In mechanical joints, the deflection shall be limited to 5 degrees for 80 to 300 mm dia, 4 degrees for 350 to 400 mm diameter and 3 degrees from 400 to 750 mm diameter pipes. Inside coating shall be by Cement mortar and outer coating shall be coal tar both carried out at the manufacturer’s works and conforming to the relevant BIS standards/codes of practice.

3.12.7 Steel

Pressure sewer mains, under water river crossings, bridge crossings, necessary connections for pumping stations, self-supporting spans, railway crossing and penstocks are some of the situations where steel pipes are preferred. Steel pipes can withstand internal pressure, impact load and vibrations much better than CI pipe. They are more ductile and withstand water hammer better. For buried sewers, spirally welded pipes are relatively stronger than horizontally welded sewers. The disadvantage of steel pipe is that it cannot withstand high external load. Further, the main is likely to collapse when it is subjected to negative pressure. Steel pipes are susceptible to various types of corrosion. A thorough soil survey is necessary all along the alignment where steel pipes are proposed. Steel pipes shall be coated inside by high alumina cement mortar or polyurea and outside by epoxy. Steel pipes shall conform to IS 3589. Electrically welded steel pipes of 200 mm to 2,000 mm diameter for gas, water and sewage and laying should conform to IS 5822.

3.12.8 Ductile Iron Pipes

Ductile iron is made by a metallurgical process, which involves the addition of magnesium into molten iron of low sulphur content. The magnesium causes the graphite in the iron to precipitate in the form of microscopic (6.25 micron) spheres rather than flakes found in ordinary cast iron. The spheroidal graphite in iron improves the properties of ductile iron. The ductile iron pipes are normally prepared using the centrifugal cast process. The ductile iron pipes are usually provided with cement mortar lining at the factory by centrifugal process to ensure a uniform thickness throughout its length.

Cement mortar lining is superior to bituminous lining as the former provides a smooth surface and prevents tuberculation by creating a high pH at the pipe wall and ultimately by providing a physical and chemical barrier to the water.

The Indian standard IS 8329 provides specification for the centrifugally cast ductile iron pipes (similar to ISO 2531 and EN 1994). These pipes are available in the range of 80 mm to 1000 mm diameter, in lengths of 5.5 to 6 m. These pipes are manufactured in the country with ISO 9002 accreditation.

The ductile iron pipes have excellent properties of machinability, impact resistance, high wear and tear resistance, high tensile strength and ductility and corrosion resistance. DI pipes, having same composition of CI pipe, will have same expected life as that of CI pipes. They are strong, both inner and outer surfaces are smooth, free from lumps, cracks, blisters and scars. The ductile iron pipes stand up to hydraulic pressure tests as required by service regulations. These pipes are approximately 30 % lighter than conventional cast iron pipes. The ductile iron pipes are lined with cement mortar in the factory by centrifugal process and unlined ductile iron pipes are also available. The ductile iron fittings are manufactured conforming to IS 9523. The joints for ductile iron pipes are suitable for use of rubber gaskets conforming to IS 5383.

3.12.9 Non-Metallic Non-Concrete Synthetic Material Pipes

The main advantage of these pipes is their ability not to be affected by corrosion from sulphides or sulphates but they require precautions as detailed in clause under the sub title “laying of sewers and need to be ascertained and sorted out on specific cases. They require precautions as detailed in clause under the sub title “laying of sewers” and evaluated on case-by-case basis. An additional criterion is the ability of these pipes to withstand the mechanical jet rodding machines as in Figure 3.10 to clear the obstructions in sewers.

The mechanized reverse jet at the nozzle releasing a “jack hammer” action.

Figure 3.10 Jet rodding cum vacuum, suction sewer cleaning machine

The jet in these machines is a “jack hammer” action through a triplex plunger pump releasing the treated sewage backwards as in Figure 3.10 and the hydraulic pressures are in the range of 50 to 60 bar. The jet in the reverse direction of sewer flow acts like an airplane jet and propels the nozzle forward, and thus drills through the choked up blocks and clears the obstructions however tough they are. While the ability of the metallic and concrete sewer pipes to withstand this jet action at that pressure is by now well established in the country mainly due to the rubber ring joints, the ability of the non-metallic synthetic material sewer pipes are to be established hereafter in the country.

Moreover, their track record in other locations in such applications shall be suitably evaluated before adoption. In general, the homogenous wall composition can be relied better than multi-layered adhesive based wall composition.

3.12.9.1 UPVC Pipe

The chief advantages of UPVC pipe are resistance to corrosion, light weight for transportation, toughness, rigidity, economical in laying, jointing, and maintenance and easy to fabricate.

To prevent buoyancy the pipes can be tied to poles driven into the ground. IS 15328 deals with non-pressure unplasticized polyvinylchloride (PVC) for use in underground sewerage system. IS 9271 deals with the unplasticized polyvinyl chloride (UPVC) single wall corrugated pipes for drainage.

3.12.9.2 High Density Polyethylene (HDPE) Pipes

The advantages of these pipes offering smooth interior surfaces and offering relatively highest resistance to corrosion are recognized and they are available in solid wall. When laid in straight gradients without humps or depressions, they can easily offer longer life cycle.

Methods of joints are usually fusion welded or flange jointed depending on straight runs or fittings. Standard specifications have been framed by the BIS in IS 14333 for sewerage application.

3.12.9.3 Structured Wall Piping

These pipes can be manufactured in PVC-U, PP and PE as per EN 13476-3 / IS 16098. The walls of these pipes are either double walled or ribbed wall. The BIS for pipes and fittings with PVC-U material having smooth external surface Type A is IS 16098 (Part-1) and for pipes and fittings with PE and PP material having non-smooth external surface Type B is IS 16098 (Part-2). The Type B pipes are generally known as Double Walled Corrugated (DWC) pipes. In India, DWC pipes are produced in sizes 75 mm ID to 1,000 mm ID with a standard length of 6 m for easy transportation and handling and to reduce the number of joints required.

3.12.9.4 Glass Fibre Reinforced Plastic Pipes (GRP)

GRP Pipes are widely used in other countries where corrosion resistant pipes are required at reasonable costs. GRP can be used as a lining material for conventional pipes which are subject to corrosion. Fibre glass can resist external and internal corrosion whether the corrosion mechanism is galvanic or chemical in nature. Standard specifications have been framed by the BIS in IS 14402.

3.12.9.5 Fibre Glass Reinforced Plastic Pipes (FRP)

Fibre-glass reinforced plastic pipe is a matrix or composite of glass fibre, polyester resin and filters. These pipes possess better strength, durability, high tensile strength, low density and are highly corrosion resistant. Fibre-glass pressure pipes are manufactured in diameters up to 2,400 mm and length up to 18 m. These pipes are now being taken up for manufacture in India.

3.12.9.6 Pitch Fibre Pipes

The pitch impregnated fibre pipes are light in weight and have shown their durability in service. The pipes can be easily jointed in any weather condition as internally tapered couplings join the pipes without the use of jointing compound.

They are flexible, resistant to heat, freezing and thawing and earth currents, which cause electrolytic action. They are also unaffected by acids and other chemicals, water softeners, sewer gases, oils and greases and laundry detergents.

They can be cut to required length on the site. Due to its longer length, the cost of jointing, handling and laying is reduced. These are generally recommended for uses such as septic tanks and house connection to sewers, farm drainage, down pipes, storm drains, industrial waste drainage, etc.

These are manufactured in India with 50 to 225 mm nominal diameter and length varying from 1.5 to 3.5 m. These pipes are joined by taper coupling joints or rubber ring joints. The details of the pipes, fittings, etc., are covered in IS 11925.

3.13 SHAPE AND SIZE OF SEWERS

a) In general, circular sewer sections are ideal from load bearing point of view in public roads and as the hydraulic properties are better for varying flows.

b) However, for large flows, the egg-shaped sections are superior for both load transmission and velocity at minimum flows plus ability to flush out sediments in the bottom V portion when peak flow arises. These are normally of RCC, either cast in situ or pre-cast as also brickwork, though brickwork has its challenges of quality assessment and quality control.

c) Box conduits are also possible provided the inner corners are chamfered and the bottom finished as cuvettes instead of flat floor. They are perhaps best suited as a cover for taking higher diameter gravity circular sewers across roads, railway crossings, river crossings, etc. These can be built in situ with brickwork or cast in situ concrete. They can also be made in pre-cast sections duly jointed. In all cases plastering is needed on the inside and soil side and on the top side and the corners shall be filleted.

d) In early stages of new housing plot layouts, it is invariably the septic tank that is provided in the built up plots and either the septage is either sucked out periodically and sometimes surreptitiously emptied at random locations or simply discharged into the road drains or officially discharged into treatment plants or pumping stations. Nevertheless, there are many places where it is merely let into roadside drains or merely on road sides which complicates environmental issues. The twin drain system can be used, which comprises an integrally built twin drain with the drain nearer to the property carrying the septic tank effluent and grey water and the drain on the road-side carrying the storm water. The sewer drains are interconnected to flow out to treatment. A typical system in use in coastal areas of Tamil Nadu in Tsunami affected rehabilitation centres is pictured in Figure 3.11 overleaf.

The twin drain under construction and after commissioning at the Tsunami rehabilitated fisher folk settlement. The house side drain receives septic tank effluent and is with removable cover slabs.

Source: M/s. Kottar Social Service Society, Nagercoil & M/s. Caritas India & M/s. Caritas Germany

Figure 3.11 Twin drain system

3.14 MINIMUM SIZE OF CIRCULAR SEWERS

The minimum diameter may be adopted as 200 mm for cities having present / base year population of over 1 lakh. However, depending on growth potential in certain areas even 150 mm diameter can also be considered. However, in towns having present / base year population of less than 1 lakh, the minimum diameter of 150 mm shall be adopted.

In the case of hilly locations, the minimum diameter of 150 mm shall be adopted. The house sewer connection pipe to public sewer shall be (a) minimum 100 mm or higher based on the number of houses / flats connected and (b) subject to the receiving public sewer being of higher diameter.

3.15 FLOW IN CIRCULAR SEWERS

If the velocity and depth of flow is the same for the length of a conduit, it is termed steady flow and as otherwise, it is non-steady flow. The hydraulic analysis of sewers is simplified by assuming steady flow conditions though the actual flow conditions are different during morning peak flows and varying flows in other parts of the 24 hours.

In the design of sanitary sewers, an attempt shall be made to obtain adequate scouring velocities at the average or at least at the maximum flow at the beginning of the design period. The flow velocity in the sewers shall be such that the suspended materials in sewage are not silted up; i.e., the velocity shall be such as to cause automatic self-cleansing effect. The generation of such a minimum self cleansing velocity in the sewer, at least once a day is important, because if depositions are takes place and is not removed, it will obstruct free flow, causing further deposition and finally leading to the complete blocking of the sewer.

The smooth interior surface of a sewer pipe gets scoured due to continuous abrasion caused by the suspended solids present in sewage. It is, therefore, necessary to limit the maximum velocity in the sewer pipe. This limiting or non-scouring velocity will mainly depend upon the material of the sewer.

Thus the sewers are designed on the assumption that although silting might occur at minimum flow, it would be flushed out during peak flows. Erosion of sewers is caused by sand and other gritty material in the sewer and by excessive velocity.

3.15.1 Minimum Velocity for Preventing Sedimentation

To ensure that deposition of suspended solids does not take place, self-cleansing velocities using Shield's formula is considered in the design of sewers.

$$V = \frac{1}{n}\left(R^{\frac{1}{6}}\sqrt{K_S\left(S_S - 1\right)d_P}\right) \tag{3.11}$$

where, n = Manning's n

R = Hydraulic Mean Radius in m

K_S = Dimensionless constant with a value of about 0.04 to start motion of granular particles and about 0.8 for adequate self cleansing of sewers

S_S = Specific gravity of particle

dp = Particle size in mm

The above formula indicates that velocity required to transport material in sewers is mainly dependent on the particle size and specific gravity and slightly dependent on conduit shape and depth of flow. The specific gravity of grit is usually in the range of 2.4 to 2.65. Gravity sewers shall be designed for the velocities as in Table 3.9.

Table 3.9 Design velocities to be ensured in gravity sewers

No	Criteria	Value
1	Minimum velocity at initial peak flow	0.6 m/s
2	Minimum velocity at ultimate peak flow	0.8 m/s
3	Maximum velocity	3 m/s

Source: WPCF, ASCE, 1982

3.15.2 Minimum Velocity for Preventing Hydrogen Sulphide in Sewers

The velocity shall be not only self cleansing but also be sufficient to keep the submerged surfaces of the sewer free from slimes and prevent the generation of Hydrogen Sulphide gas which can attack the cement concrete sewers. It is useful to define a climatic condition as the combination of the average temperature for the warmest three months of the year and the average 6-hour high flow BOD for the day. Where diurnal BOD curves have not been made, it may be assumed that this BOD is 1.25 times the BOD of flow proportioned 24-hour composite. The effective BOD defined by the equation:

$$(EBOD) = (BOD)_c \times 1.07^{(T_c-20)} \tag{3.12}$$

Where

EBOD = Effective EBOD in mg/l

$(BOD)_c$ = Climatic BOD in mg/l

T_c = Climatic temperature in degrees Celsius

1.07 = Empirical coefficient

The reference for E BOD is from ASCE Manuals and Reports on Engineering Practice No. 60.

3.15.2.1 Potential for Sulphide Build up

Another indicator of the likelihood of sulphide build up in relatively small gravity sewers (not over 600 mm diameter) is given by the formula:

$Z = [EBOD/(S^{0.50} \times Q^{0.33})] \times (P/b)$ (3.13)

Where

Z = Defined function

S = Hydraulic slope

Q = Discharge volume in m3/sec

P = Wetted perimeter in meters

b = Surface width in meters.

The reference for sulphide generation is WPCF, ASCE, 1982.

The sulphide generation based on Z values are given in Table 3.10

Table 3.10 Sulphide generation based on Z values

Z Values	Sulphide Condition
Z < 5,000	Sulphide rarely generated
5,000 ≤ Z ≤ 10,000	Marginal condition for sulphide generation
Z > 10,000	Sulphide generation common

Source: WPCF, ASCE, 1982

3.15.3 Maximum Velocity

Erosion is caused by sand and other gritty material and is compounded by high velocities and hence the maximum velocity shall be limited to 3 m/s. In hilly areas where the slope and flows gets fixed, the velocity also gets automatically fixed. If such velocities exceed 3 m/sec in hilly areas, use of cast iron and ductile iron pipes shall be made with socket and spigot joints and O rings and the sockets facing uphill. The provision of structures like drop manholes can also be made to dissipate the energy.

3.15.4 Manning's Formula for Gravity Flow

$$V = [(1/n)] \times \left[R^{2/3} S^{1/2} \right] \quad (3.14)$$

For circular conduits

$$V = (1/n)\left(3.968 \times 10^{-3}\right) D^{2/3} S^{1/2} \quad (3.15)$$

and

$$Q = (1/n)\left(3.118 \times 10^{-6}\right) D^{2.67} S^{1/2} \quad (3.16)$$

where,

Q : Discharge in l/s

S : Slope of hydraulic gradient

D : Internal diameter of pipe line in mm

R : Hydraulic radius in m

V : Velocity in m/s

n : Manning's coefficient of roughness as in Table 3-11

A chart for Manning's formula is in Appendix A.3.2 A and A 3.2 B for the stated ranges of discharges therein. These can be used to initially verify

(a) the tentative size, and slope of the required sewer for a given flow rate and velocity, or

(b) The tentative flow rate and slope of a chosen sewer size and velocity.

It is not easy to read these values precisely to decimal values from the graph and hence, it is recommended to recheck the values in the MS Excel spreadsheet given in Appendix A.3.3. There are also similar nomograms, etc. but the precision is best obtained in MS Excel.

3.15.5 Design Depth of Flow

The sewers shall not run full as otherwise the pressure will rise above or fall below the atmospheric pressure and condition of open channel flow will cease to exist. Moreover, from consideration of ventilation, sewers should not be designed to run full. In case of circular sewers, the Manning's formula reveals that:

- The velocity at 0.8 depth of flow is 1.14 times the velocity at full depth of flow.
- The discharge at 0.8 depth of flow is 0.98 times the discharge at full depth of flow.

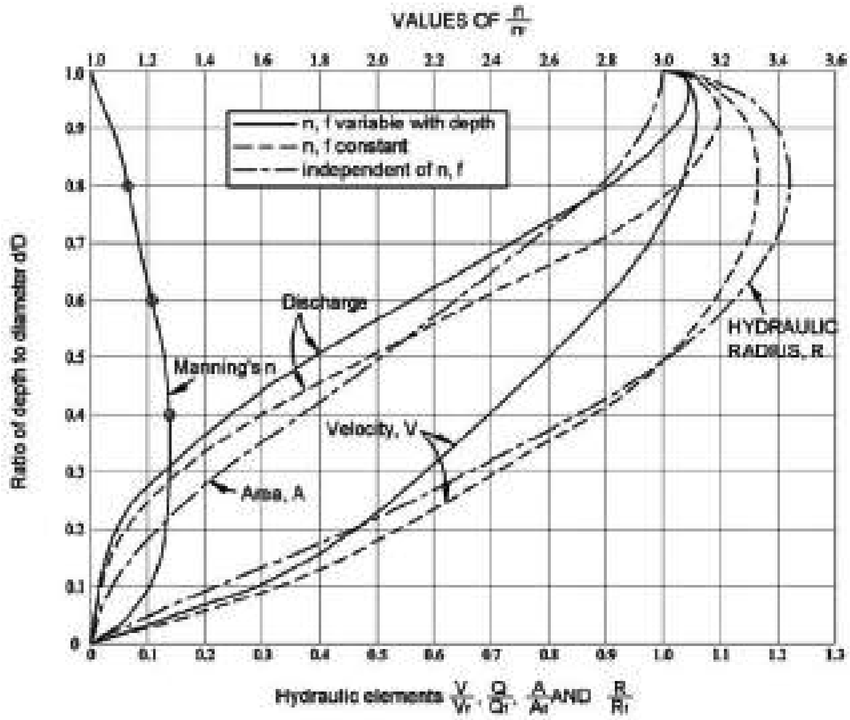
Accordingly, the maximum depth of flow in design shall be limited to 0.8 of the diameter at ultimate peak flow. In order to facilitate the calculations easily, the hydraulic properties at various depths of flow are compiled in Figure 3.12 and Figure 3.13 and Table 3.12.

Table 3.11 Manning's coefficient of roughness n for stated materials

Type of Material	Condition	Manning's n
Salt glazed stone ware pipe	(a) Good	0.012
	(b) Fair	0.015
Cement concrete pipes (With collar joints)	(a) Good	0.013
	(b) Fair	0.015
Spun concrete pipes (RCC & PSC),With S / S Joints, (Design Value)		0.011
Masonry	(a) Neat cement plaster	0.018
	(b) Sand and cement plaster	0.015
	(c) Concrete, steel troweled	0.014
	(d) Concrete, wood troweled	0.015
	(e) Brick in good condition	0.015
	(f) Brick in rough condition	0.017
	(g) Masonry in bad condition	0.020
Stone-work	(a) Smooth, dressed ashlar	0.015
	(b) Rubble set in cement	0.017
	(c) Fine, well packed gravel	0.020
Earth	(a) Regular surface in good condition	0.020
	(b) In ordinary condition	0.025
	(c) With stones and weeds	0.030
	(d) In poor condition	0.035
	(e) Partially obstructed with debris or weeds	0.050
Steel	(a) Welded	0.013
	(b) Riveted	0.017
	(c) Slightly tuberculated	0.020
	(d) With spun cement mortar lining	0.011
Cast Iron / Ductile Iron	(a) Unlined	0.013
	(b) With spun cement mortar lining	0.011
Asbestos cement		0.011
Plastic (smooth)		0.011
FRP		0.01
HDPE/UPVC		0.01

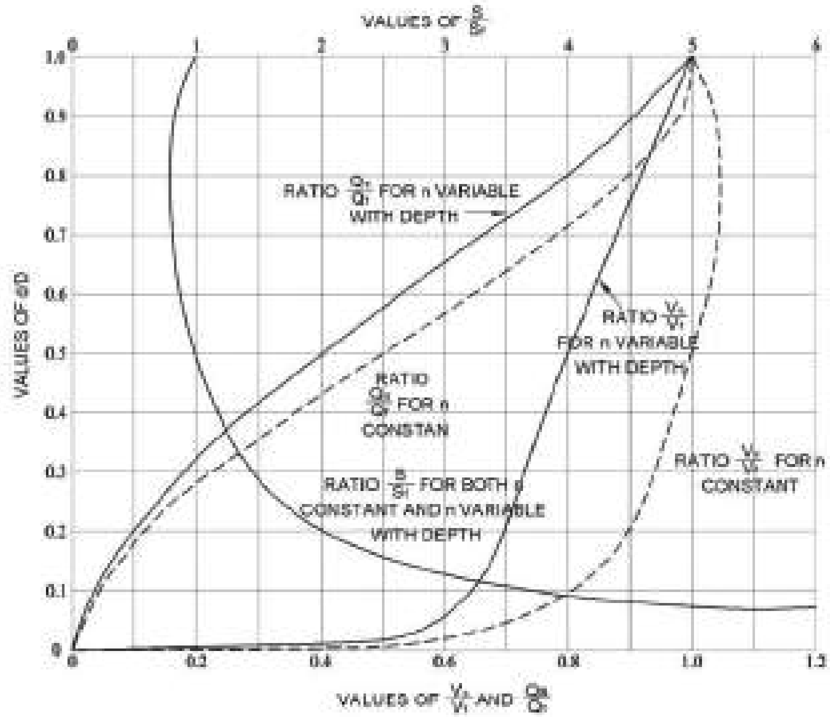
Note: Values of n may be taken as 0.015 for unlined metallic pipes and 0.011 for plastic and other smooth pipes.

Source: CPHEEO, 1999



Source: CPHEEO, 1993

Figure 3.12 Hydraulic – Element graph for circular sewers



Source: CPHEEO, 1993

Figure 3.13 Hydraulic elements of circular sewers that possess equal self-cleansing properties at all depths

Table 3.12 Hydraulic properties of circular sections for Manning's formula

Constant (n)			Variable (n)		
d/D	v/V	q/Q	n_d/n	v/V	q/Q
1.0	1.000	1.000	1.00	1.000	1.000
0.9	1.124	1.066	1.07	1.056	1.020
0.8	1.140	0.968	1.14	1.003	0.890
0.7	1 120	0.838	1.18	0.952	0712
0.6	1.072	0.671	1.21	0.890	0.557
0.5	1.000	0.500	1.24	0.810	0.405
0.4	0.902	0.337	1.27	0.713	0.266
0.3	0 776	0.196	1.28	0.605	0.153
0.2	0.615	0,088	1.27	0,486	0.070
0.1	0.401	0.021	1.22	0.329	0.017

Source: CPHEEO, 1993

Where,

D = Depth of flow (internal dia) d = Actual depth of flow
V = Velocity at full depth v = Velocity at depth 'd'
n = Manning's coefficient at full depth nd = Manning's coefficient at depth 'd'
Q = Discharge at full depth q = Discharge at depth 'd'

For practical purposes, it is not possible to measure the value of n_d and hence only the fixed value of Manning's n shall be used. The method of using the Table 3-12 is illustrated in Appendix A.3.4 A and A 3.4 B for stated ranges of discharges for typical cases in day-to-day situations of design of circular sewer pipes under gravity flow conditions. In as much as the determination of the varying values of n is difficult and has many uncertainties, the formula shall be used with constant values of n only.

3.15.6 Slope of Sewer

The minimum slopes in Table 3.13 shall be applicable:

Table 3.13 Minimum slopes of sanitary sewers

Sewer Size (mm)	Minimum Slope		Sewer Size (mm)	Minimum Slope	
	As percent	As 1 in		As percent	As 1 in
150	0.6	170	375	0.15	670
200	0.40	250	450	0.12	830
250	0.28	360	≥525	0.10	1000
300	0.22	450			

3.16 HYDRAULICS OF SEWERS FLOWING UNDER PRESSURE

3.16.1 Type of Flow

The hydraulic analysis of pumping mains is approached based on turbulent flow conditions to ensure that the suspended matter does not settle during pumping.

3.16.2 Hazen-Williams Formula

$$V = 0.849\, C\, R^{0.63}\, S^{0.54} \tag{3.17}$$

for circular conduits, the expression becomes

$$V = 4.567 \times 10^{-3}\, C\, D^{0.63}\, S^{0.54} \tag{3.18}$$

and

$$Q = 1.292 \times 10^{-5} C\, D^{2.63}\, S^{0.54} \tag{3.19}$$

where,

Q : Discharge in m^3/hr
D : Internal diameter of pipe in mm
V : Velocity in m/s
R : Hydraulic radius in m
S : Slope of hydraulic gradient and
C : Hazen – Williams coefficient as in Table 3.14 overleaf.

A chart for Hazen William's formula is in Appendix A. 3.5 A and A 3.5 B for stated ranges of discharges. This can be used to initially verify -:

(a) The tentative size, and slope of the required sewer for a given flow rate and velocity, or

(b) The tentative flow rate and slope of a chosen sewer size and velocity.

It is not easy to read these values precisely to decimal values from the graph and hence, it is recommended to recheck the values in the MS Excel given in Appendix A.3.6.

3.17 SEWER TRANSITIONS

3.17.1 Connections of Different Sewers

Where sewers of different characteristics are connected, sewer transitions occur. The difference may be in terms of flow, area, shape, grade, alignment and conduit material, with a combination of one or all characteristics. Transitions may be in the normal cases streamlined and gradual and can occur suddenly in limiting cases. Head lost in a transition is a function of velocity head and hence assumes importance in the flat terrain. Deposits also impose significant losses. For design purposes, it is assumed that energy losses and changes in depth, velocity and invert elevation occur at the centre of transition and afterwards these changes are distributed throughout the length of transition.

Table 3.14 Hazen-Williams coefficients

No.	Conduit Material	Recommended values for	
		New Pipes(A)	Design
1	Unlined metallic pipes		
	Cast iron, Ductile iron	130	100
	Mild steel	140	100
2	Centrifugally lined metallic pipes		
	Cast iron, Ductile iron and Mild steel pipes lined with cement mortar or epoxy		
	Up to 1200 mm dia	140	140
	Above 1200 mm dia	145	145
3	Projection method cement mortar lined metallic pipes		
	Cast iron, Ductile iron and Mild steel pipes	130(B)	110(C)
4	Non-metallic pipes		
	RCC spun concrete upto 1200 mm diameter	140	140
	RCC spun concrete above 1200 mm diameter	145	145
	Pre-stressed concrete upto 1200 mm diameter	140	140
	Pre-stressed concrete above 1200 mm diameter	145	145
	Asbestos cement	150	140
	HDPE, UPVC, GRP, FRP	150	140

Note:

A The C values for new pipes included in the above table are for determining the acceptability of surface finish of new pipelines. The user agency may specify that flow test may be conducted for determining the C values of laid pipelines.

B For pipes of diameter 500 mm and above, the range of C values may be from 90 to 125 for pipes less than 500 mm.

C In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher rates upto 130 may be adopted.

Note: Even though the C value can be taken as 145 for Water supply, but for sewage, 140 shall be taken for design purpose.

Source: CPHEEO, 1999

The energy head, piezometric head (depth) and invert as elevation are noted and working from Energy grade line, the required invert drop or rise is determined. However, if the calculations indicate a rise in invert it is ignored since such a rise will create a damming effect leading to deposition of solids.

For open-channel transition in subcritical flow the loss of energy is expressed as:

$$Head\ Loss = K\ (V^2 / 2g) \tag{3.20}$$

Where

$(V^2/2g)$ is change of velocity head before and after transition,

K = 0.1 for contractions and 0.2 for expansions.

In transitions for supercritical flow, additional factors must be considered, since standing waves of considerable magnitude may occur or in long transitions air entrapment may cause backing of flow. Allowance for the head loss that occurs at these transitions has to be made in the design.

Manholes shall be located at all such transitions and a drop shall be provided where the sewer is intercepted at a higher elevation for streamlining the flow, taking care of the head loss and to help in maintenance. The vertical drop may be provided only when the difference between the elevations is more than 60 cm, below which it can be avoided by adjusting the slope in the channel and in the manhole connecting the two inverts. The following invert drops are recommended:

(a)	For sewers less than 400 mm	Half the difference in dia.
(b)	400 mm to 900 mm	2/3 the difference in dia.
(c)	Above 900 mm	4/5 the difference in dia.

Transition from larger to smaller diameters shall not be made. The crowns of sewers are always kept continuous. In no case, the hydraulic flow line in the large sewers shall be higher than the incoming one. To avoid backing up, the crown of the outgoing sewer shall not be higher than the crown of incoming sewer.

3.17.2 Bends

The head loss in bends is expressed by:

$$h_b = k_b\ V^2 / 2g \tag{3.21}$$

Where

k_b is a friction coefficient, which is a function of the ratio of radius of curvature of the bend to the width of conduit, deflection angle, and cross section of flow.

The friction factor for various fittings are given in Table 4-2 in Chapter 4.

3.17.3 Junction

A junction occurs where one or more branch sewers enter a main sewer. The hydraulic design is in effect, the design of two or more transitions, one for each path of flow. Apart from hydraulic considerations, well-rounded junctions are required to prevent deposition. Because of difficulty in theoretically calculating the hydraulic losses at junctions, some general conditions may be checked to ensure the proper design of junctions. If available energy at junctions is small, gently sloping transitions may be used. The angle of entry may be 30 degrees or 45 degrees with reference to axis of main sewer, whenever ratio of branch sewer diameter to main sewer diameter is one half or less. Junctions are sized so that the velocities in the merging streams are approximately equal at maximum flow. If considerable energy is available in long sewers at a junction, a series of steps may be provided in the branch to produce a cascade or it may be designed as a hydraulic jump to dissipate energy in the branch before entering the main sewer. Vertical pipe drops are used frequently at junctions for which main sewer lies well below the branch sewers, particularly if the ratio of branch sewer diameter and main sewer diameter is small. These pipe drops are designed with an entrance angle of 30 degrees with the main sewer.

3.17.4 Vertical Drops and Other Energy Dissipators

In developed areas, it may sometimes be necessary and economical to take the trunk sewers deep enough like tunnels. In such cases, the interceptors and laterals may be dropped vertically through shafts to the deep trunk sewers or tunnels. Hydraulic problems encountered with such deep vertical drops may be difficult to solve and may be some times solved by model studies. Vertical drops must be designed to avoid entrapment of air. Such entrapped air in a shaft can result in surges, which may reduce the capacity of intake. Entrapped air may not be able to flow along the sewer and escape through another ventilation shaft. Air problems can be minimised by designing a shaft with an open vortex in the middle for full depth of drop. To accomplish this, the flow is to be inducted tangentially into inlet chamber at the head of the shaft. If the vertical drop is likely to cause excessive turbulence, it may be desirable to terminate the drop in the branch to dampen the flow before it enters the main flow. Another type of vertical drop incorporates a water cushion to absorb the impact of a falling jet. Water cushion required has been found to be equal to h1/2 d1/3 in which h is the height of fall and d is depth of the crest. Special chutes or steeply inclined sewers can be constructed instead of vertical drops. All drops cause release of gasses and maintenance problems, and hence shall be avoided to the extent where possible.

3.17.5 Inverted Siphon

When a sewer line dips below the hydraulic grade line, it is called an inverted siphon. The purpose is to carry the sewer under the obstruction and regain as much elevation as possible after the obstruction is passed. They shall be resorted to only where other means of passing the obstruction are not feasible, as they require considerable attention in maintenance. As the siphons are depressed below the hydraulic grade line, maintenance of self-cleansing velocity at all flows is very important. Two considerations that govern the profile of a siphon are provision for hydraulic losses and ease of cleaning. It is necessary to ascertain the minimum flows and the peak flows for design.

To ensure self-cleansing velocities for the wide variations in flows, generally, two or more pipes not less than 200 mm diameter are provided in parallel so that up to the average flows, the first pipe is used and when the flow exceeds the average, the balance flow is taken by the second and subsequent pipes. Siphons may need cleaning more often than gravity sewers and hence shall not have any sharp bends either horizontal or vertical. Only smooth curves of adequate radius shall be used and the entry and exit piping shall be at a slope of as close to 30 degrees to horizontal. The design criteria for inverted siphons are given in IS 4111, Part III. Some of the important criteria are given below.

3.17.6 Hydraulic Calculations for Inverted Siphon

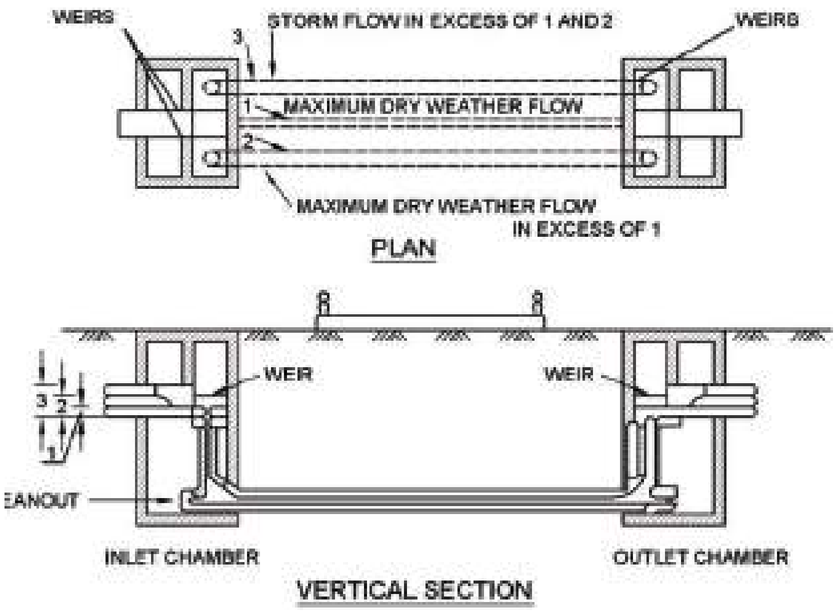
As the inverted siphon is a pipe under pressure, a difference in the water levels at the inlet and outlet is the head under which the siphon operates. This head shall be sufficient to cover the entry, exit and friction losses in pipes. The friction loss through the barrel will be determined by the design velocity. The Hazen-Williams formula can be used for calculation of head loss.

3.17.7 Velocity in Inverted Siphon Sewers

It is necessary to have a self-cleansing velocity of 1.0 m/s for the minimum flow to avoid deposition in the line.

3.17.8 Size and Arrangement of Pipes

In the multiple-pipe siphons, the inlet shall be such that the pipes come into action successively as the flow increases. This may be achieved by providing lateral weirs with heights kept in accordance with the depth of flow at which one or more siphon pipes function. Figure 3.14 gives the general arrangement for a three-way siphon. In the two-pipe siphon, the first pipe shall take 1.25 to 1.5 times the average flow and second shall take the balance of the flow.



Source: CPHEEO, 1993

Figure 3.14 Inverted siphon or suppressed sewer for combined sewage

3.17.9 Inlet and Outlet Chambers

The design of inlet and outlet chambers shall allow sufficient room for entry for cleaning and maintenance of siphons. The outlet chambers shall be so designed as to prevent the backflow of sewage into pipes which are not being used at the time of minimum flow.

3.17.10 General Requirements

Provision shall be made for isolating the individual pipes as well as the siphon to facilitate cleaning. This can be done by providing suitable penstocks or stop boards at the inlet and outlet of each pipe and by providing stop valve at its lower point if it is accessible. A manhole at each end of the siphon shall be provided with clearance for rodding. The rise, out of the siphon for small pipes shall be on a moderate slope so that sand and other deposits may be moved out of the siphon. The rising leg shall not be so steep as to make it difficult to remove heavy solids by cleaning tools that operate on hydraulic principle. Further, there shall be no change of diameter in the barrel since this would hamper cleaning operation. It is desirable to provide a coarse screen to prevent the entry of rags etc. into the siphon.

Proper bypass arrangements shall be provided from the inlet chamber and if required special arrangements shall be made for pumping the sewage to the lower reach of sewer line. Alternatively, a vacuum pump may be provided at the outlet to overcome maintenance problems arising out of clogging and silting of siphons. If it is possible a blow off may be installed at the low point to facilitate emergency maintenance operations.

Positive pressure develops in the atmosphere upstream of a siphon because of the downstream movement of air induced by the sewage flow. This air tends to exhaust from the manhole at the siphon inlet. The exiting air can cause serious odour problems. Conversely, air is drawn in at the siphon outlet. Attempts can be made to close the inlet structure tightly so that the air gets out at manholes or vents upstream. However, this causes depletion of oxygen in the sewer and leads to sulphide generation. To avoid this, sufficient ventilation arrangements have to be provided.

3.18 LEAPING WEIRS FOR SEGREGATING STORM FLOWS

Even though sometimes sewers are required to be designed for accommodating the storm flows as a combined sewer, it may not be necessary to design the treatment plant for the full combined flow. The classic principle that storm water being lighter in density will be floating over the denser sewage is recognized to design the leaping weirs by which the lighter storm water is diverted before the treatment plant. The two variations of this facility are presented herein.

3.18.1 Side Flow Leaping Weirs

A side flow weir constructed along one or both sides of a combined sewer delivers excess flows during storm periods to relief sewers or natural drainage courses. The crest of the weir is set at an elevation corresponding to the desired depth of flow in the sewer. The weir length must be sufficiently long for effective regulation. The length of the side flow weir is given by the formula devised by Babbitt.

$$L = 7.6 \times 10^{-4} \times V \times D \times \text{Log}\,(h_1/h_2) \qquad (3.22)$$

where,

L : Required length in m
V : Velocity of approach in m/s
D : Dia of the sewer in mm
h1, h2 : Heads in m above the crest of the weir upstream and downstream

The formula is limited to conditions in which the weir is placed in the side of a circular pipe at a distance above the bottom, greater than d/4 and less than d/2 where 'd' is the diameter of the pipe and the edge of the weir is sharp and parallel to the invert of the channel. Its usefulness is limited in that it was devised for pipes between 450 and 600 mm in diameter and where the depth of flow above the weir should not exceed 3d/4. A typical sideways leaping weir is shown in Figure 3.15.

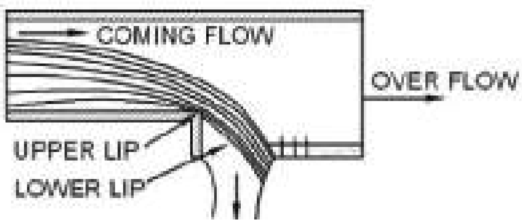
Incoming flows in excess of the desired dry weather flow will "leap" over the weirs on the sides to be diverted to storm drans. This can also be used for a flow equalisation to the STP. When the sewage level drops below the weir lip, submersible pumps can pump back the sewage from the bottom well so that a constant rate of flow can be maintained to the STP. The downstream screen after this unit and Parshall flume with stilling well and level metre are also seen.

Source: Daniel Sztruhar et.al.

Figure 3.15 Typical sideways leaping weir

3.18.2 Floor Level Leaping Weir

A floor-level leaping weir is formed by a gap in the invert of a sewer through which the dry-weather flow falls and over which a portion of the entire storm leaps. Leaping weirs have the advantage of operating as regulators without moving parts, but the disadvantage of concentrating grit in the low flow channel. Some formulae based in empirical findings are available for design. However, from practical considerations, it is desirable to design the weirs with moving crests to make the opening adjustable as indicated in Figure 3.16. A typical-floor level leaping weir is shown in Figure 3.17 overleaf.



Source: CPHEEO, 1993

Figure 3.16 Leaping weir

The incoming sewage (towards the reader), will drop through the floor level weir and be taken away through the pipe to the right of the reader. During heavy inflows, the excess quantity will leap past the floor level weir and led away to either storm sewer or balancing tank in case of STP to a balancing tank below and to be pumped back during low flows for constant flow to the STP.

Source: Daniel Sztruhar et.al.

Figure 3.17 Typical floor level leaping weir

3.19 RELIEF SEWERS

An overloaded existing sewer may require relief, with the relief sewer constructed parallel to the existing line. Relief sewers are also called supplementary sewers. In the design, it must be decided whether (a) the proposed sewer is to share all the rates of flow with the existing sewer or (b) it is to take all flows in excess of predetermined quantity or (c) it is to divert a predetermined flow from the upper end of the system.

The topography and available head may dictate which alternative is selected. If flows are to be divided according to a ratio, the inlet structure to the relief sewer must be designed to divide the flow. If the relief sewer is to take all flows in excess of a predetermined quantity, the excess flow may be discharged through a weir to the relief sewer. If the flow is to be diverted in the upper reaches of a system, the entire flow at the point of diversion may be sent to the relief sewer or the flow may be divided in a diversion structure.

A decision as to the method of relief to be chosen depends on available velocities. Self-cleansing velocities have to be maintained in both sewers even after diversion of flows. Otherwise, nuisance conditions may result. If the relief sewer is designed to take flows in excess of a fixed quantity the relief sewer itself will stand idle much of the time and deposits may occur, in some cases, it might be better to make the new sewer large enough to carry the total flow and abandon the old one.

PART - 3 DESIGN OF SEWER NETWORK

3.20 BASIC INFORMATION

Before the sewer network can be designed, accurate information regarding the site conditions is essential. This information may vary with the individual scheme but, shall in general, be covered by the following:

a) Site plan - A plan of the site to scale with topographical levels, road formation levels, level of the outfall, location of wells, underground sumps and other drinking water sources

b) The requirements of local bye-laws

c) Subsoil conditions - Subsoil conditions govern the choice of design of the sewer and the method of excavation

d) Location of other services (such as position, depth and size of all other pipes, mains, cables, or other services, in the vicinity of the proposed work)

e) Topography

3.20.1 Preliminary Investigation for Design of Sewer System

The anticipation of future growth in any community in terms of population or commercial and industrial expansion forms the basis for preparation of plan for providing the amenities including installation of sewers in the area to be served. The anticipated population, its density and its waste production is generally estimated for a specified planning period. The recommended planning period is 30 years; however, this may vary depending upon the local conditions. The prospective disposal sites are selected and their suitability is evaluated with regard to physical practicability for collection of sewage, effects of its disposal on surrounding environment and cost involved.

3.20.2 Detailed Survey

The presence of rock or underground obstacles such as existing sewers, water lines, electrical or telephone wires, tunnels, foundations, etc., have significant effect upon the cost of construction. Therefore, before selecting the final lines and grades for sewers necessary information regarding such constructions is collected from various central and state engineering departments.

Besides the location of underground structure, a detailed survey regarding the paving characteristics of the streets, the location of all existing underground structures, the location and basement elevations of all buildings, profile of all streets through which the sewer will run, elevations of all streams, culverts and ditches, and maximum water elevations therein are also made. The above details are noted on the map. The scale of the map may vary depending upon the details desired. It is recommended to adopt the following scales for various plans and drawings depending upon the detailed information desired.

(a) Index Plan	-	1 : 100,000 or 1 : 200,000
(b) Key Plan and general layout	-	1 : 10,000 or 1 : 20,000
(c) Zonal Plans	-	1 : 2,500 or 1 : 5,000
(d) Longitudinal sections of sewers	-	1 : 500 or 1 : 2,250 or 1 : 2,500
(e) Structural drawings	-	1 : 20 or 1 : 50 or 1 : 100 or 1 : 200

3.20.3 Layout of System

The sewer system layout involves the following steps:

(a) Selection of an outlet or disposal point

(b) Prescribing limits to the drainage valley or Zonal Boundaries

(c) Location of Trunk and Main Sewers

(d) Location of Pumping Stations if found necessary

In general, the sewers will slope in the same direction as the street or ground surface and will be connected to trunk sewers. The discharge point may be a treatment plant or a pumping station or a water course, a trunk sewer or intercepting sewer. It is desirable to have discharge boundaries following the property limits. The boundaries of sub zones are based on topography, economy or other practical consideration.Trunk and main sewers are located in the valleys. The most common location of sanitary sewer is in the centre of the street. A single sewer serves both sides of the street with approximately same length for each house connection.

In very wide streets it may be economical to lay a sewer on each side. In such cases, the sewer may be adjacent to the road curb or under the footpath & interference with other utilities has to be avoided. Sometimes sewers may be located in the back of property limits to serve parallel rows of houses in residential area. However, access to such locations becomes difficult and hence sewer locations in streets are often preferred. Sewers as a rule are not located in proximity to water supplies. When such situations are unavoidable the sewers may be encased in sleeve pipes or encased in concrete.

The Puducherry Public Works Department has been historically adopting a practice of connecting the house services of a few houses by a rider sewer on the foot path with chambers and then connect to the sewer manhole in the road as in Figure 3.18 (overleaf).

A tentative layout is prepared by marking sewer lines along the streets or utilities / easements. The direction of flow is shown using arrows, which is generally the direction in which the ground slopes. Manholes are provided at all sewer intersections, changes in horizontal direction, major change in slopes, change in size and at regular intervals.

The depth of cut is dictated by the need to ensure a minimum cover and the desirability of mandatory cushion depending upon the pipe size and expected loads.

It is the standard design practice to provide a minimum cover of 1 m at the starting point in the case of sanitary sewer network and 0.5 m for storm drainage system.

Photograph on left shows connection from interceptor chamber to manhole and photograph on right shows connection from chamber to manhole

Source: Puducherry PWD

Figure 3.18 House sewer connections

If the sewer changes in direction in a manhole without change of size, a drop of usually 30 mm is provided in the manhole. If the sewer changes in size, the crown of inlet and outlet sewers are set at the same elevation. The vertical drop may be provided as described in Section 3.17. Sewers as a design practice are not located in proximity to water supplies. When such situations are unavoidable, the sewers should be encased in sleeve pipes or encased in concrete. Even when a plot is empty, it is better to lay the house service connection sewer to the nearest manhole from a temporary chamber in the vacant plot and plug it.

3.20.4 Profile of Sewer System

The vertical profile is drawn from the survey notes for each sewer line. All longitudinal sections are indicated with reference to the same datum line. The vertical scale of the longitudinal sections are usually magnified ten times the horizontal scale. The profile shows ground surface, tentative manhole locations, grade, size and material of pipe, ground and invert levels and extent of concrete protection, etc. At each manhole the surface elevation, the elevation of sewer invert entering and leaving the manhole are generally listed.

3.20.5 Available Head

Generally, the total available energy is utilized to maintain proper flow velocities in the sewers with minimum head loss. However, in hilly terrain excess energy may have to be dissipated using special devices. Hence, the sewer system design is limited on one hand by hydraulic losses, which must be within the available head and on the other to maintain self-cleansing velocities. It becomes difficult to meet both conditions with increasing variation in rate of flow. Where differences in elevations are insufficient to permit gravity flow, pumping may be required. The cost of construction, operation, and maintenance of pumping stations are compared with the cost of construction and maintenance of gravity sewers. Apart from the cost considerations the consequences of mechanical and electrical failures at pumping stations may also be considered, which may necessitate a gravity system even at a higher cost.

3.20.6 Plans and Nomenclature

The following procedure is recommended for the nomenclature of sewers:

- First distinct number such as 1, 2, 3, etc., is allotted to the manholes of the trunk sewers commencing from the lower end (outfall end) of the line and finishing at the top end.
- Manholes on the mains or sub mains are again designated numbers 1, 2, 3, etc., prefixing the number of the manhole on trunk/main sewer where they join. Similar procedure is adopted for the branches to branch main. When all the sewer lines connected to the main line have thus been covered by giving distinctive numbers to the manholes, the manholes on the further branches to the branch mains are similarly given distinctive numbers, again commencing with the lower end.
- If two branches, one on each side meeting the main sewer or the branch sewer, letter 'L' (to represent left) or letter 'R' (to represent right) is prefixed to the numbering system, depending on the direction of flow.
- If there is more than one sewer either from the left or right they are suitably designated as L_1, L_2, L_3, or R_1, R_2, R_3, the subscript refer to the line near to the sewer taking away the discharge from the manhole.

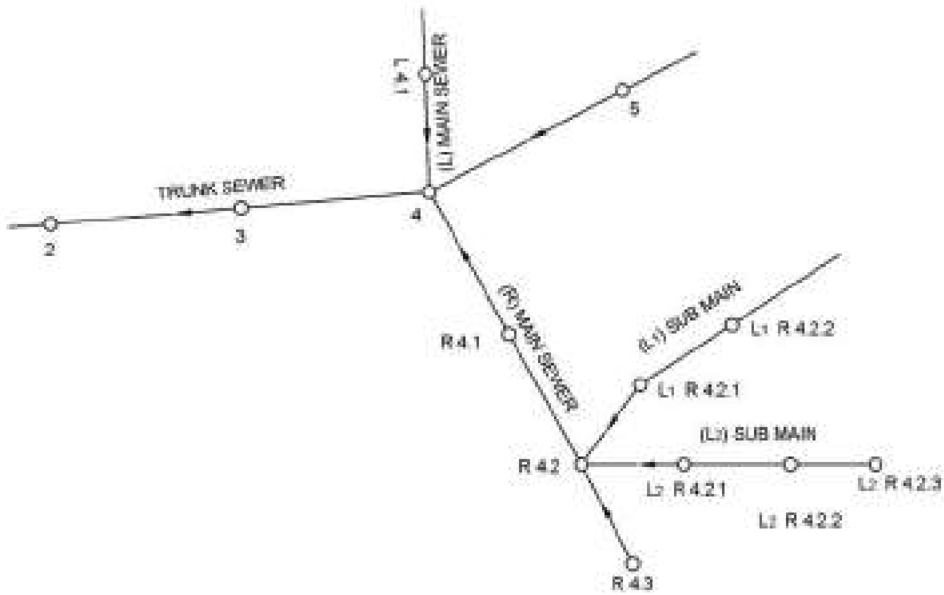
Thus, L_2R4.2.3 (Figure 3-19 overleaf) will pinpoint a particular manhole on the sub main from which the flow reaches manhole number 4 on the trunk sewer through a sub main and a main. The first numeral (from the left) is the number of the manhole on the trunk sewer. The numerals on the right of this numeral, in order, represent the manhole numbers in the main, sub main, etc., respectively.

The first letter immediately preceding the numeral denotes the main and that it is to the right of the trunk sewer. Letters to the left in their order represent sub main, branch respectively. The same nomenclature is used for representing the sections, e.g. Section L_2R4.2.3 identifies the section between the manhole L_2R4.2.3 and the adjoining downstream manhole.

All longitudinal sections should be indicated with reference to the same datum line. The vertical scale of the longitudinal sections should be magnified ten times the horizontal scale.

The trunk sewer should be selected first and drawn and other sewers should be considered as branches. The trunk sewer should be the one with the largest diameter that would extend farthest from the outfall works. Whenever two sewers meet at a point, the main sewer is the larger of the incoming sewers. (e.g., 3.2 represents the second manhole on the main sewer from the manhole no. 3 on the trunk sewer).

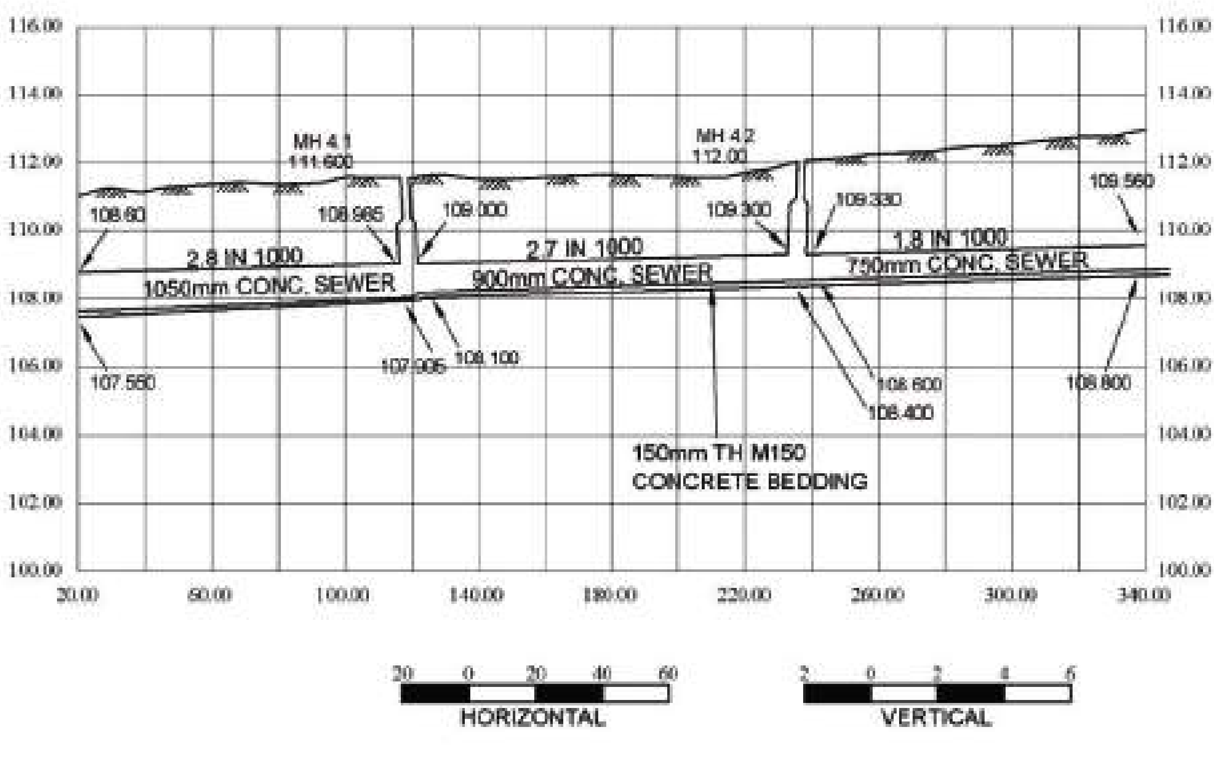
Once the rough sections have been prepared, the designer should review the work for improving the spacing of manholes, the sizes and gradients of the sewers and so forth, economising on materials and excavation to the extent possible. At the same time the designer must ensure that the sewer will serve all users and that they can be actually laid according to the alignments shown in the drawing and have sufficient gradients.



Source: CPHEEO, 1993

Figure 3.19 Nomenclature of sewers

The sewers should be shown as thick lines and the manholes as small circles in the plan. In the section, the sewer may be indicated by a line or two lines depending upon the diameters and scales adopted. Grade, size and material of pipe, ground and invert levels and extent of concrete protection should be indicated as shown in Figure 3.20.



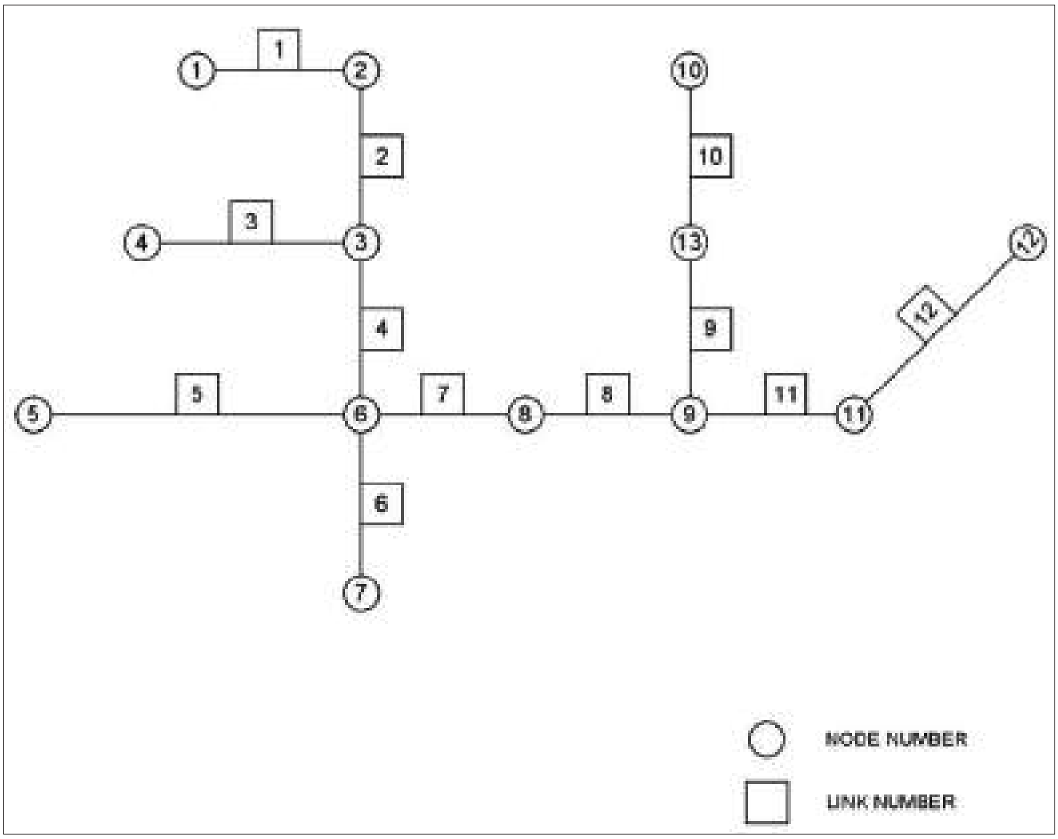
Source: CPHEEO, 1993

Figure 3.20 Typical sewer section

Standard vertical plan filing systems are now available and are very convenient for storing of plans and taking them out quickly for reference. Normally, size A0 and A1 (trimmed size 841   1,189 mm and 594   841 mm respectively) should be used along with soft copies on CD / DVD while submitting the project drawings for approval.

All documents including drawings, design calculations, measurement sheets of estimates, etc., should be in metric system. In drawings, length should be indicated either entirely in meters, corrected up to two decimals or entirely in mm (for thickness etc.). If this practice is followed, units would be obvious and in certain cases, writing of m or mm with the figure can be omitted. The flow should normally be indicated in litres per second (lps) or cubic meters per hour (m^3/hr) except for very large flows which may be indicated in cubic meters per second (cum/sec). For uniformity, lps for sewage flows and cum/sec for storm flows is recommended. Similarly, areas in sewer plans and design calculations may be indicated in hectares (ha). While writing figures they should be grouped into groups of three with a single space between each group and without comma. In case of a decimal number, this grouping may be on either side of the digit (e.g., 47 342.294 31).

In case of design of sewer network using computer programme, there is no restriction in the nomenclature of the sewers and manholes as required for the manual design. It is sufficient to give node numbers as well as pipe (link) numbers in any manner in the sewer network for design of the network for using computer software. The numbering of the network may be adopted as shown in Figure 3.21 and illustrated in Appendix A.3.7.



Source: CPHEEO, 1993

Figure 3.21 Example sewer network

3.20.7 Precautions

Design of sewer systems for rocky strata especially in hilly terrain in walled cities may have to invoke controlled blasting or chipping and chiseling both of which can cause hindrance to traffic for long periods of time and may also cause damages to heritage structures. In such situations, it is necessary to consider the shallow sewer options on both sides of the roads and if drains are already in position, construction of the additional twin of the drain and manage the collection system. The herringbone cutting for house service connections damages the roads in construction and O&M.

PART 4 TYPES AND CONSTRUCTION OF MANHOLES

3.21 DEFINITION

A manhole is an opening through which a man may enter a sewer for inspection, cleaning and other maintenance and is fitted with a removable cover to withstand traffic loads in sewers. The manholes first constructed before the sewers are laid interconnecting these. The stated depths of sewers and diameter of circular manholes are in Table 3.15.

Table 3.15 Diameters of circular manholes for stated depths of sewers

Z Values	Sulphide Condition
Z < 5,000	Sulphide rarely generated
5,000 ≤ Z ≤ 10,000	Marginal condition for sulphide generation
Z > 10,000	Sulphide generation common

Source: CPHEEO, 1993

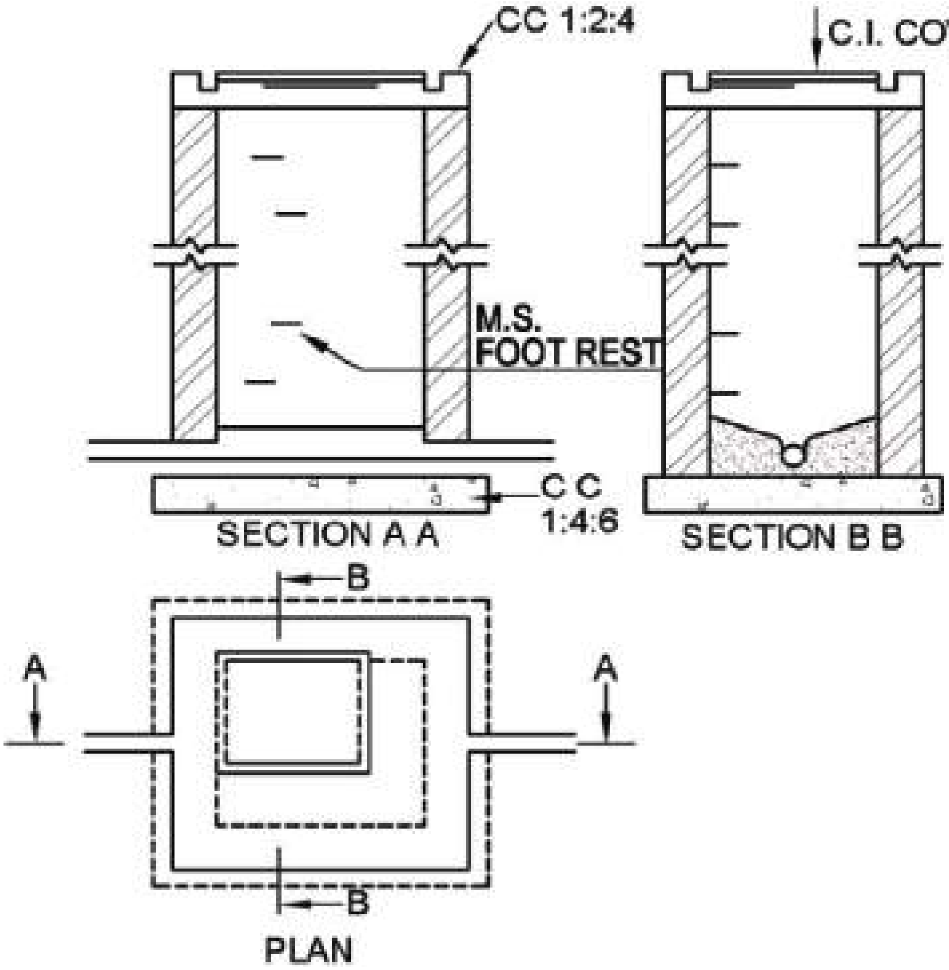
Note:
Where depths exceed 6 m below GL, lift stations as in Section 4.18 shall be inserted and sewage lifted to initial cover depth of 0.9 m.
In specific situations deeper depth can be justified in the DPR like outfall, etc

3.22 TYPES OF BRICKWORK MANHOLES

These are shown in Figure 3.22, Figure 3.23, Figure 3.24 and Figure 3.25.

3.23 RCC AND COMBINATION MANHOLES

In lieu of entire brickwork, RCC or RCC with brickwork combination manholes have the advantage of better quality control in raw materials and workmanship, besides easier fixing in the field with maximum speed and minimum disturbance to traffic. This is admittedly advantageous especially in case of difficulties in obtaining good bricks and the non-availability of trained masons in getting the corbelled cone portion and lapses there can lead to potential fatal accidents on public roads. There is however the issue of the concern about the concrete corrosion of the inside by sulphide gas and the soil side by sulphate in soil water. In view of this, the use of high alumina cement is advisable in manufacture itself or sulphate resistant cement with extra lining of 25 mm thickness over inner wall with high alumina cement.



Source: CPHEEO, 1993

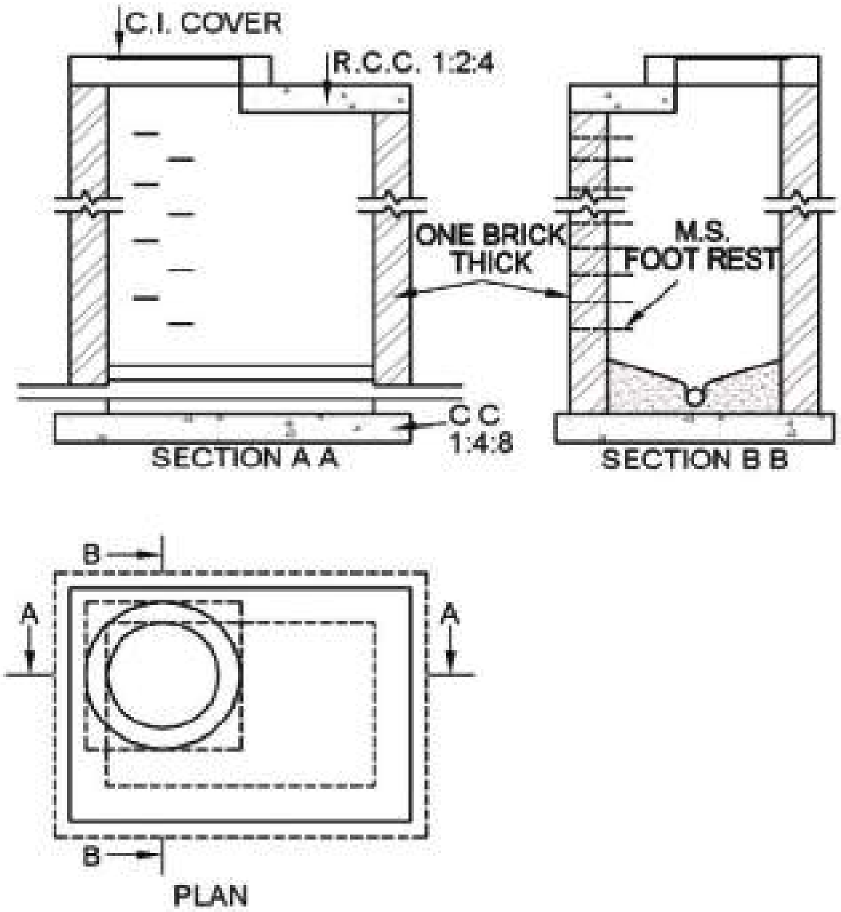
Figure 3.22 Rectangular manhole for 0.9 m × 0.8 m clear in plan and depth less than 0.9 m

Two types of RCC manholes can be used -

- Manholes with vertical shaft in RCC and the corbelled cone portion in brickwork
- Entire manhole in RCC and corbelled cone portion separately precast and jointed

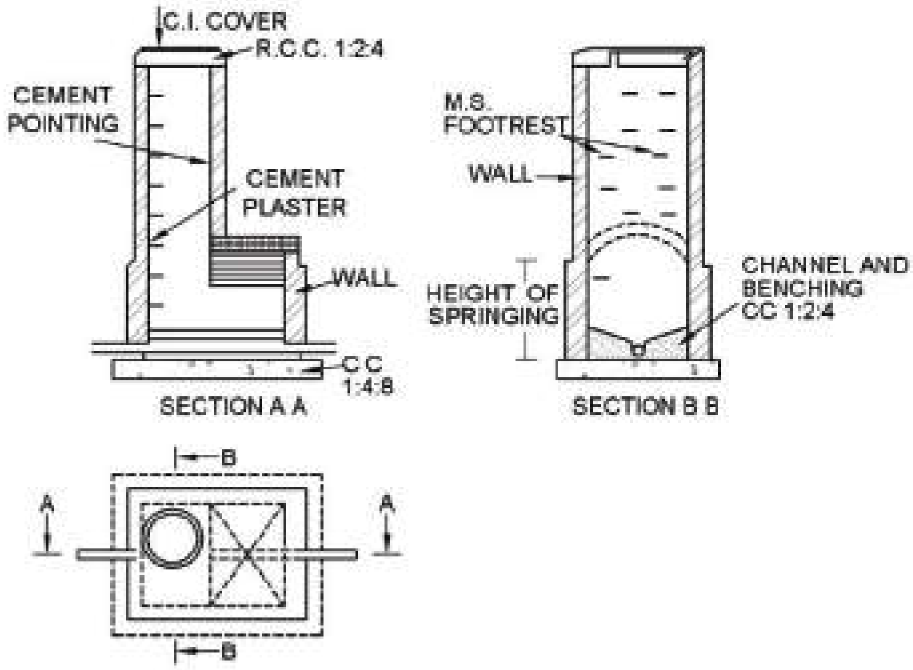
The entries and exits of main sewers as well as house service sewers requires careful detailing because the issue of puncturing the walls for insertions of especially house service sewers later on is impossible. These shall be managed as detailed below.

- The corbelled cone portion which is eccentric with one vertical edge, shall be separately cast and its design standardized with respect to the diameter of its base.
- The vertical shaft is best pre-cast to have a better quality control of raw materials and workmanship, which is otherwise very suspect in local situations of every manhole.
- The shaft itself shall be made of rings with lap joints of the annular rim and duly jointed at site by cement mortar or elasto-polymers.



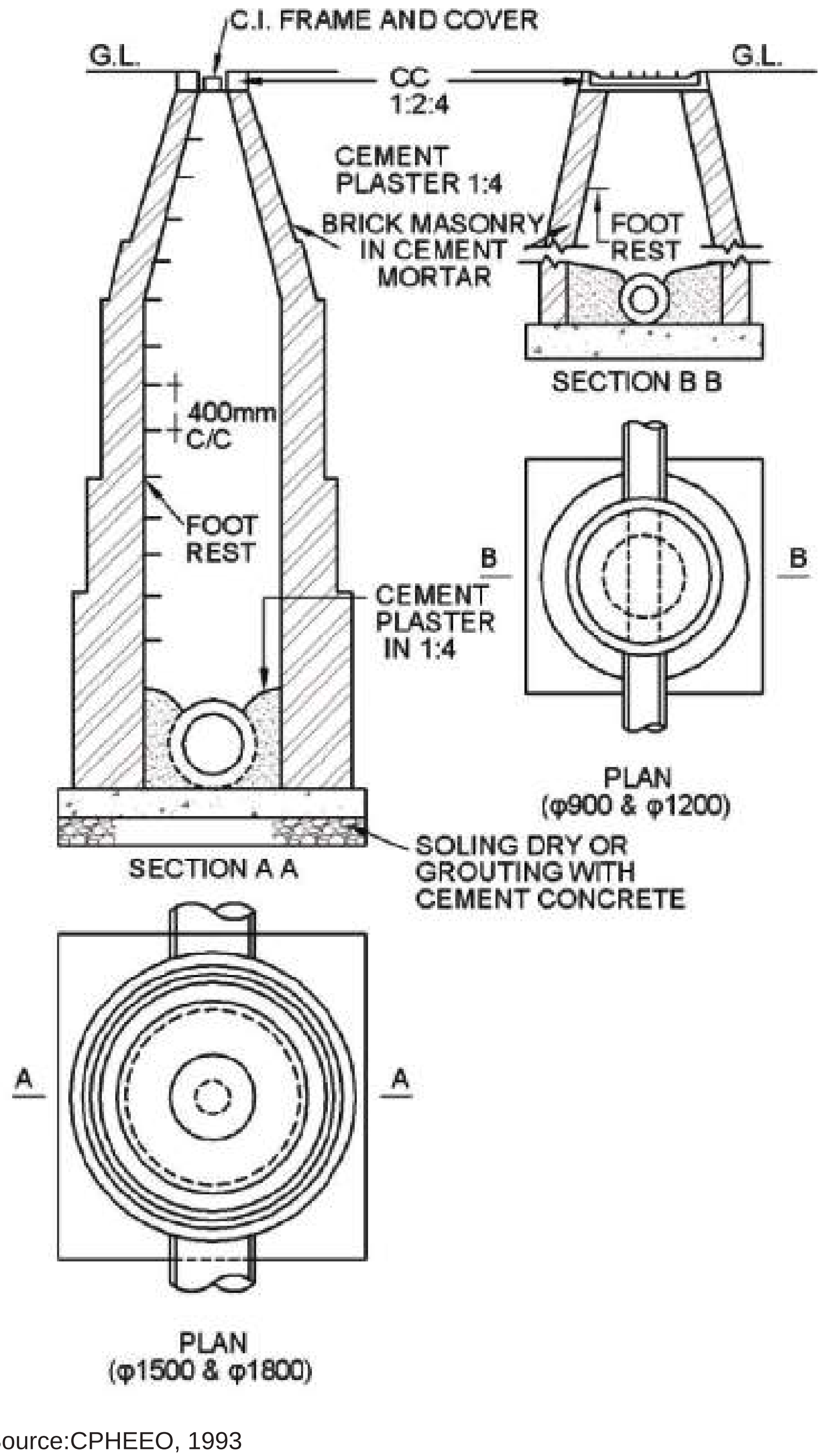
Source: CPHEEO, 1993

Figure 3.23 Rectangular manhole for 1.2 m × 0.9 m clear in plan and depth 0.9 m to 2.5 m



Source: CPHEEO, 1993

Figure 3.24 Arch type manhole for 1.4 m × 0.9 m clear in plan and deeper than 2.5 m



Source:CPHEEO, 1993

Figure 3.25 Typical circular manhole of diameters and depths as in Table 3-15

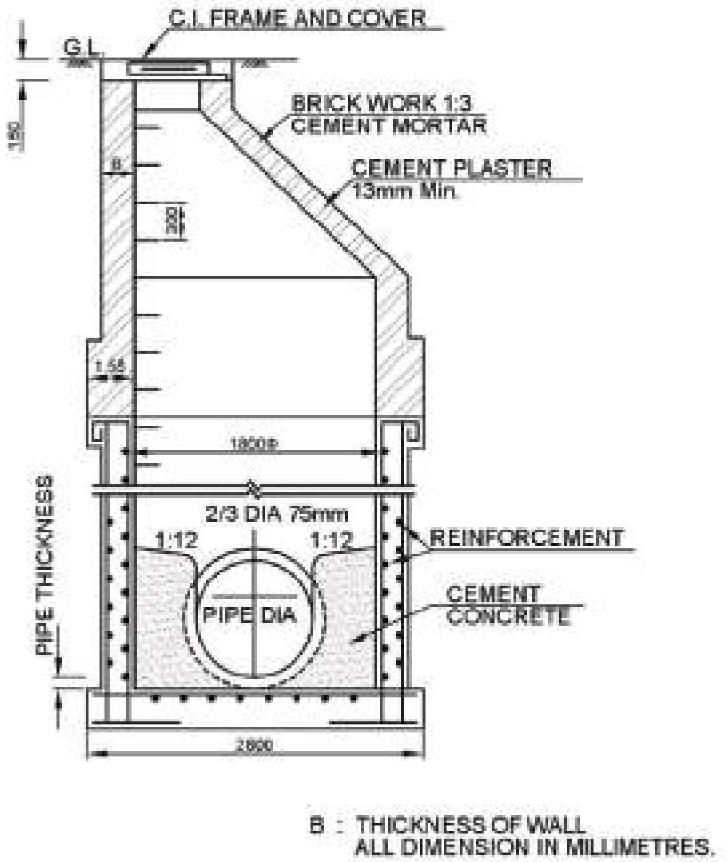
- The varying heights of the manhole are obtained by choosing the bottom ring deeper than the fractional height needed there and filling up the bottom floor after placing the ring such that the invert level of the sewer is obtained thereby.

- This ring shall have a vertical inverted U cut out in casting itself to insert the sewer pipes and caulk the annular space using cement concrete with cement-based water proofing admixtures. The dimensions of the U cut out shall be standardized to match the OD of proposed sewers and a clear cover of 50 cm all round for caulking.

- The position of the vertical inverted U cut outs will normally be 180 degrees apart in plan but in cases of junction manholes and drop manholes it may be at differing angles in plan and needs to be precast suitably and shall not be chiseled out in the field.

- For insertion of the house service sewers into the manholes, it is necessary to have a precast ring section below the corbel portion, with holes at 45 degrees to the public sewer line to facilitate insertion of three house service sewers on each side of the public sewer axis. Usually the house service sewers shall be 110 mm or 160 mm UPVC 4 kg/sqcm (as detailed in sewer laying section). Accordingly, the height of the ring shall be 250 mm and 300 mm to permit filling of the annular interspaces between the sewer and the opening with cement concrete of at least 50 mm around the finished sewer.

A typical example of a combination manhole is shown in Figure 3.26, Figure 3.27 and Figure 3.28.



Source:CPHEEO, 1993

Figure 3.26 RCC and Brickwork Combination Manhole

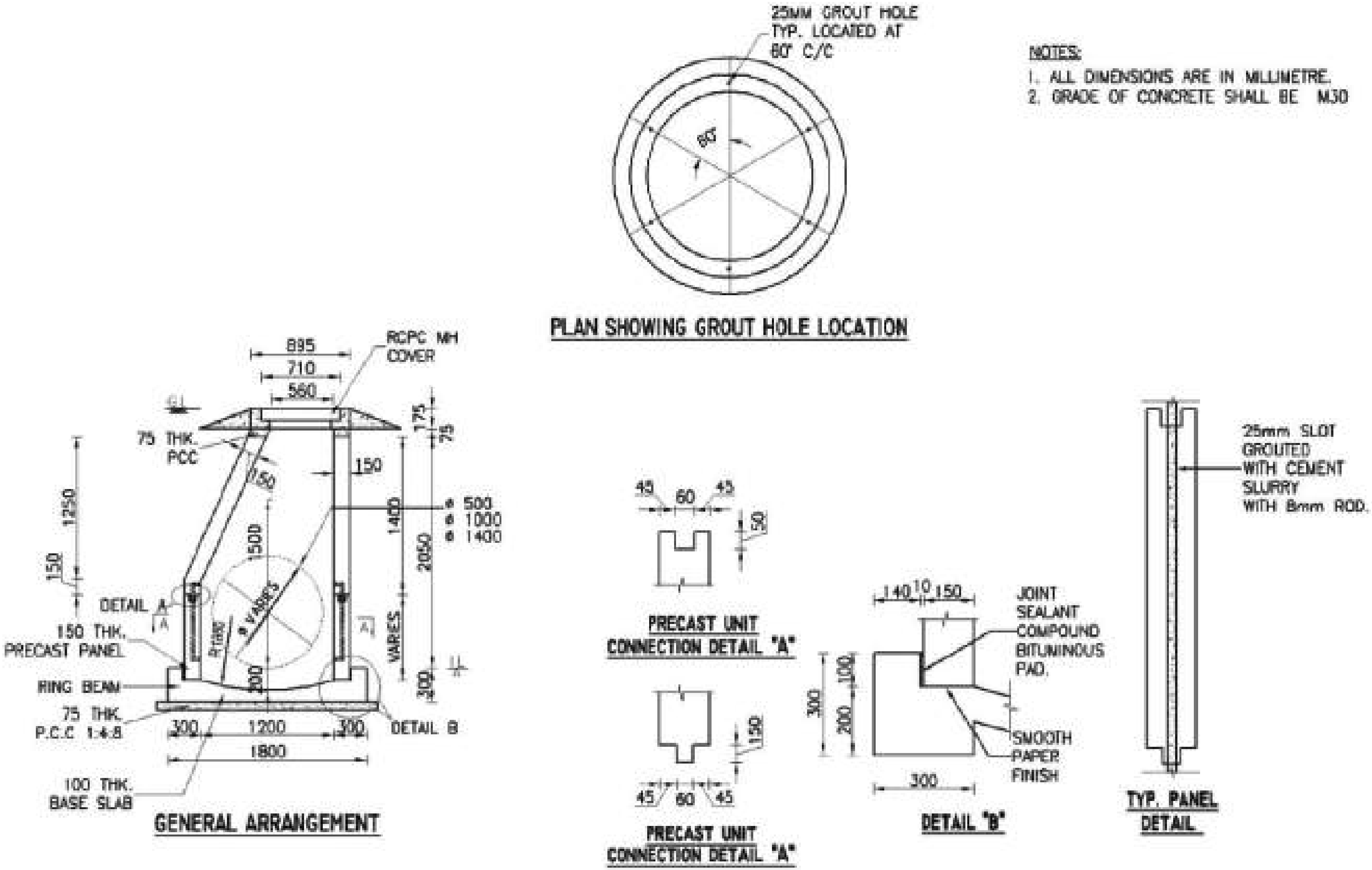


Figure 3.27 RCC Manhole for a depth between 0.6 m to 2.3 m

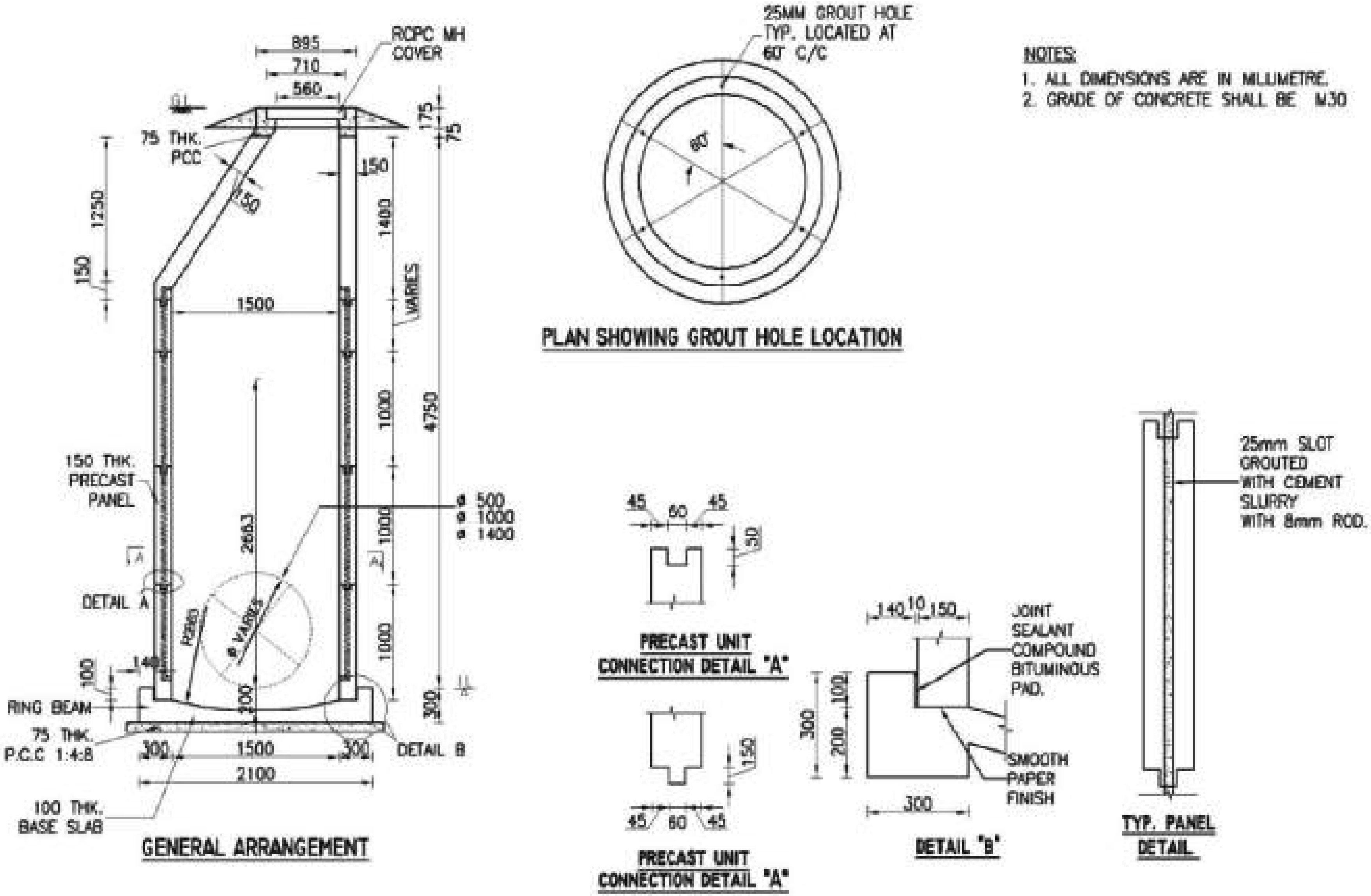


Figure 3.28 RCC Manhole for a depth between 2.3 m to 5 m

3.24 HDPE MANHOLES

HDPE manholes with EN 13598-2: 2009 and ISO (ISO 9001: 2008) specifications are recent entrants. However, the Indian standards are yet to be brought out by BIS. These being ready made can speed up the construction as compared to brickwork manholes. However, if desired for a specific location they are to be safeguarded against the uplift pressure due to high ground water table and crushing under high traffic load etc. by suitably anchoring and, the cost of these shall not be compromised.

3.25 DROP MANHOLES

Difference in elevations of incoming and outgoing sewers, which would result in holding up of solids that can cause nuisance to the maintenance personnel, should be avoided. When it is necessary to drop the elevation of the sewer at a manhole, the drop should be made by means of an outside connection - in this regard, the dimensions of the related fittings govern the minimum vertical outside drop that can be made.

The designer’s judgment will determine, in each case, where the difference in elevation warrants using an outside drop instead of lowering the upstream or branch sewer. The outside connection is provided for the protection of the person who may enter the manhole. Therefore, sometimes when a lateral sewer joins a deeper, sub main sewer or the use of a drop manhole will reduce the amount of excavation needed by allowing the lateral to maintain a shallow slope. The sewage drops into the lower sewer through the vertical pipe at the manhole.

Encasement of the entire outside drop in concrete or brick masonry is needed to protect it against damage during the backfilling of the trench. Maintenance may be facilitated by providing a cross instead of a tee at the top of the vertical drop, with a cast iron riser from the cross to the surface of the ground where a cast iron lamp hole frame and cover are installed. When such a drop is plugged, a ball or a chain is dropped down to break any sticks, thereby permitting the plugging material to be washed out.

When a sewer connects with another sewer, and the difference in level between the sewers of the main line and the invert level of branch line is more than 600 mm or a drop of more than 600 mm is required to be given in the same sewer line, it is uneconomical or impractical to arrange the connection within 600 mm. At that point, a drop connection shall be provided for which a manhole may be built incorporating a vertical or nearly vertical drop pipe from the higher sewer to the lower sewer.

This pipe may either be outside the shaft and encased in concrete or may be supported on brackets inside the shaft, which should be suitably enlarged.

If the drop pipe is outside the shaft, a continuation of the sewer should be built through the shaft wall to form a rodding and inspection eye. This should be provided with a half-blank flange.

If the drop pipe is inside the shaft, if should be in cast iron and it would be advantageous to provide adequate means for rodding and a cushion of 150 mm depth should also be provided.

The drop pipe should terminate at its lower end with a plain or duck-foot bend turned so as to discharge its flow at 45 degrees or less to the direction of the flow in the main sewer and the pipe, shall be cast iron, or surrounded with 150 mm of concrete. In the case of sewers that are over 450 mm in diameter the drop in level may be accomplished by one of the following methods:

a) A Cascade

This is a steep ramp composed of steps, over which the flow is broken up and retarded. A pipe connecting the two levels is often concreted under the steps to allow small flows to pass without trickling over the steps. The cascade steps may be made of heavy duty bricks of Class I quality IS 2180, cement concrete with granolithic finish or dressed granite.

b) A Ramp

A ramp may be formed by increasing the grade of the last length of the upper sewer to about 45 degrees or by constructing a steeply graded channel or culvert leading from the high level to the low level sewer. In order to break up the flow down the ramp and minimize the turbulence in the main sewer, the floor of a culvert ramp should be obstructed by raised transverse ribs of either brick or concrete at 1.15 m intervals and a stilling pool provided at the bottom of the ramp.

c) By Drops in Previous Successive Manholes

Instead of providing the total drop required at the junction manhole, the same might be achieved by giving smaller drops in successive manholes preceding the junction manhole. Thus, for example, if a total drop of 2.4 m is required to be given, 0.6 m drop may be given in each of the previous three manholes and the last 0.6 m drop may be given at the junction manhole.

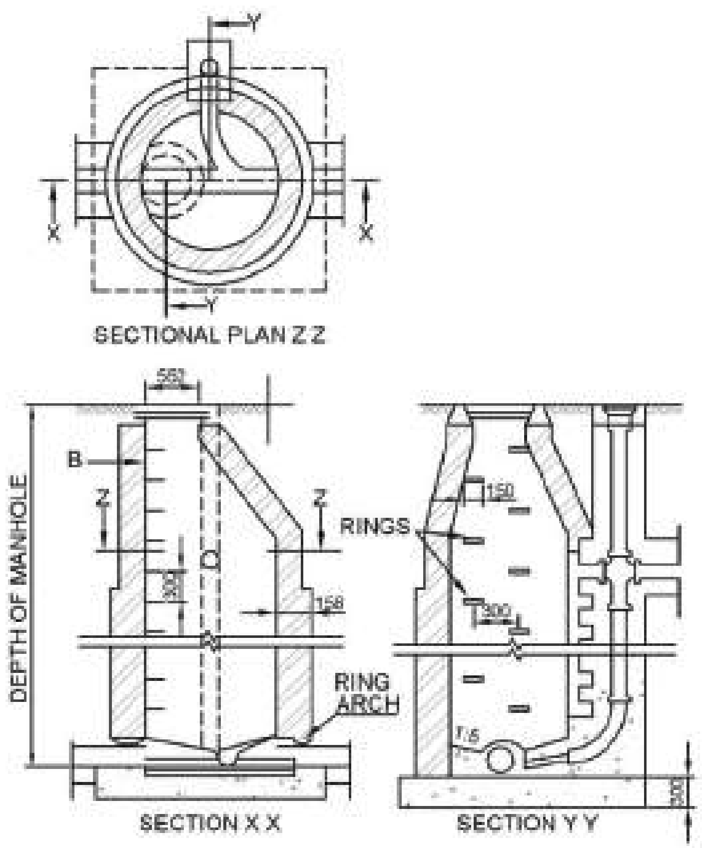
The diameter of the back-drop should be at least as large as that of the incoming pipe. A typical illustration of a drop manhole is shown in Figure 3-29 overleaf.

3.26 JUNCTION MANHOLES

A manhole should be built at every junction of two or more sewers, and the curved portions of the inverts of tributary sewers should be formed within the manhole. To achieve this with the best economy of space, the chamber may be built of a shape other than rectangular. The soffit of the smaller sewer at a junction should not be lower than that of the larger sewer, in order to avoid the surcharging of the former when the latter is running full, and the hydraulic design usually assumes such a condition. The gradient of the smaller sewer may be steepened from the previous manhole sufficiently to reduce the difference of invert level at the point of junction to a convenient amount.

3.27 SIDE ENTRANCE MANHOLES

In large sewers or where it is difficult to obtain direct vertical access to the sewer from ground level, owing to existing services, gas, water, etc., the access shaft should be constructed in the nearest convenient position off the line of sewer, and connected to the manhole chamber by a lateral passage.



Source: CPHEEO, 1993

Figure 3.29 Illustrative arrangement of drop manhole

In the tunnelled sewers the shaft and lateral access heading may be used as a working shaft, the tunnel being broken out from the end of the heading, or alternatively the shaft and heading may be constructed after the main tunnel is complete, provision having been made for breaking in from the access heading to build the chamber.

The floor of the side-entrance passage, which should fall at about 1 in 30 towards the sewer, should enter the chamber not lower than the soffit level of the sewer. In large sewers where the floor of the side entrance passage is above the soffit, either steps or a ladder (which should be protected either by a removable handrail or by safety chains) should be provided to reach the benching.

3.28 SCRAPPER (SERVICE) TYPE MANHOLE

It has been proposed in the earlier 1993 edition of the manual that the scraper manholes shall be used at specified intervals for desilting the sewer systems.

In the interim period, advancements in mechanized sewer cleaning, like jet rodding and vacuum suction machines have occurred and are being used. In order to avoid man entry into sewer manholes these scraper manholes shall be discontinued forthwith. Instead, the numbers of these mechanized sewer-cleaning equipment as recommended in Part B of the O&M manual shall be included in the project in the TOR stage itself and procured.

3.29 FLUSHING MANHOLES

Where it is not possible to obtain self-cleansing velocities due to flatness of the gradient especially at the top ends of branch sewers, which receive very little flow, it is essential that some form of flushing device be incorporated in the system. This can be done by making grooves at intervals of 45 m to 50 m in the main drains in which wooden planks are inserted and water allowed to head up and which will rush on with great velocity when the planks are removed. Alternatively, an elevated tank is built and filled with treated sewage from which connections are made through pipes and flushing hydrants to rush water to the sewers. The relevant Indian Standard is IS 4111 Part 2. Flushing can also be very conveniently accomplished by the use of a fire hydrant or tanker and hose.

Where flushing manholes are provided, they are located generally at the head of a sewer. Sufficient velocity shall be imparted in the sewer to wash away the deposited solids. The flush is usually effective up to a certain distance after which the imparted velocity gets dissipated.

The automatic systems, which are operated by mechanical units often get corroded by the sewer gases and do not generally function satisfactorily and hence, are not recommended.In case of hard choking in sewers, care should be exercised to ensure that there is no possibility of backflow of sewage into the ground and entering defective water supply mains.

Approximate quantities of water needed for flushing are given in Table 3.16

Table 3.16 Quantity of water needed for flushing

Quantity of water (Litres)			
Slope	200 mm dia.	250 mm dia.	300 mm dia.
1 in 200	2,300	2,500	3,000
1 in 130	1,500	1,800	2,300
1 in 100	1,300	1,500	2,000
1 in 50	500	800	1,000
1 in 33	400	500	700

Source:CPHEEO, 1993

A more practical and relatively safer method is to deploy the modern jet rodding machines at head manholes and use the treated sewage from the STP, but then, the cost of the machine is involved. A simpler option will be to use the possibility of positioning a butterfly valve at the head sewer mouth in the manhole and is kept open by an extended handle, which can be operated from ground level when the manhole cover is opened. After opening the manhole cover, the valve is closed by a quarter turn and the manhole is filled with treated sewage brought by a tanker from the STP. After filling, the valve is opened to enable flushing. Usually a sewer tanker can hold about 6000 litres and is adequate to flush at least two manholes per trip.

3.30 DIFFERENT DIAMETERS OF SEWERS IN THE SAME MANHOLE

Manholes should be built to cause minimum head loss and interference with the hydraulics of the sewer line. One way to maintain a relatively smooth flow transition through the manhole, when a small sewer joins one of a larger diameter, is to match the pipe crown elevations at the manhole.

3.31 TERMINAL CLEANOUT STRUCTURE

Terminal cleanouts are sometimes used at the ends of branch or lateral sewers. Their purpose is to provide a means for inserting cleaning tools for flushing or for inserting an inspection light into the sewer. In fact, a terminal cleanout amounts to an upturned pipe coming to the surface of the ground. The turn is made with bends so that flexible cleaning rods can be passed through them. The diameter of a bend should be the same as that of the sewer. The cleanout is capped with a cast iron frame and cover. Care should be taken to maintain proper alignment of the pipe while encasing it with concrete. The frame and cover of the terminal cleanout structure should be made of grey cast iron. Tees were often used, instead of pipe bends, in older engineering practice, and the structures were called “lamp holes”. Modern sewer cleaning equipment cannot be passed from the surface through such structures, so their use is to be discontinued forthwith. Terminal cleanouts are limited in usefulness and should never be used as a substitute for a manhole. They are permitted under some state regulations only at the ends of branch sewers, which may never be extended and must lie within 50 m of a manhole.

3.32 CONSTRUCTION OF BRICKWORK MANHOLES

a) If the sewer is constructed in a tunnel, the manhole should be located at the access or working shafts and the manhole chamber may be constructed of a size to suit the working shaft or vice-versa. The width/diameter of the manhole should not be less than internal diameter of the sewer + 150 mm benching on both sides (150 mm + 150 mm).

b) The opening for entry into the manhole (without cover) should be of such minimum dimensions as to allow a work with the cleaning equipment to get access into the interior of the manhole without difficulty. A circular opening is generally preferred. A minimum clear opening of 60 cm is recommended. Suitable steps, usually of malleable cast iron shall be provided for entry.

c) Access shaft shall be circular in shape and shall have a minimum internal diameter of 750 mm; where the depth of the shaft exceeds 3 m suitable dimensions shall be provided to facilitate cleaning and maintenance. Access shaft where built of brickwork, should be corbelled on three sides to reduce it to the size of the opening in the cover frame, and to provide easy access on the fourth side to step irons or ladder. In determining the sizes, the dimensions of the maintenance equipment likely to be used in the sewers, shall be kept in view.

d) The manhole base slab shall be 150 mm for manholes up to 1 m depth, 200 mm for manholes from 1 to 2 m depth and 300 mm for greater depths. In all cases, the thickness shall be counter checked for uplift conditions based on maximum ground water elevations at the site on the soil side by considering empty manhole conditions.

e) Where subsoil water condition exists, a rich mix may be used and it shall further be waterproofed with addition of approved water proofing compound in a quantity as per manufacturer's specifications.

f) The brickwork manholes shall be first constructed to the required invert and with circular openings to facilitate the laying of sewer pipes later on. These manholes facilitate the judgement in the field when trenches are dug up and sewer pipes are to be laid to give the levels using a levelling instrument or with boning rods.

g) All brickwork shaft shall be in English bond and the jointing faces being well buttered with cement mortar before laying, so as to ensure a full joint and brickwork shall be in accordance with IS 2212 code of practice for brickwork. The cement mortar used shall not be weaker than 1:3 and in accordance with IS 2250 code of practice for preparation and use of masonry mortars and its revisions.

h) The thickness of walls shall be typically one brick (23 cm) for up to I.5 m deep manholes and one and a half brick (35 mm) for depths greater than 1.5 m. The actual thickness in any case shall be verified on the basis of engineering design in difficult soil conditions.

i) The jointing of brickwork and plastering on both sides (20 mm) shall be in a mix of cement mortar 1:3. Admixtures for water proofing if desired shall be cement based.

j) Salt glazed or concrete half channel pipes of the required size and curve shall be laid and embedded in cement concrete base to the same line and fall as the sewer. These can also be finished as semi-circular channels with cement mortar 1:2 and of diameter equal to that of the sewer. Above the horizontal diameter, the sides shall be extended vertically to the same level as the crown of the outgoing pipe and the top edge shall be sloped up at 1:10 towards the wall and suitably rounded off. The branch channels shall also be similar.

k) Bricks on edge shall be cut to a proper form and laid around the upper half of all the pipes entering or leaving the manhole, to form an arch.

l) All around the pipe there shall be a joint of cement mortar 12 mm thick between the pipe and the bricks. The ends of the pipes shall be built in and neatly finished off with cement mortar.

m) The entire height of the manhole shall be tested for water-tightness by closing both the incoming and outgoing ends of the sewer and filling the manhole with water. A drop in water level not more than 50 mm per 24 hours shall be permitted.

n) It should be ensured that there is no leakage of ground water into the manhole by observing the manhole for 24 hours after emptying it.

o) The top of the manhole shall be flush with the finished road level as per IS 4111 Part I.

3.33 CONSTRUCTION OF RCC MANHOLES

The idea of RCC manholes is essentially to quicken the work of construction in the roads by adopting precast sections assembled at site.

Thus, the issues related to their construction are more of design itself and quality control in casting. The provisions of IS 456 and IS 3370 Parts I, II and IV shall inter alia apply to the design. The entire structure shall at all times be designed to the condition where the ground water is at ground level itself and the inside is empty and there is no superimposed load on the manhole to counter the uplift force and not considering the skin friction of the manhole sidewall with the soil.

3.34 COVERS AND FRAMES

The size of manhole covers should be such that there should be clear opening of not less than 560 mm diameter for manholes exceeding 0.9 m depth. When cast iron manhole covers and frames are used they shall conform to IS 1726. The frames of manhole shall be firmly embedded to correct alignment and level in plain concrete on the top of masonry. The precast frame and cover can also be of steel fibre reinforced concrete (SFRC) conforming to IS 12592 and shall be of approved make. The frame and cover shall be of LD/ MD/ HD/ EHD grade, size and thickness as mentioned in the description of the item. The standard for DI manhole covers is EN 123.

3.35 RUNGS

As per the US Department of Environmental Conservation, Model Sewer Use Law, Section 504, Manholes and Manhole Installation clause 6, “No steps or ladder rungs shall be installed in the inside or outside manhole walls at any time” (http://www.dec.ny.gov/chemical/8729.html). This implies the total ban on man entry into manholes (leave alone the nomenclature) and in turn underscores the fully mechanized methods of attending to sewer blocks. Though it is the ideal condition, the relatively lesser per capita water supply rates and the absence of strict enforcement of grinders below faucets in kitchens etc. and handicapped financial positions of local bodies defy the adoption of such an ideal situation in our country for some more time and may be adopted in stages starting from mega cities. Till such time, the rungs shall continue to be used.

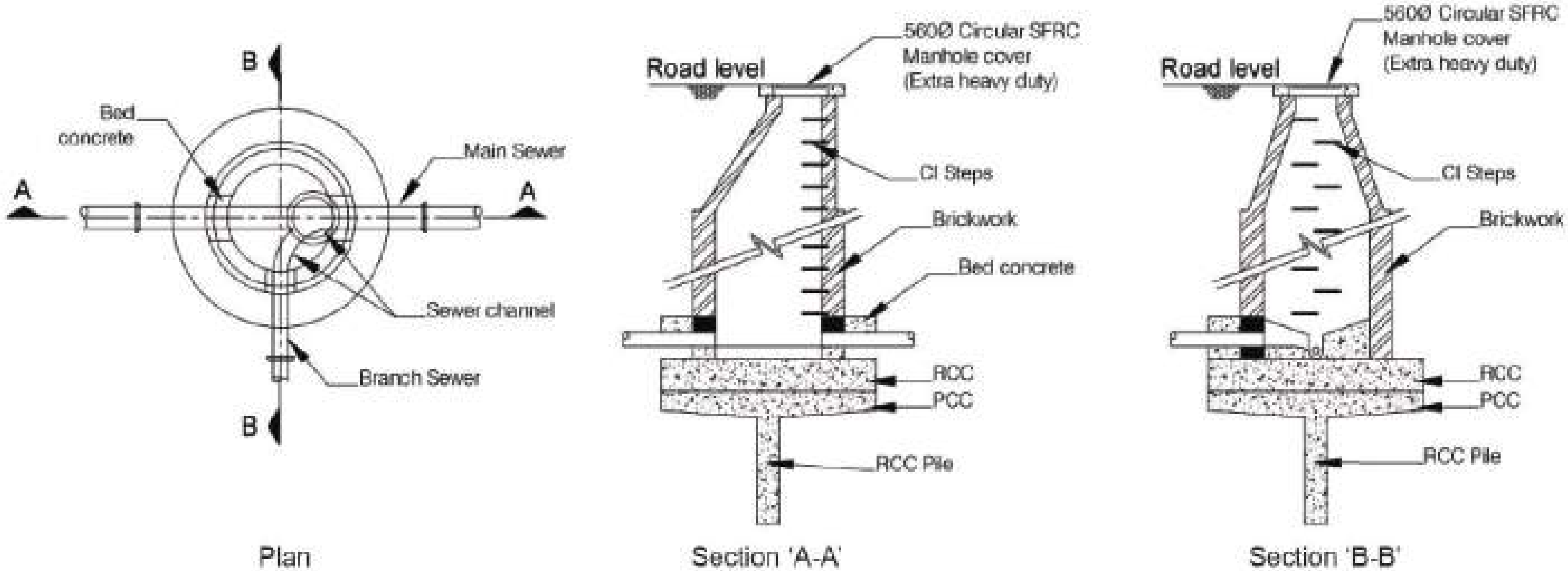
Where the depth of the manhole exceeds 90 cm below the surface of the ground, rungs shall be built into the brickwork. The vertical distance between the two consecutive rungs shall not be more than 30 cm and the centre-to-centre horizontal distance between alternate rungs shall not exceed 38 cm. The rungs shall have a width of 15 cm.

3.36 PILE SUPPORTS IN LOOSE SOILS

Where the soil is weak, RCC piles shall be driven to hard stratum and the pile cap made of the same size of the PCC of the manhole and after pouring the RCC for the pile and capping slab, immediately the RCC for foundation of the manhole shall also be poured and integrally cured. Tremie pipe shall be used along with bentonite lining as the case may be. The typical diagram for the RCC pile support is in Figure 3-30 overleaf.

3.37 MANHOLE REHABILITATION

While preparing DPR for augmentation of sewerage in already existing sewered habitations, it is necessary to look into the needs of rehabilitation of the old system and include the appropriate financial provisions.



Pile will be of RCC. Driving through soil water will need bentonite casing and pouring using tremmie pipe. Sulphate resistant cement is best used here. Pile should be driven to hard strata irrespective of depth. RCC and PCC to be poured intergrally and concrted using sulphate resistant cement

Source: CPHEEO, 1993

Figure 3.30 Illustrative arrangement of manholes in loose soils or slushy soils or quick sand

With sewer systems in our country dating back to as old as 75 years and all manholes being of brickwork, there is a need to look into the manhole rehabilitation contingencies. The following approach is recommended.

a) Institute an ultrasonic survey of the structural integrity of all manholes known to be more than 30 years, the accepted life cycle of civil works and maintain an annual repeat record, which will indicate the manholes requiring immediate attention.

b) Isolate the manhole from service by plugging the sewers with inflatable special balloons and transfer pumping from upstream manhole to downstream manhole using submersible pump sets in the upstream manhole; prepare the surface by cleaning it and removing loose particles.

c) Adopt the lining of the insides by the commercially available fast-curing elastomeric/other material that can be directly applied to existing concrete or brickwork using spray techniques whereby a homogeneous, non-porous and monolithic lining is formed. This can provide effective surface protection against wear, corrosion and water infiltration.

d) This will mark a new era in sustaining the infrastructures created in sewer manholes and forestall major issues when the rehabilitation needs arise suddenly.

e) Recent technologies provide for spray lining of the manholes without man entry. A polymeric/elastomer material like polyurea is obtained as a powder and a solution is made at site and is pumped through a vertical guide pipe in the central axis with a spray nozzle at its base and rotating full 360 degrees in plan. The thickness of lining is controlled by the rate of solution pumped, the revolutions per minute and the rate of rise of the guide pipe. There is no need to block the entry and exit sewers, as the spray entering these under pressure will line these pipes also to a certain length as well. An illustration is shown in Figure 3.31.

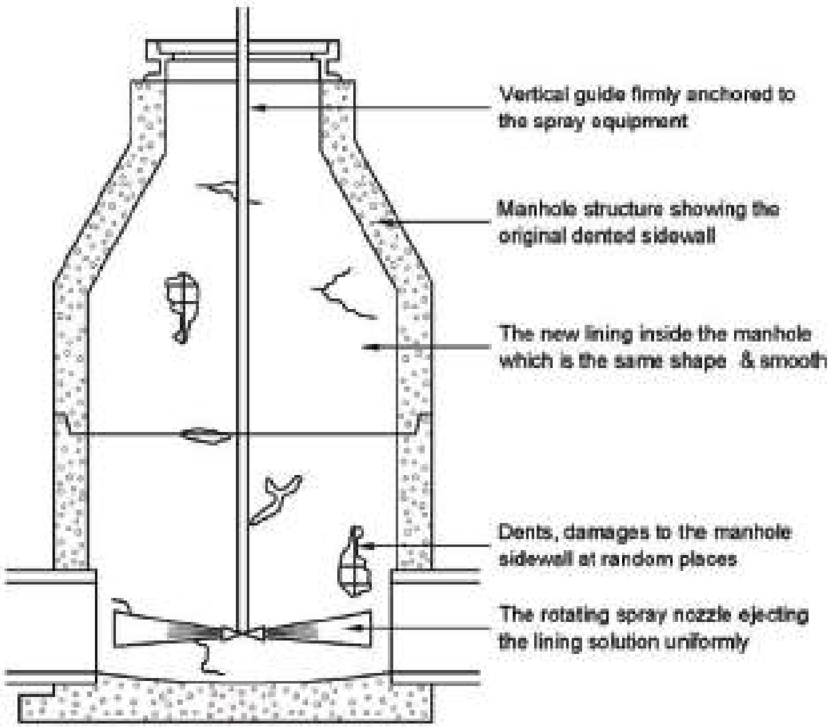


Figure 3.31 Depiction on in-situ sewer manhole lining (Spray equipment not shown)

PART - 5 LAYING, JOINTING AND CONSTRUCTION OF SEWERS

3.38 STAGING OF SEWERAGE WORKS

Due to enormous scope of sewerage work, it is desirable to set up priorities for taking up the works of different component for execution. It is generally noticed that most of sewerage schemes are not completed for want of funds, land, as well as due to public litigation and execution of work in improper sequence. The partially executed schemes could not be made functional. Therefore, the priorities of works shall be followed during execution in sequence as shown below.

(1) Sewage treatment plants

(2) Trunk mains

(3) Sewage pumping stations (if required)

(4) Main sewers

(5) Sub main sewers

(6) Sewers (Laterals)

The works at Sr. No. 1 to 3 mentioned above can be taken simultaneously. However, only after completion of all works from Sr. 1 to 6 mentioned above, the property connections shall be given. In case, part of main sewer or sub main sewer is not laid for want of land acquisition issues or any public litigation, the work of sewer lines joining that particular sub main/main sewer shall be postponed. Following such priorities, the executed works could be put into use, thus the expenditure made on structures shall not be proved unfruitful.

3.39 SEWER CONSTRUCTION

In sewer construction work, two operations are of special importance, namely, excavation of trenches, and laying of sewer pipes in trenches and tunnels. Most of the trench work involves open cut excavation; and in urban areas, it includes:

1) Removing pavement

2) Removal of the material from the ground, and its separation, its classification where necessary, and its final disposal

3) Sheeting and bracing the sides of the trench

4) Removal of water (if any) from the trench

5) Protection of other structures, both underground and on the surface, whose foundations may be affected

6) Backfilling, and

7) Replacement of the pavement

The most common type of sewer construction practice involves the use of open trenches and prefabricated pipes. However, larger sewer systems, and unusual situations may require tunnelling, jacking of pipes through the soil, or cast-in-situ concrete sewers.

On all excavation work, safety precautions for the protection of life and property are essential; and measures to avoid inconveniences to the public are desirable. Such measures and precautions include the erection and maintenance of signs (to forewarn public), barricades, bridges and detours; placing and maintenance of lights both for illumination and also as danger signals; provision of watchmen to exclude unauthorised persons, particularly children, from trespassing on the work; and such other precautions as local conditions may dictate.

(i) Each pipe section should be uncracked.

(ii) Proper placement (i.e., bedding) has to be there for each pipe section that is laid.

(iii) There should be proper joining of pipe sections.

(iv) There should be proper alignment (direction and longitudinal slope) of the line.

(v) Pipes should be covered properly with clean fill material (backfilled).

The structural design of a sewer is based on the relationship: the supporting strength of the sewer as installed divided by a suitable factor of safety which must equal or exceed the load imposed on it by the weight of earth and any superimposed loads.

The essential steps in the design and construction of buried sewers or conduits to provide safe installations are therefore:

i) Determination of the maximum load that will be applied to the pipe based on the trench and backfill conditions and the live loads to be encountered

ii) Computation of the safe load carrying capacity of the pipe when installed and bedded in the manner to be specified using a suitable factor of safety and making certain the design supporting strength thus obtained is greater than the maximum load to be applied

iii) Specifying the maximum trench widths, the type of pipe bedding and the manner in which the backfill is to be made in accordance with the conditions used in the design

iv) Checking each pipe for structural defects before installation and making sure that only sound pipes are installed and

v) Ensuring by adequate inspection and engineering supervision that all trench widths, sub grade work, bedding, pipe laying and backfilling are in accordance with design assumptions as set forth in the project specifications.

Proper design and adequate specifications alone are not enough to ensure protection from dangerous or destructive overloading of pipe. Effective value of these depends on the degree to which the design assumptions are realized in actual construction. For this reason, thorough and competent inspection is necessary to ensure that the installation conforms to the design requirements and specifications.

3.40 TYPE OF LOADS

In a buried sewer, stresses are induced by external loads and by internal pressure in case of a pressure main. The stress due to external loads is of utmost importance and may be the only one considered in the design. Besides, if the sewer is exposed to sunlight, temperature stresses induced may be considerable and these will have to be taken into consideration particularly in case of metallic pipes. The external loads are of two categories viz., load due to backfill material known as backfill load and superimposed load, which again is of two types viz. concentrated load and distributed load. Moving loads may be considered as equivalent to uniformly distributed load. Sewer lines are mostly constructed of stoneware, concrete or cast iron, which are considered as rigid pipes (while steel pipes, if used, are not considered as rigid pipes). The flexibility of the pipe affects the load imposed on the pipe and the stresses induced in it.

3.41 LOADS ON CONDUIT DUE TO BACKFILL

Methods for determining the vertical load on buried conduits due to gravity earth forces in all commonly encountered conditions, as developed by A. Marston, are generally accepted as the most suitable and reliable for computation. Theoretically stated, the load on a buried conduit is equal to the weight of the prism of earth directly over the conduit, called the interior prism of earth plus or minus the frictional shearing forces transferred to the prism by the adjacent prism of earth.

The considerations are:

a) The calculated load due to the backfill is the load that will develop when ultimate settlement has taken place.

b) The magnitude of the lateral pressure causing the shearing force is computed by Rankine's theory.

c) There is negligible cohesion except for tunnel conditions.

The general form of Marston's formula is

$$W = C\,wB^2 \qquad (3.23)$$

where,

W : Vertical load in kgs per metre length acting on the conduit due to gravity earth loads

w : Unit weight of earth, kg/m^3

B : Width of trench or conduit in meters depending upon the type of installation conditions, m

C : Dimensionless co-efficient that measures the effect of

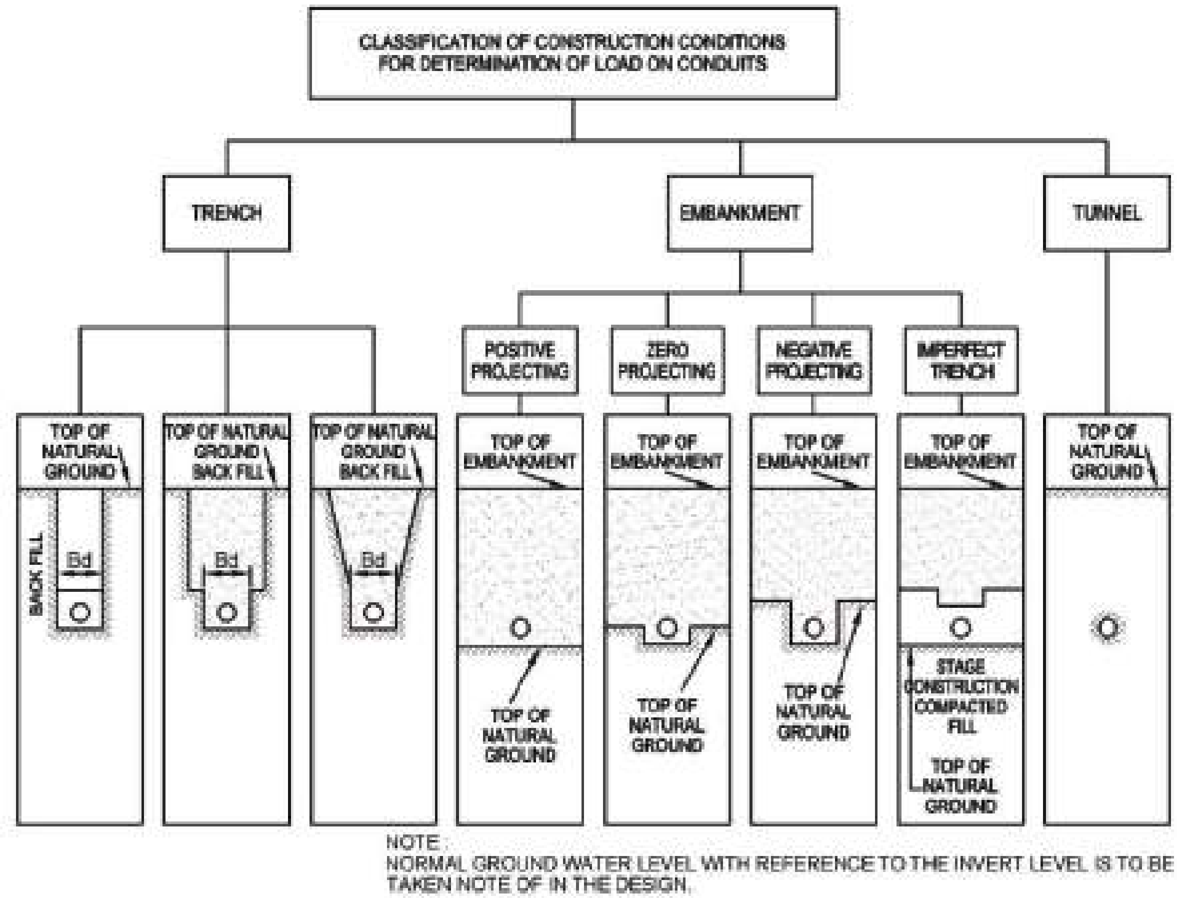
a) Ratio of height of fill to width of trench or conduit

b) Shearing forces between interior and adjacent earth prisms and

c) Direction and amount of relative settlement between interior and adjacent earth prisms for embankment conditions.

3.42 TYPES OF INSTALLATION OR CONSTRUCTION CONDITIONS

The accepted types of installation or construction conditions are shown in Figure 3.32. There are three classifications for the construction conditions:



Source: CPHEEO, 1993

Figure 3.32 Classification of construction conditions

1) Embankment condition
2) Trench condition and
3) Tunnel condition

Embankment condition prevails when the conduit is covered with fill above the original ground surface or when a trench in undisturbed ground is so wide that trench wall friction does not affect the load on the pipe. The embankment condition is further classified depending upon the position of the top of conduit in relation to the original ground surface as

i) Positive projecting condition
ii) Zero projecting condition
iii) Negative projecting condition and
iv) Imperfect trench condition

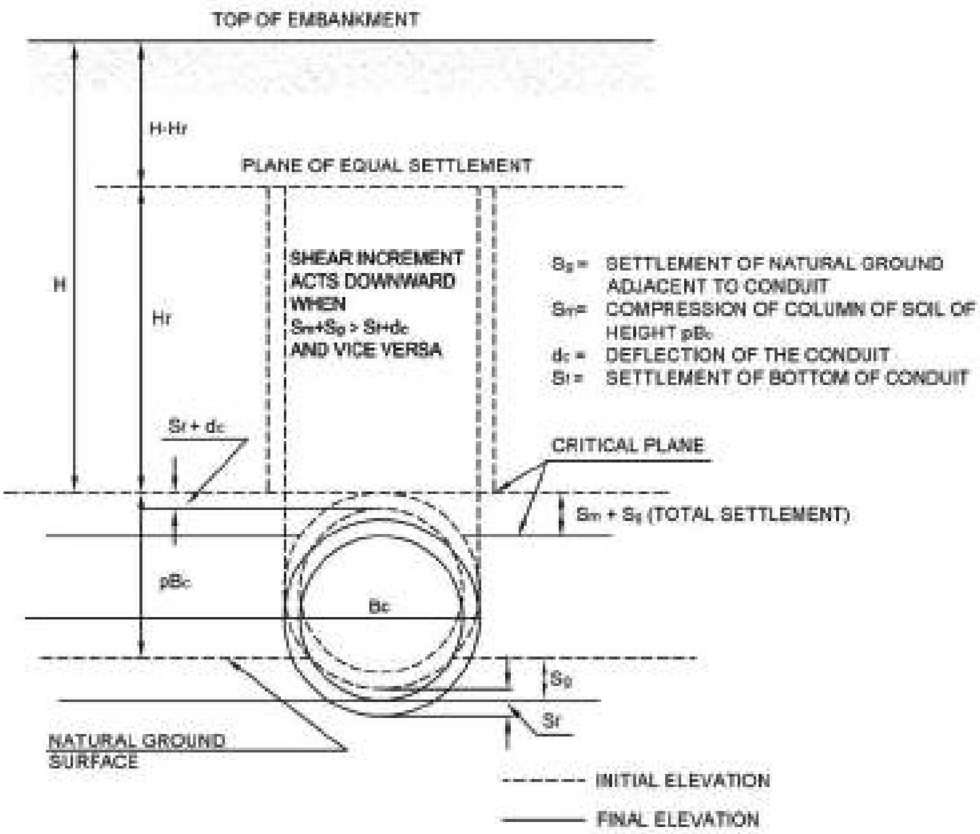
Trench condition exists when the pipe or conduit is installed in a relatively narrow trench (not wider than twice the external diameter of the pipe) cut in undisturbed soil and then covered with earth backfill up to the original ground surface. Tunnel condition exists when the sewer is placed by means of jacking or tunnelling.

3.43 LOADS FOR DIFFERENT CONDITIONS

3.43.1 Embankment or Projecting Conduit Condition

a) Positive Projecting Conduit

A conduit is said to be laid as a positive projecting conduit when the top of the conduit is projecting above the natural ground into the overlying embankment (Figure 3-33).



Source: CPHEEO, 1993

Figure 3.33 Settlements that influence loads on positive projecting conduits

i) Load Producing Forces

The load on the positive projecting conduit is equal to the weight of the prism of soil directly above the structure plus or minus vertical shearing forces, which act in a vertical plane extending upward into the embankment from the sides of the conduit. These vertical shearing forces ordinarily do not extend to the top of the embankment but terminate in a horizontal plane at some elevation above the top of the conduit known as the plane of equal settlement as shown in Figure 3.33, which also shows the elements of settlement ratio.

$$\text{Settlement ratio } r_{SD} = \frac{\text{Settlement of critical plane} - \text{settlement of top of conduit}}{\text{Compression of height of column } H \text{ of embankment}} \tag{3.24}$$

$$= \frac{(S_m + S_g) - (S_f + d_c)}{S_m}$$

where,

H : Height of top of conduit above adjacent natural ground surface (initial) or the bottom of a wide trench

p. Bc: where p is the projection ratio and Bc is the outside width of conduit

Sm : Compression column of height H of embankment

Sg : Settlement of natural ground adjacent to the conduit

Sf : Settlement of the bottom of conduit and

dc : Deflection of conduit or shortening of its vertical height under load.

When $(S_m+S_g) > (S_f + d_c)$, r_{SD} is positive, i.e., the shearing forces act downwards. Therefore, the load on conduit is equal to weight of critical prism plus shear force. When $(S_m+S_g) < (S_f + d_c)$, r_{SD} is negative and the shear force acts in the upward direction. The settlement ratio r_{SD} therefore, indicates the direction and magnitude of the relative settlement of the prism of earth directly above and adjoining the conduit. The product of r_{SD} and p gives the relative height of plane of equal settlement and hence of the magnitude of the shear component of the load. When $r_{SD} \times p = 0$, the plane of equal settlement coincides with the critical plane and there are no shearing forces and the load is equal to the weight of the central prism. It is not practicable to predetermine the rsd value. The recommended design values based on actual experience are given in Table 3.17.

Table 3.17 Recommended design values of settlement ratios

	Type of Conduit	Type of Soil	Settlement Ratio
1	Rigid	Rock or unyielding foundation	+ 1.0
2	Rigid	Ordinary foundation	+ 0.5 to + 0.8
3	Rigid	Yielding foundation	0 to + 0.5
4	Rigid	Negative projecting installation	0.3 to 0.5
5	Flexible	Poorly compacted side fill	0.4 to 0
6	Flexible	Well compacted side fill	0

Source:CPHEEO, 1993

ii) Computation of Loads

Marston’s formula for positive projecting conduits (both rigid and flexible) is mentioned overleaf.

$$W_c = C_c \; w \, B_c^{\;2} \qquad (3.25)$$

where,

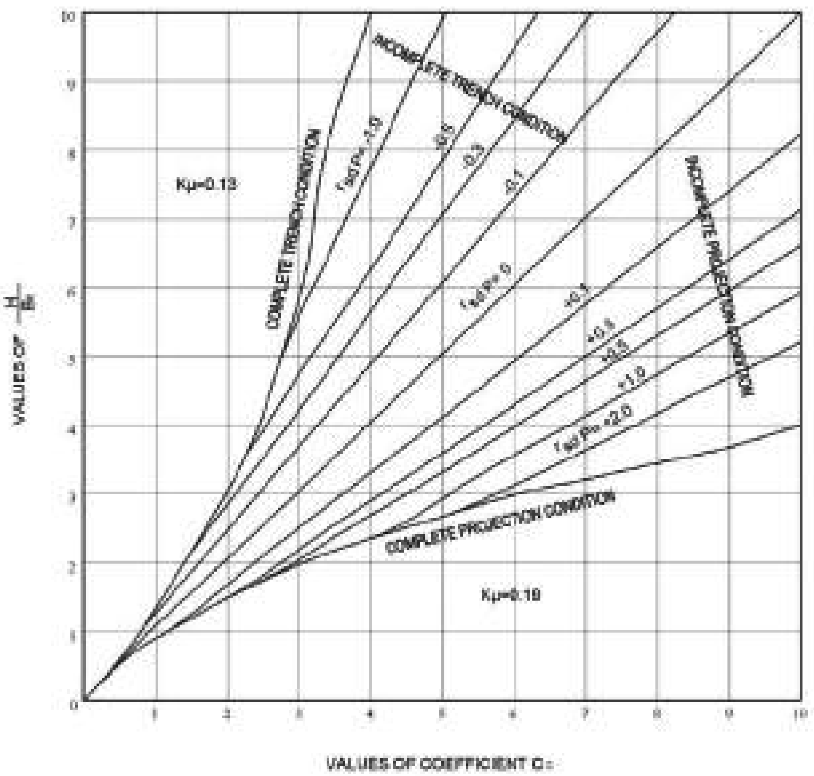
W_c : Load on conduit in kg/m

w : Unit weight of backfill material in kg/m^3

B_c : Outside width of conduit in m

C_c : Load coefficient, which is a function of the product of the projection ratio and the settlement ratio and ratio of the height of fill above the top of the conduit to the outside width of the conduit (H/B). It is also influenced by the coefficient of internal friction of the backfill material and the Rankine's ratio of lateral pressure to vertical pressure Ku. Suggested values for Ku for positive and negative settlement ratios are 0.19 and 0. 13, respectively.

The value of C_c can be obtained from Figure 3.34.

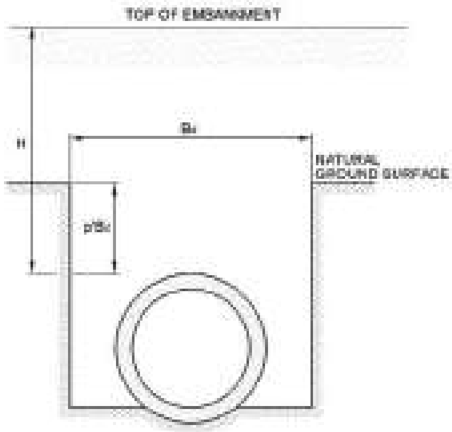


Source: CPHEEO, 1993

Figure 3.34 Diagram for coefficient Cc for positive projecting conduits

b) Negative Projecting Conduit

A conduit is said to be laid in a negative projecting condition when it is laid in a trench, which is narrow with respect to the size of pipe and shallow with respect to depth of cover. Moreover, the native material of the trench is of sufficient strength that the trench shape can be maintained dependably during the placing of the embankment, the top of the conduit being below the natural ground surface and the trench refilled with loose material and the embankment constructed above (Figure 3.35). The prism of soil above the conduit, being loose and greater in depth compared to the adjoining embankment, will settle more than the prism over the adjoining areas thus generating upward shear forces which relieve or reduce the load on the conduit.



Source: CPHEEO, 1993

Figure 3.35 Negative projecting conduit

i) Computation of Loads

Marston's formula for negative projecting conduits is given by

$$W_c = C_n \, w B_d^{\,2} \tag{3.26}$$

where,

W_c : Load on the conduit in kg/m

B*d* : Width of trench in m

w : Unit weight of soil in kg/m^3

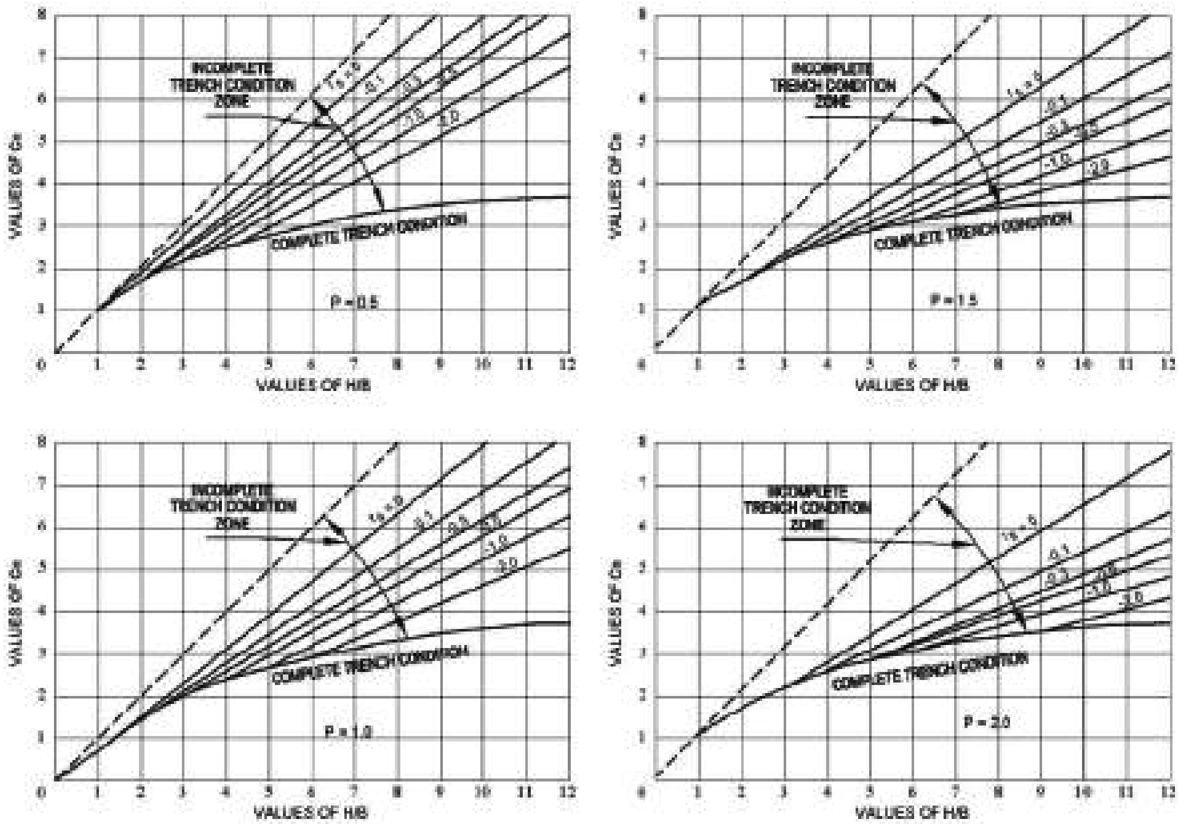
C_n : Load coefficient, which is a function of the ratio (H/Bd) of the height of fill and the width of trench equal to the projection ratio p (Vertical distance from the firm ground surface down to the top of the conduit/width of the trench) and the settlement ratio rsd given by the expression,

$$r_{sd} = \frac{\text{settlement of natural ground} - \text{settlement of critical plane}}{\text{compression of the backfill within the height } pB_d} \tag{3.27}$$

$$= \frac{S_g - (S_d + S_f + d_c)}{S_d}$$

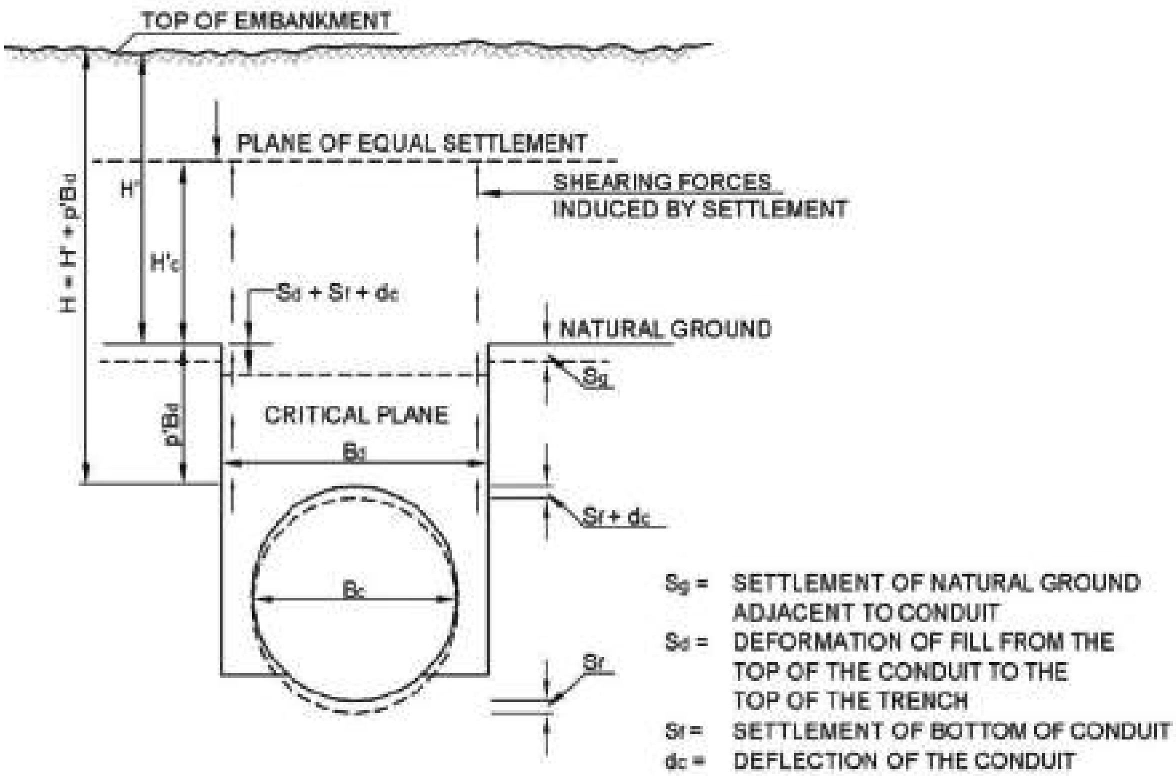
Values of C_n, for various values of H/B_d, r_{sd}, and p are given in Figure 3.36 overleaf.

Exact determination of the settlement ratio is very difficult. Recommended value of r_{sd} is 0.3 for design purposes. Elements of settlement ratios are shown in Figure 3.37 overleaf.



Source: CPHEEO, 1993

Figure 3.36 Coefficient Cn for negative projecting conduits and imperfect trench conditions

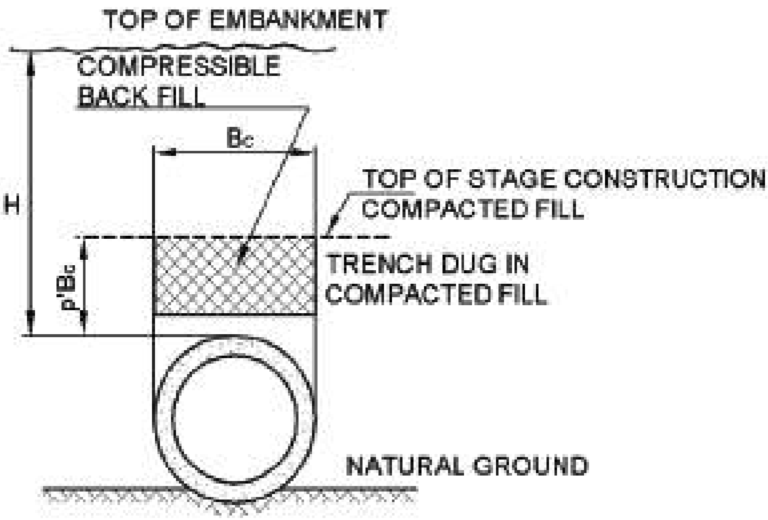


Source: CPHEEO, 1993

Figure 3.37 Settlements that influence loads on negative projecting conduits

c) Imperfect Trench Conduits

An imperfect trench conduit is employed to minimize the load on a conduit under embankments of unusual heights. The conduit is first installed as a positive projecting conduit. The embankment is then built up to some height above the top and thoroughly compacted as it is placed. A trench of the same width as the conduit is excavated directly over it down to or near its top. This trench is refilled with loose compressible material and the balance of the embankment completed in a normal manner (Figure 3.38).



Source: CPHEEO, 1993

Figure 3.38 Imperfect trench conditions

The Marston's formula for this installation condition is again given by

$$W_c = C_n \ w \ B_c^{\ 2} \tag{3.28}$$

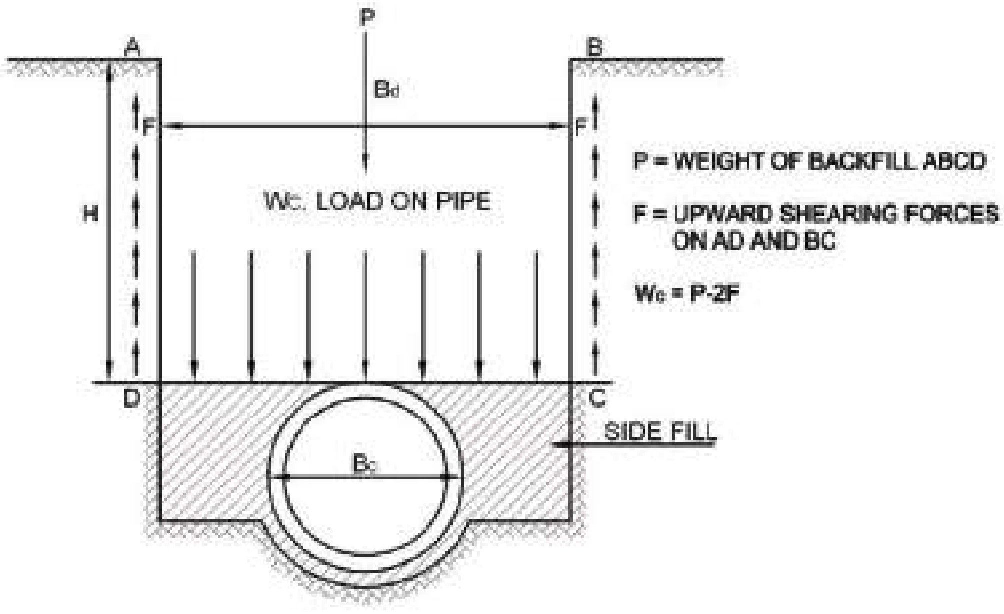
The values of Cn in this case also may be obtained from Figure 3.36 for negative projecting conduits taking Bc= Bd on the assumption that the trench fill is no wider than the pipe.

3.44 TRENCH CONDITION

Generally, sewers are laid in ditches or trenches by excavation in natural or undisturbed soil and then covered by refilling the trench to the original ground level.

a) Load Producing Forces

The vertical dead load to which a conduit is subjected under trench conditions is the resultant of two major forces. The first component is the weight of the prism of soil within the trench and above the top of the pipe and the second is due to the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench produced by settlement of backfill. The resultant load on the horizontal plane at the top of the pipe within the trench is equal to the weight of the backfill minus these upward shearing forces as shown in Figure 3.39 overleaf.



Source: CPHEEO, 1993

Figure 3.39 Load producing forces

b) Computation of Loads

The load on rigid conduits in trench condition is given by the Marston's formula in the form

$$W_c = C_d \; w \, B_d^{\;2} \tag{3.29}$$

where,

- W_c : Load on the pipe in kg per linear meter
- w : Unit weight of backfill soil in kg/m^3
- B_d : Width of trench at the top of the pipe in m and
- C_d : Load coefficient which is a function of a ratio of height of fill to width of trench (H/B_d) and of the friction coefficient between the backfill and the sides of the trench.

Weights of common filling materials (w) and values of C_d for common soil conditions encountered are given in Table 3.18 and Table 3.19, respectively.

Table 3.18 Weights of common filling material

Materials	Weight (kg/m^3)	Materials	Weight (kg/m^3)
Dry Sand	1,600	Saturated Clay	2,080
Ordinary (Damp Sand)	1,840	Saturated Top Soil	1,840
Wet Sand	1,920	Sand and Damp Soil	1,600
Damp Clay	1,920		

Source: CPHEEO, 1993

Table 3-19 Values of C_d for calculating loads on pipes in trenches

Ratio H/B_d	Safe working Values of C_d				
	Minimum possible without cohesion**	Maximum for Ordinary Sand***	Completely Top Soil	Ordinary maximum for clay****	Extreme maximum for clay*****
0.5	0.455	0.461	0.464	0.469	0.474
1.0	0.830	0.852	0.864	0.881	0.898
1.5	1.140	1.183	1.208	1.242	1.278
2.0	1.395	1.464	1.504	1.560	1.618
2.5	1.606	1.702	1.764	1.838	1.923
3.0	1.780	1.904	1.978	2.083	2.196
3.5	1.923	2.075	2.167	2.298	2.441
4.0	2.041	2.221	2.329	2.487	2.660
4.5	2.136	2.344	2.469	2.650	2.856
5.0	2.219	2.448	2.590	2.798	3.032
5.5	2.286	2.537	2.693	2.926	3.190
6.0	2.340	2.612	2.782	3.038	3.331
6.5	2.386	2.675	2.859	3.137	3.458
7.0	2.423	2.729	2.925	3.223	3.571
7.5	2.454	2.775	2.982	3.299	3.673
8.0	2.479	2.814	3.031	3.366	3.764
8.5	2.500	2.847	3.073	3.424	3.845
9.0	2.518	2.875	3.109	3.476	3.918
9.5	2.532	2.898	3.141	3.521	3.983
10.0	2.543	2.918	3.167	3.560	4.042
11.0	2.561	2.950	3.210	3.626	4.141
12.0	2.573	2.972	3.242	3.676	4.221
13.0	2.581	2.989	3.266	3.715	4.285
14.0	2.587	3.000	3.283	3.745	4.336
15.0	2.591	3.009	3.296	3.768	4.378
Very Great	2.599	3.030	3.333	3.846	4.548

W_c = load on pipe in kg per linear meter
C_d = Coefficient
w = Weight of trench filling material in kg/m^3
B_d = Width of trench a little below the top of the pipe in meters
* Ratio of height of fill above top of pipe to width of trench a little below the top of the pipe.
** These values give the loads generally imposed by granular filling materials before tamping or settling.
*** Use these values as safe for all ordinary cases of sand filling.
**** Thoroughly wet. Use these values as safe for all ordinary cases of clay filling.
***** Completely saturated. Use these values only for extremely unfavourable conditions.
Source: CPHEEO, 1993

Equation (3.29) gives the total vertical load due to backfill in the horizontal plane at the top of the conduit as shown in Figure 3.39 if the pipe is rigid. For flexible conduits, the formula may be modified as

$$W_c = C_d \; w B_c \; B_d \tag{3.30}$$

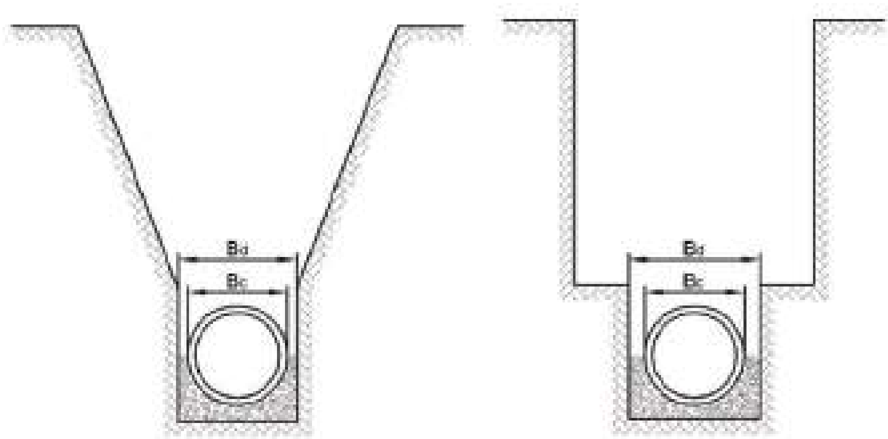
where,

B_c : Outside width of the conduit in m

c) Influence of Width of Trench

It has been experimentally seen that when the width of the trench excavated is not more than twice the external width of the conduit, the assumption made in the trench condition of loading holds good. If the width of the trench goes beyond three times the outside dimension of the conduit, it is necessary to apply the embankment condition of loading. In the transition width from $B_d = 2B_c$ to $B_d = 3B_c$ computation of load by both the procedures will give the same results.

In case of excavations with sloping sides (possible in undeveloped areas), the provision of a sub-trench (Figure 3.40) minimizes the load on the pipe by reducing the value of B_d.



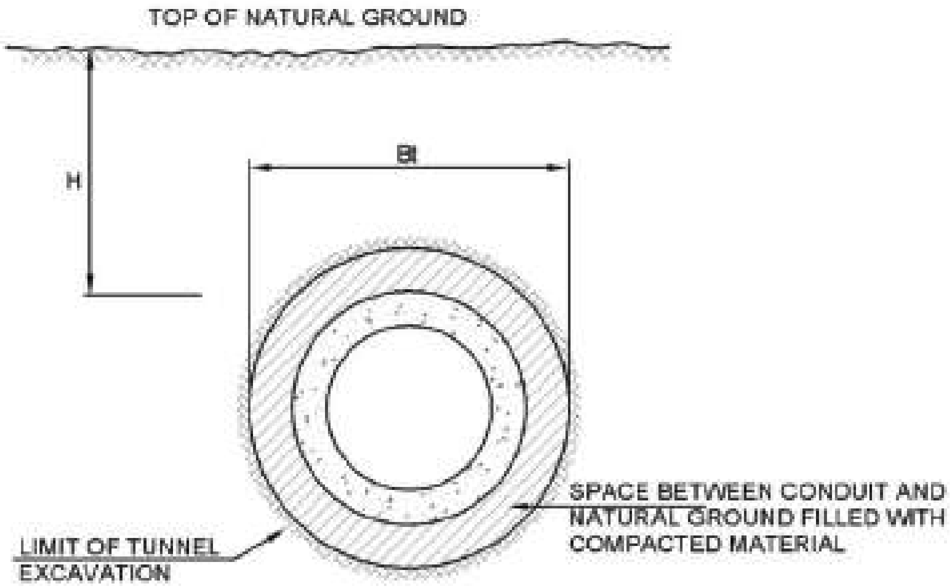
Source: CPHEEO, 1993

Figure 3.40 Examples of sub trench

3.45 TUNNEL CONDITION

When the conduit is laid more than 9 m to 12 m deep or when the surface obstructions are such that it is difficult to construct the pipeline by the conventional procedure of excavation and backfilling, it may be more economical to place the conduit by means of tunnelling. The general method in this case is to excavate the tunnel, to support the earth by suitable means and then to lay the conduit. The space between the conduit and the tunnel is finally filled up with compacted earth or concrete grout as indicated in Figure 3.41 overleaf.

If the length of tunnel is short say 6 m to 10 m, the entire circular section can be constructed as one unit. For longer tunnels, construction may be in segments, with refilling proceeding simultaneously.



Source: CPHEEO, 1993

Figure 3.41 Conduit in tunnel

a) Load Producing Forces

The vertical load acting on the tunnel supports and eventually the pipe in the tunnel is the resultant of two major forces viz., the weight of the overhead prism of soil within the width of the tunnel excavation and the shearing forces generated between the interior prisms and the adjacent material due to friction and cohesion of the soil.

b) Load Computations

Marston's formula to be used in this case of installation of conduit is given by:

$$W_t = C_t \, B_t \left(wB_t - 2C \right) \tag{3.31}$$

where,

W_t : Load on the pipe or tunnel support in kg/m

w : Unit weight of soil above the tunnel in kg/m^3

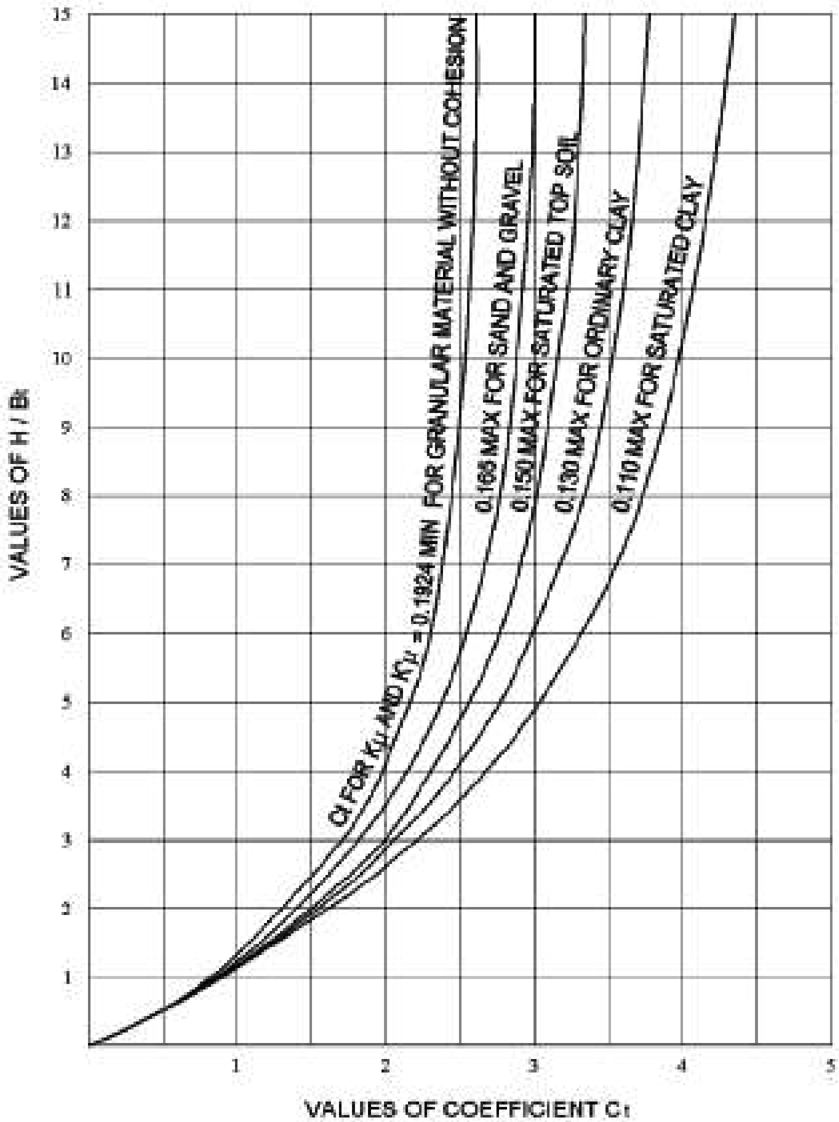
B_t : Maximum width of the funnel excavation in m

C : Coefficient of cohesion in kg/m^2 and

C_t : Load coefficient which is a function of the ratio (H/Bt) of the distance from the ground surface to the top of the tunnel to the maximum width of tunnel excavation and of the coefficient of internal friction of the material of the tunnel.

When the coefficient of cohesion is zero, the formula reduces to the same form as in trench condition equation (3.29).

Value of C for various values of H/Bt and different soil conditions are to be obtained from Figure 3-42.



Source: CPHEEO, 1993

Figure 3.42 Diagram for coefficient C_t for tunnels in undisturbed soil

Recommended values of coefficient of cohesion for different types of soils are given in Table 3.20.

Table 3.20 Cohesion coefficients for different soils

Type of Soil	kg/m^2	Type of Soil	kg/m^2
Soft Clay	200	Silty Sand	500
Medium Clay	1,200	Dense Sand	1,400
Hard Clay	4,700	Saturated Top Soil	500
Loose Dry Sand	0		

Source: CPHEEO, 1993

3.46 EFFECT OF SUBMERGENCE

Sewers may be laid in trenches or under embankment in areas that may be temporarily or permanently submerged in water. The fill load, in such cases, will be reduced and correspond to the buoyant weight of the fill material. However, effect of submergence could be ignored which provides an additional factor of safety, but it may be necessary to check whether a pipe is subject to flotation. Under submergence, the minimum height of the fill material that will be required to prevent flotation ignoring the frictional forces in the fill can be determined from the equation:

$$H_{min}\ B_c\ (w_s - w_o) + W_c = (\pi / 4)\ B_c^{\ 2}\ w_o \qquad (3.32)$$

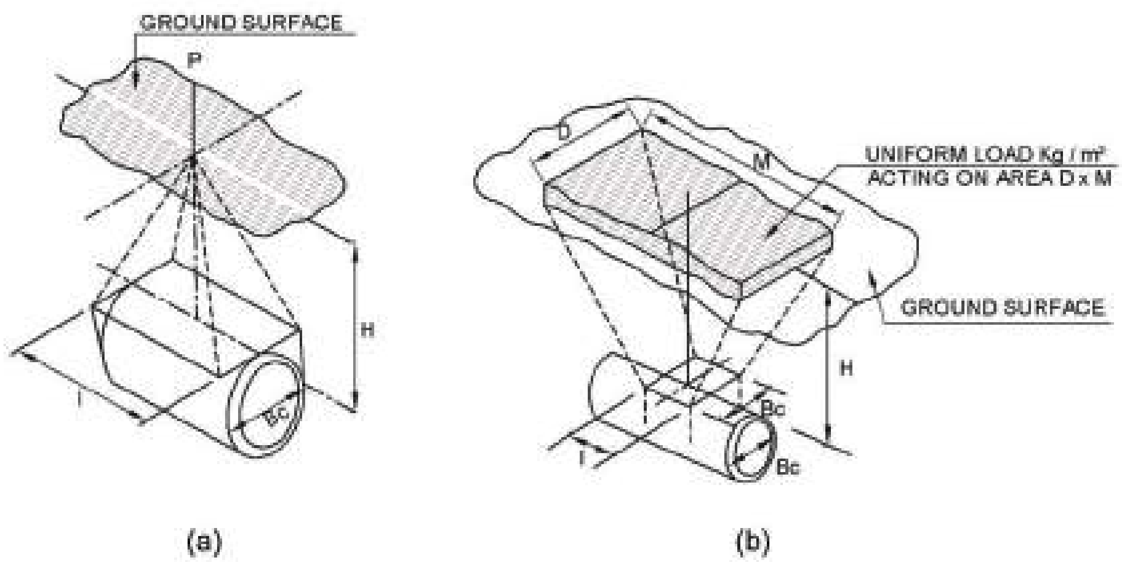
where,

- H_{min} : Minimum height of fill material in m
- w_s : Saturated density of the soil in kg/m^3
- w_0 : Density of water in kg/m^3
- W_c : Unit weight of the empty pipe in kg/linear metre and
- B_c : Outside width of the conduit in m.

Wherever sufficient height of fill material is not available, anti-flotation blocks should be provided. (As shown in Example IX in Appendix A.3.8)

3.46.1 Load on Conduit due to Superimposed Loads

The types of superimposed loads, which are generally encountered in buried conduits may be categorized as (a) concentrated load and (b) distributed load. These are explained diagrammatically in Figure 3.43.



Source:CPHEEO, 1993

Figure 3.43 (a) Concentrated superimposed load vertically centred over conduit (left)
(b) Distributed superimposed load vertically centred over conduit (right)

3.47 CONCENTRATED LOADS

The formula for load due to superimposed concentrated load such as a truck wheel (Figure 3.43) is given in the following form by Holl's integration of Boussinesq's formula

$$W_{sc} = C_s \left(PF / L \right) \tag{3.33}$$

where,

- W_{sc} : Load on the conduit in kg/m
- P : Concentrated load in kg acting on the surface
- F : Impact factor (1.0 for air field runways, 1.5 for highway traffic and air field taxi ways, 1.75 for railway traffic) and
- C_s : Load coefficient which is a function of

$$\frac{B_c}{2H} \quad and \quad \frac{L}{2H} \tag{3.34}$$

where,

- H : Height of the top of the conduit to ground surface in m
- B_c : Outside width of conduit in m, and
- L : Effective length of the conduit to which the load is transmitted in m.

Values of Cs for various values of ($B_c/2H$) and ($L/2H$) are obtained from Table 3.20

The effective length of the conduit is defined as the length over which the average load due to surface traffic units produces the same stress in the conduit wall, as does the actual load, which varies in intensity from point to point. This is generally taken as 1 m or the actual length of the conduit if it is less than 1 m.

3.48 DISTRIBUTED LOAD

For the case of distributed superimposed loads, the formula for load on conduit is given by

$$W_{sd} = C_s \ p \ F \ B_c \tag{3.35}$$

where,

- W_{sd} : Load on the conduit in kg/m
- P : Intensity of the distributed load in kg/m^2
- F : Impact factor
- B_c : Width of the conduit in m
- C_s : Load coefficient, a function of $D/2H$ and $L/2H$ from Table 3.21
- H : Height of the top of conduit to the ground surface in m
- D, L : Width and length in m respectively of the area over which the distributed load acts

Table 3.21 Values of load coefficients, Cs for concentrated and distributed superimposed loads vertically centred over conduits

$\frac{D}{2H}$ or $\frac{B_c}{2H}$	$\frac{M}{2H}$ or $\frac{L}{2H}$													
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.766	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

Source: CPHEEO, 1993

For class AA IRC loading, in the critical case of wheel load of 6.25 tones, the intensity of distributed load with wheel area 300mm × 150mm is given by

$$P = \frac{6.25}{0.3 \times 0.15} \quad in \quad T/m^2$$

3.49 CONDUITS UNDER RAILWAY TRACK

The load on conduits under railway track is given by

$$W = 4\,C_s\,U\,B_c \tag{3.36}$$

where,

U: Uniformly distributed load in tonnes / m2 from the surface directly over the conduit and equal to

$$U = \frac{PF + 2W_t B}{4AB} = \frac{PF}{4AB} + \frac{W_t}{2A} \tag{3.37}$$

where,

P : Axle load in tonnes (22.5 tonnes for Broad gauge)

F : Impact factor for railroad =1.75

2A : Length of the sleeper in m (2.7 m for Broad gauge)

2B : Distance between the two axles (1.84 m for broad gauge)

W_t : Weight of the track structure in tones/m (0.3 tonnes/m for broad gauge)

C_s : Load coefficient which depends on the height of the top of sleeper from the top of the conduit

B_c : Width of the conduit in m

For broad gauge track, the formula will reduce to:

$$W = 32.14\,C_s\,B_c \tag{3.38}$$

3.50 SUPPORTING STRENGTH OF RIGID CONDUIT

The ability of a conduit to resist safely the calculated earth load depends not only on its inherent strength but also on the distribution of the vertical load and bedding reaction and on the lateral pressure acting against the sides of the conduit. The inherent strength of a rigid conduit is usually expressed in terms of the three edge bearing test results, the conditions of which are, however, different from the field load conditions. The magnitude of the supporting strength of a pipe as installed in the field is dependent upon the distribution of the vertical load and the reaction against the bottom of the pipe. It also depends on the magnitude and distribution of the lateral pressure acting on the sides of the pipe.

3.50.1 Laboratory Test Strength

All rigid pipes may be tested for strength in the laboratory by the three-edge-bearing test (ultimate load). Methods of test and minimum strength for concrete (unreinforced and reinforced) stoneware and AC pipes and other details are given in Appendix A.3.9.

3.50.2 Field Supporting Strength

The field supporting strength of a rigid conduit is the maximum load per unit length, which the pipe will support while retaining complete serviceability when installed under specified conditions of bedding and backfilling. The field supporting strength however does not include any factor of safety. The ratio of the strength of a pipe under any stated condition of loading and bedding to its strength measured by the three-edge-bearing test is called the load factor.

The load factor does not contain a factor of safety. Load factors have been determined experimentally and analytically for the commonly used construction condition for both trench and embankment conduits. The basic design relationship between the different design elements is the safe supporting strength (W),

$$\begin{aligned} W &= \text{Field supporting strength / Factor of safety} \\ &= \text{Load factor} \times \text{three edge bearing strength / Factor of safety} \end{aligned} \tag{3.39}$$

A factor of safety of at least 1.5 should be applied to the specified minimum three-edge-bearing strength to determine the working strength for all the rigid conduits.

3.50.3 Protection and Bedding of Sewers

3.50.3.1 Guidelines

The factor of safety recommended for concrete pipes for sewers is ‘1.5’, which is considerably less as compared to that for most engineering structures which have a factor of safety of at least 2.5. As the margin of safety against the ultimate failure is low, it becomes imperative to guarantee that the loads imposed on sewer pipes are not greater than the design loads for the given installation conditions. In order to achieve this objective the following procedures are recommended:

1. Minimum width of the trench should be specified in consonance with the requirements of adequate working space to allow access to all parts and joints of pipes.

2. Specification should lay proper emphasis on the limit of the width of trench to be adopted in the field, which should not exceed that adopted in the design calculations. Any deviations from this requirement during the construction should be investigated for their possible effect on the load coming on the pipe and steps should be taken to improve the safe supporting strength of pipe for this condition of loading by adopting suitable bedding or such other methods when necessary.

3. The field engineer should keep in touch with the design engineer throughout the duration of the project and any deviation from the design assumptions due to the exigencies of work, should be immediately investigated and corrective measures taken in time.

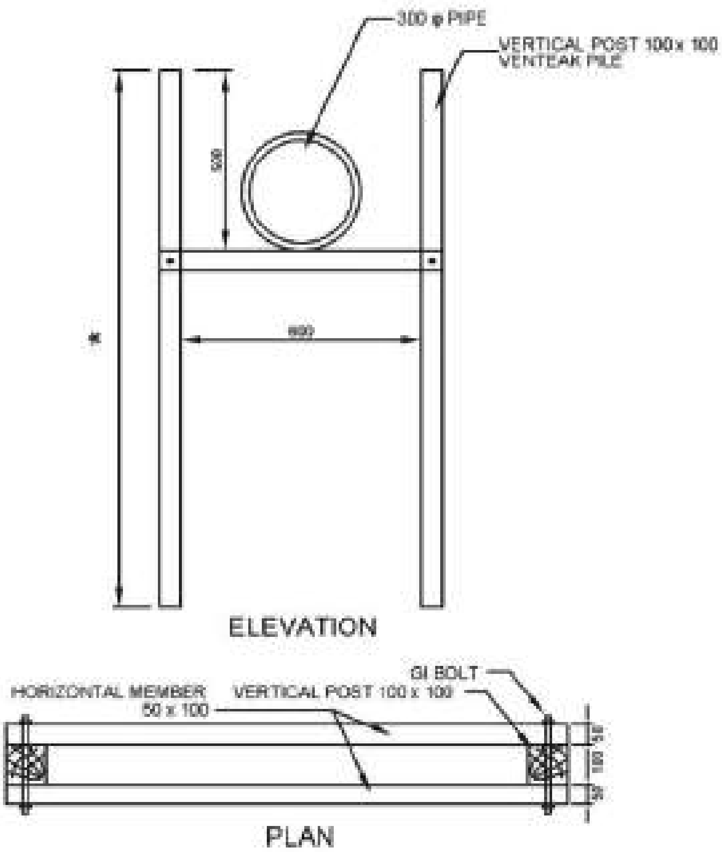
4. All pipes used on the work should be tested as per the IS specifications and test certificates of the manufacturers should be furnished for every consignment brought to the site.

5. Whenever shoring is used, the pulling out of planks on completion of work should be carried out in stages and this should be properly supervised to ensure that the space occupied by the planks is properly backfilled.

6. Proper backfilling methods both as regards to selection of materials, methods of placing and proper compaction should be in general agreement with the design assumptions.

3.50.3.2 Bedding in Quicksand Soil Conditions

In quick sand conditions, it is necessary to anchor the sewer to the ground and hold it at the grade as laid in the face of soil sinkage. This is done by using the Venteak piles, which are driven on both sides of the sewer into the soil right up to hard strata and connecting the two by a cross beam at the soffit of the sewer. Then the sewer is tied securely to the cross beam by a 8 mm thick nylon rope in two rounds and singeing the ends of the rope integrally to prevent slippages. An example is shown in Figure 3-44. The venteak pile cross bracing can be a single brace inserted between the piles for non-metallic smaller sewers and double bracing for metallic higher sized sewers as in Figure 3.44. A work in progress in such conditions is shown in Figure 3.45 overleaf.



(The sewer pipe should be cross-braced with the horizontal supports by means of non-biodegradable nylon rope of 8 mm multi-stranded and with multiple wraps around and the edges singed to heat weld the entire rope without loosening or unwinding)

Figure 3.44 Example of Venteak supported sewer pipe

Source: L&T Shipbuilding Limited, Kattupalli, Chennai, 2012

Figure 3.45 Typical arrangements for laying sewers in high subsoil locations using dewatering pump sets, tube wells and Venteak piles with cross brace and nylon rope wrapping around the sewers securing it to the venteak piles and brace

3.50.3.3 Type of Bedding

The type of bedding (granular, concrete cradle, full concrete encasement etc.) would depend on the soil strata and depth at which sewer is laid. The load due to backfill, superimposed load (live load) and the three-edge-bearing strength of pipe (IS: 458) are the governing criteria for selection of appropriate bedding factors.

$$\text{Bedding Factor} = \frac{\text{Design Load X Factor of Satety*}}{\text{Three Edge Bearing Strength}} \qquad (3.40)$$

* Factor of safety = 1.5

The type of bedding to be used depends on the bedding factor and the matrix of type of bedding for different diameters and different depths has been tabulated in Table 3.22 and Table 3.23.

Table 3.22 Type of bedding for sewer pipes

Bedding Factor	Type of Bedding
Up to 1.9	Class B : Granular (GRB)
1.9 - 2.8	Class Ab : Plain Concrete cradle (PCCB)
2.8 - 3.4	Class Ac : Reinforced Concrete cradle (RCCB) with 0.4 % Reinforcement
> 3.4	Class Ad : Reinforced concrete arch with 1.0% reinforcement

Table 3.23 Selection of bedding for different depths and different diameters

Diameter	Bedding type for cover depth in m				Diameter	Bedding type for cover depth in m			
mm	up to 2.5	2.5 - 3.5	3.5 - 5.0	5.0 - 6.0	mm	up to 2.5	2.5 - 3.5	3.5 - 5.0	5.0 - 6.0
400	Ab	Ab	Ab	Ac	1,400	B	Ab	Ab	Ab
500	Ab	Ab	Ab	Ab	1,500	B	Ab	Ab	Ab
600	B	Ab	Ab	Ab	1,600	B	Ab	Ab	Ab
700	B	Ab	Ab	Ab	1,800	B	Ab	Ab	Ab
750	B	Ab	Ab	Ab	2,000	B	Ab	Ab	Ab
800	B	Ab	Ab	Ab	2,200	B	Ab	Ab	Ac
900	B	Ab	Ab	Ab	2,400	B	Ab	Ab	Ac
1,000	B	Ab	Ab	Ab	2,600	B	Ab	Ab	Ac
1,200	B	Ab	Ab	Ab	2,800	B	Ab	Ab	Ac

3.50.3.4 Classes of Bedding for Trench Conditions

Four classes, A, B, C and D, of bedding used most often for pipes in trenches are illustrated in Figure 3-46 overleaf. Class A bedding may be either concrete cradle or concrete arch. Class B is bedding having a shaped bottom or compacted granular bedding with a carefully compacted backfill. Class C is an ordinary bedding having a shaped bottom or compacted granular bedding but with a lightly compacted backfill. Class D is one with flat bottom trench with no care being taken to secure compaction of backfill at the sides and immediately over the pipe and hence is not recommended. Class B or C bedding with compacted granular bedding is generally recommended. Shaped bottom is impracticable and costly and hence is not recommended.

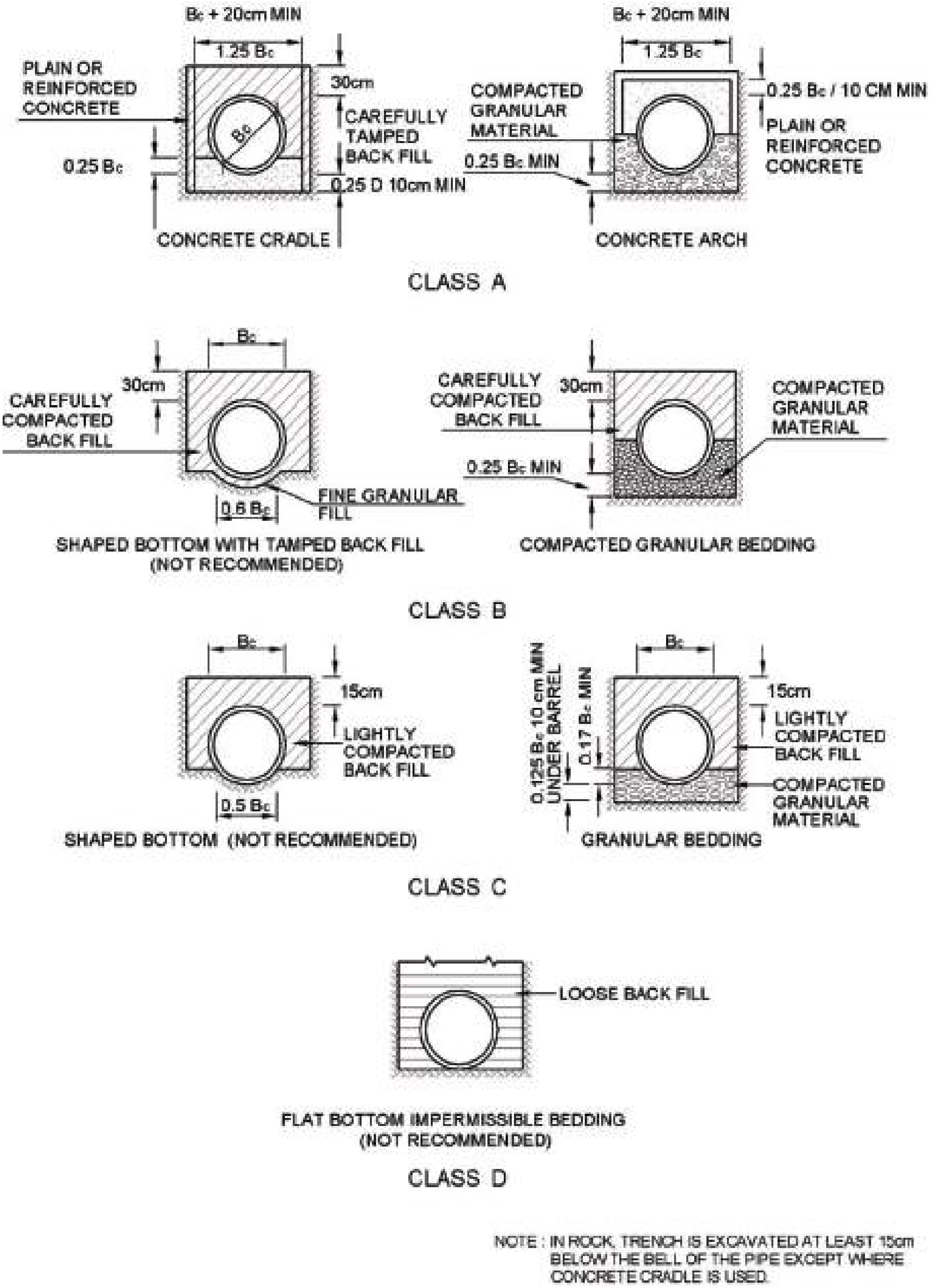
The pipe bedding materials must remain firm and not permit displacement of pipes.

The material has to be uniformly graded or well graded. Uniformly graded materials include pea gravel or one-size materials with a low percentage of over and undersized particles.

Well-graded materials containing several sizes of particles in stated proportions, ranging from a maximum to minimum size coarse sand, pea gravel, crushed gravel, crushed screenings, can be used for pipe bedding.

Fine materials or screenings are not satisfactory for stabilizing trench bottoms and are difficult to compact in a uniform manner to provide proper pipe bedding.

Well-graded material is most effective for stabilizing trench bottom and has a lesser tendency to flow than uniformly graded materials. However, uniformly graded material is easier to place and compact above sewer pipes.



Source: CPHEEO, 1993

Figure 3.46 Classes of bedding for conduit in trench

3.50.3.5 Load Factors for Bedding

The load factors for the different classes of bedding are given in Table 3-24.

Table 3.24 Load factor for different classes of bedding

Class of bedding	Condition	Load factor
Aa	Concrete cradle-plain concrete and lightly tamped backfill	2.2
Ab	Concrete cradle-plain concrete and carefully tamped backfill	2.8
Ac	Concrete cradle-RCC with P-0.4%	Up to 3.4
Ad	Arch type – plain concrete	2.8
	RCC with P = 0.4%	Up to 3.4
	RCC with P = 0.1%	Up to 4.8
	P is ratio of area of steel to area of concrete at the crown)	
B	Shaped bottom or compacted granular bedding with carefully compacted backfill	1.9
C	Shaped bottom or compacted granular bedding with lightly compacted backfill	1.5
D	Flat bottom trench	1.1

Source:CPHEEO, 1993

The granular material used must stabilize the trench bottom in addition to providing a firm and uniform support for the pipe. Well-graded crushed rock or gravel with the maximum size not exceeding 25 mm, is recommended for the purpose. Where rock or other unyielding foundation material is encountered, bedding may be according to one of the Classes A, B or C, but with the following additional requirements.

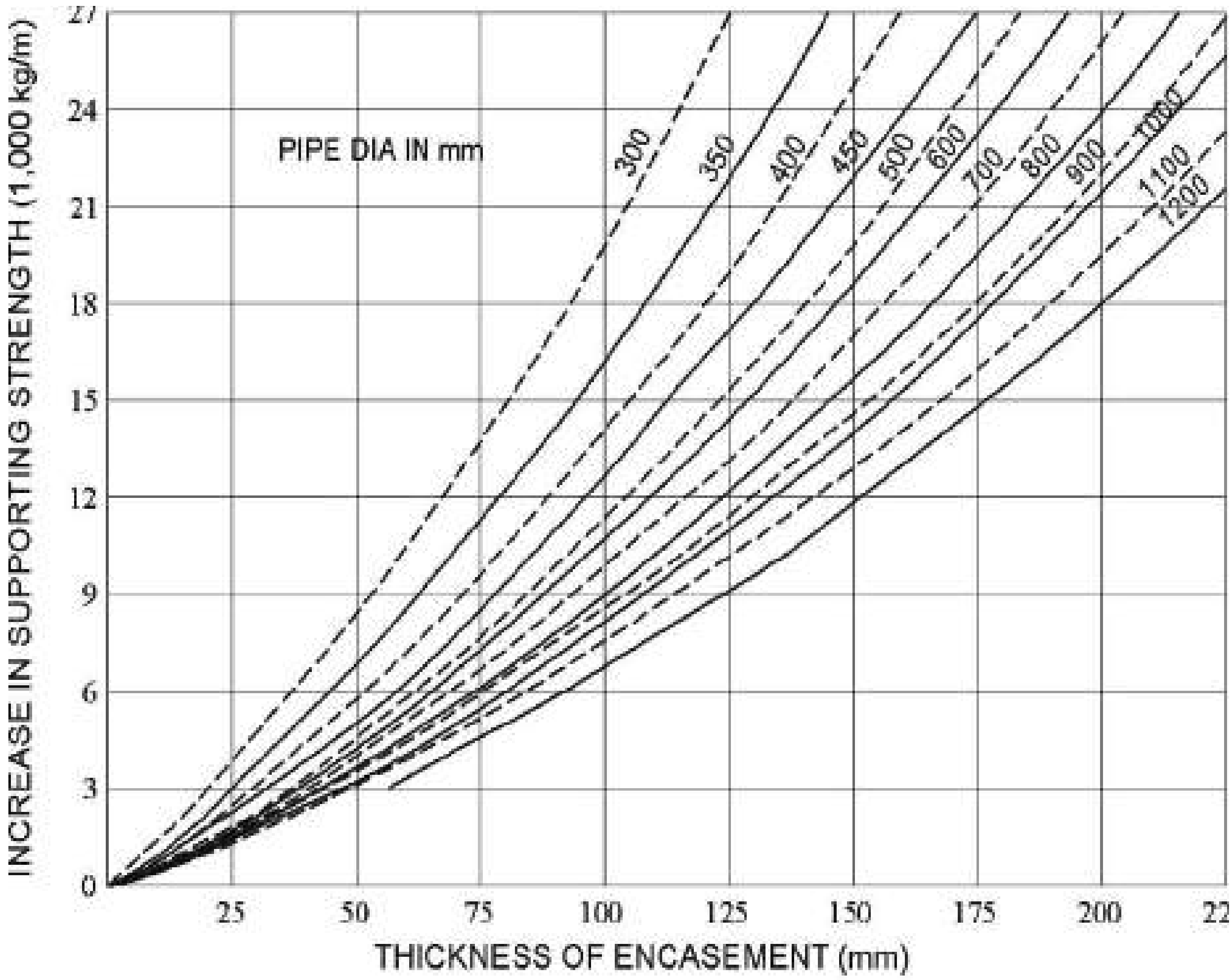
Class A: The hard unyielding material should be excavated down to the bottom of the concrete cradle.

Class B or Class C: The hard, unyielding material should be excavated below the bottom of the pipe and pipe bell, to a depth of at least 15 cm.

The width of the excavation should be at least 1.25 times the outside diameter of the pipe and it should be refilled with granular material.

Total encasement of non-reinforced rigid pipe in concrete may be necessary where the required safe supporting strength cannot be obtained by other bedding methods.

The load factor for concrete encasement varies with the thickness of concrete. The effect of M-20 concrete encasement of various thicknesses on supporting strength of pipe under trench conditions is given in Figure 3.47 overleaf.



Source: CPHEEO, 1993

Figure 3.47 Effect of M-20 concrete encasement of various thickness on supporting strength of pipe under trench conditions

3.50.3.6 Supporting Strength in Embankment Conditions

The soil pressure against the sides of a pipe placed in an embankment may be significant in resisting the vertical load on the structure.

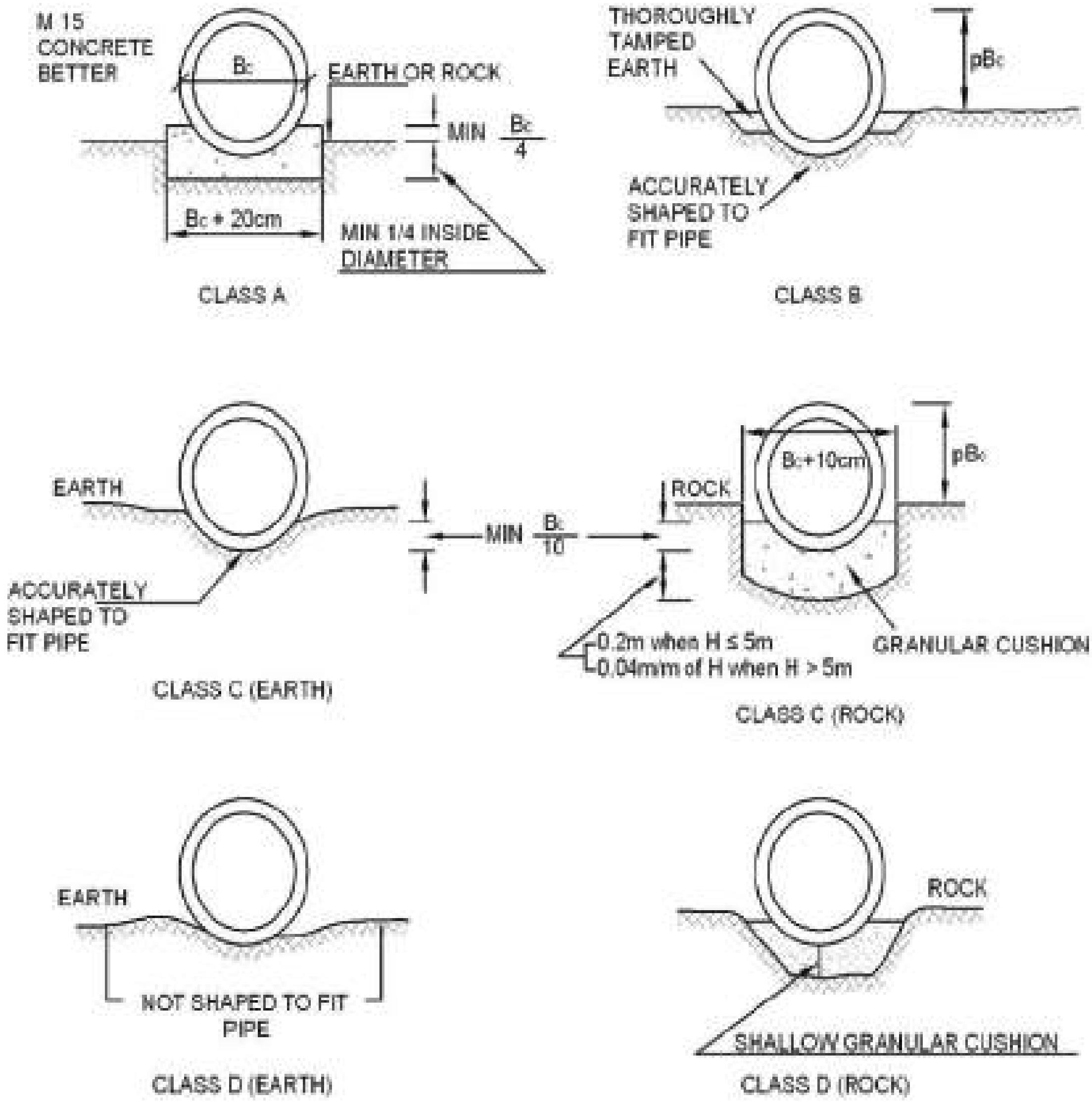
3.50.3.7 Classes of Bedding

The beddings, which are generally adopted for projecting conduits laid under the embankment conditions of installation are illustrated in Figure 3.48 overleaf.

The classifications of the beddings are as under:

CLASS A: In this case, the conduit is laid on a mat of concrete.

CLASS B: The conduit is laid on accurately shaped earth to fit the bottom of the pipe and the sides are filled with thoroughly tamped earth.



Source: CPHEEO, 1993

Figure 3.48 Classes of bedding for projecting conduits

CLASS C: In this type of bedding the conduit is laid on accurately shaped earth to fit the bottom surface of the conduit. For rock foundations, the conduit is laid on a layer of granular cushion and the sides of the conduit are filled up.

CLASS D: The conduit is laid on earth not shaped to fit the bottom of the conduit. In case of rocky soil, the conduit is laid on a shallow granular cushion.

3.50.3.8 Load Factors for Rigid Pipes

The load factor for rigid pipes, installed as projecting conduits under embankments or in wide trenches, is dependent on the type of bedding, the magnitude of the active lateral soil pressure and on the area of the pipe over which the active lateral pressure acts.

The load factor for projecting circular conduits may be calculated by the formula:

$$L_f = \frac{1.431}{NZq} \tag{3.41}$$

where,

L_f : Load factor
N : Parameter dependent on the type of bedding
Z : Parameter dependent upon the area over which the lateral pressure acts effectively
q : Ratio of total lateral pressure to total vertical load on pipe

a) Positive Projecting Conduits

The ratio q for positive projecting conduits may be estimated by the formula

$$q = (mk/C_c)\left[(H/B_c) + (m/2)\right] \tag{3.42}$$

where,

k : Rankine's ratio which may be taken as 0.33.

The value of N for different types of beddings for circular pipes is given in Table 3.25.

Table 3-25 Values of N for different pipe beddings

Type of Bedding	Value of 'N'	Type of Bedding	Value of 'N'
'A' - Reinforced concrete cradle	0.42 to 0.51	'C'	0.84
'A' - Plain concrete cradle	0.51 to 0.64	'D'	1.31
'B'	0.71		

Source: CPHEEO, 1993

The value of Z in case of circular pipes is given in Table 3.26.

Table 3.26 Values of Z for different pipe beddings

Fraction of conduit on which lateral pressure act 'm'	Value of 'Z' for		Fraction of conduit on which lateral pressure act 'm'	Value of 'Z' for	
	'A' Class Beddings	Other Beddings		'A' Class Beddings	Other Beddings
0.0	0.150	0.000	0.7	0.811	0.594
0.3	0.743	0.217	0.9	0.678	0.655
0.5	0.856	0.423	1.0	0.638	0.638

Source:CPHEEO, 1993

b) Negative Projecting Conduits

The load factor for negative projecting conduits may also be determined by the equations (3.41) and (3.42) with a value of k of 0.15, provided the side fills are well compacted.

c) Imperfect Trench Conditions

The equations for positive projecting conditions will hold good for those conditions as well.

3.50.3.8.1 Conduits under Simultaneous Internal Pressure and External Loading

Simultaneous action of internal pressure and external load gives a lower supporting strength of a pipe than what it would be if the external load acted alone.

If the bursting strength and the three-edge strength of a pipe are known, the relation between the internal pressure and external loads, which will cause failure may be computed by means of the formula:

$$t = \frac{T\left(1 - s^2\right)}{S} \qquad (3.43)$$

where,

t : Internal pressure in kg/cm^2 at failure when external load is simultaneously acting

T : Bursting strength of a pipe in kg/cm^2 when no external load is simultaneously acting

s : Three-edge-bearing load at failure in kg/linear metre when there is simultaneous action of internal pressure and

S : Three-edge-bearing load at failure in kg/linear metre when there is no internal pressure simultaneously acting.

3.51 RELATIONSHIP BETWEEN DIFFERENT ELEMENTS IN STRUCTURAL DESIGN

The basic design relationships between the different design elements are as follows for rigid pipe

$$\text{Safe working strength} = \frac{\text{Ultimate three edge bearing strength}}{\text{Factor of safety}} \qquad (3.44)$$

$$\text{Safe field supporting strength} = \text{Safe working strength} \times \text{load factor} \qquad (3.45)$$

Appendix A.3.9 gives the details of three-edge-bearing tests.

It is but obvious, that sewers have to be sturdy enough to sustain the load of the backfill material (dead load), as well as the load due to the vehicular traffic (live load). Factors like, depth of the backfill, type of this material, and width of the trench influence the magnitude of the dead load; while the parameters that determine the load-carrying capacity of the sewer line are the crushing strength of the pipe, and the characteristics of the pipe bedding. Bedding defines the way in which a pipe is placed on the bottom of the trench.

Proper bedding distributes the load around the circumference of the pipe, and this increases the supporting strength of the pipe. The ratio of actual field supporting strength to the crushing strength of the pipe is known as load factor.

It may be pointed out that class D bedding is the weakest of all, and hence is not generally adopted. Here, the trench bed being left flat and bare, the pipe is not fully supported due to its projecting bell-ends. Further, if the backfill is placed loosely over the sewer without the necessary compaction, the barrel may not get properly supported by the bedding. The ordinary bedding (Class C), offers a better support, say, with a load factor of 1.5. In first class bedding (Class B), the granular material extends halfway up the pipe, and a carefully compacted backfill can give a load factor of even 1.9. In Class A bedding, the barrel is supported by a concrete bed (yielding a load factor of 2.8) with a careful compaction of the backfill. It is common, in such engineering constructions to define a safely factor (SF) as well, such as:

$$\text{Safety Factor} = \frac{\text{Field Supporting Strength}}{\text{Safe Supporting Strength}} \tag{3.46}$$

Safety factor of 1.5 is normally adopted for clay or unreinforced concrete sewers to address the possibility of using poor quality materials or for faulty construction. With a view to selecting the best bedding condition, it is to be ensured that the safe supporting strength is equal to or greater than the total expected load over the pipe.

For pipelines situated in shallower trenches (such as, storm sewers or even some water mains), the component of load due to vehicular traffic may be a substantial part of the total load on the line. However, for deeper trenches (such as, sanitary sewers), the proportion of live load may not be significant compared to the dead load. In USA, Marston’s Formula is commonly used to determine the load due to backfill, as in Equation (3.23).

3.51.1 Field Layout and Installation

It is understood that the straight line and slope of a sewer has to be carried out meticulously as per design. The horizontal layout determines the location as well as direction of the sewer line, while slope of the line provides the necessary hydraulic carrying capacity of the sewerage system.

The location of the trench is generally laid out first as an offset line running parallel to the proposed sewer centre line. This offset line is demarcated by wooden stakes driven into the ground surface at intervals of, say, 15 m. The offset line, as is clear, is quite away from the sewer centre line with a view not to allow it being disturbed during construction; however, it has to be proximate enough so that the transfer of measurements to the actual trench can readily be done. The wooden stakes are set with their tops at a specific height above the designed trench bottom (horizontal slope line) thus, the checking of the trench depth during excavation, etc., can be done with ease.

Two procedures are available to lay pipe sections in the open trench, namely, by batter boards, and by laser beams. Batter boards are placed across the trench at uniform intervals. The tops of these boards can be set at even height above the designed sewer invert elevation.

The centre line of the sewer is traced on the boards by extending a line of sight with a transit level or a theodolite and a string is stretched from board to board along this very line. Later on, this line is transferred onto the trench bed by means of a plumb bob for the invert levels. Invert levels and characteristics indicated by vertical rods are marked off in even increments and the lower end of each rod is placed on the pipe invert bedding plane, and the string over the batter boards helps to check if it matches with the proper elevation mark on the rod, by appropriate adjustment of the pipe placement.

In the laser method, advantage is taken of an intense, narrow beam of light that is projected by the laser instrument, over a long distance. This beam is aligned through a sewer pipe to strike a target held at the other end of the pipe.

A transit that is placed above a manhole helps establish the alignment of the sewer with reference to field survey points, and transfer it down to the laser instrument that is mounted inside the manhole. Lasers can achieve an accuracy up to 0.01 per cent over a distance of up to 300 m.

3.52 CROSS DRAINAGE WORKS

Cross drainage, works arise when a sewer has to cross another service like electricity, water line, gas piping, telecommunication cable, river course, nalah, etc. The following shall be mandatorily implemented without fail.

In regard to the electric power cables, the sewer shall be laid above the electric power cable and horizontally away from the power cable with clearances of minimum 30 cm all round as per IS: 1255. In regard to water lines, the sewer shall always travel below the water line.

With regard to gas lines, the sewer has to travel above the gas line so that sewer gases, if they escape, need not accidentally set off an ignition of the gas line. With regard to telecommunication cables, lateral separation of at least 30 cm shall be followed. In cases of river crossing and nalah crossing, each situation shall be decided on its site conditions.

Gravity sewers, if possible, may be converted to pumped sewer lines by a low lift dedicated pumping station, before the crossing discharging into the gravity section after crossing the water course; this will help in keeping the pumped sewer visible to the eye or close to the ground at all times.

3.53 SEWER VENTILATORS

In a modern, well-designed sewerage system, there is no need to provide ventilation on such elaborate scale considered necessary in the past, especially with the present day policy to omit intercepting traps in house connections.

The ventilating columns are not necessary where intercepting traps are not provided. It is necessary however, to make provision for the escape of air to take care of the exigencies of full flow and to keep the sewage as fresh as possible, especially in outfall sewers. In case of storm sewers, this can be done by providing ventilating manhole covers.

3.54 PREVENTION OF CROSS CONNECTION

3.54.1 Visual Separation

A cross connection between water main and sewer main seldom occurs because of the sizes of these mains. However, where the location is complicated, the water mains shall be either blue coloured pipes or shall be painted with blue florescent coloured paint.

3.54.2 Protection of Water Mains

A minimum offset of equal to half the width of the manhole plus 30 cm shall be the lateral offset between water mains and sewer lines. It is advisable to encase the sewer than the water mains.

3.54.3 Relation to Waterworks Structures

Gravity sewers shall not be laid closer to water retaining structures and the effort should be to detour as far as possible. In case of leakages in sewer joints, the leakage may gain access to the sidewalls of the water retaining structures.

A simpler precaution if possible will be to use CI or DI pipes for that length of sewer that runs close to the water retaining structure

3.54.4 Construction Methods

The design and the construction of sewers are interdependent; the knowledge of one is an essential prerequisite to the competent performance of the other.

The ingenuity of the designer and supervising engineer is continually called for, to reduce the construction cost and to achieve quality workmanship. Barring unforeseen conditions, it shall be the responsibility of the supervising engineer and the contractor to complete the work as shown on the plans at minimum cost and with minimum disturbance of adjacent facilities and structures.

3.54.5 Trench

3.54.5.1 Dimensions

The width of trench should be the minimum necessary for the proper installation of the sewer with the due consideration to its bedding. It depends on the type of shoring (single stage or two stage), working space required in the lower part of the trench and the type of ground below the surface. The width of the trench at different levels from the top of the sewer to the ground surface is primarily related to its effect upon the adjoining services and nearby structures.

In undeveloped areas or open country, excavation with side slope shall be permissible from the top of the sewer to the ground surface instead of vertical excavation with proper shoring. In developed areas, however, it is essential to restrict the trench width to protect the existing facilities and properties and to reduce the cost of restoring the surface. Increase in width over the minimum required would unduly increase the load on the pipe.

3.54.5.2 Excavation

Excavation for sewer trenches for laying sewers shall be in straight lines and to the correct depths and gradients required for the pipes as specified in the drawings. The material excavated from the trench shall not be deposited very close to the trench to prevent the weight of the materials from causing the sides of the trench to slip or fail. The sides of the trench shall, however, be supported by shoring where necessary to ensure proper and speedy excavation. In case, the width of the road or lane where the work of excavation is to be carried out is so narrow as to warrant the stacking of materials near the trench, the same shall be taken away to a place to be decided by the Engineer-in-Charge. This excavated material shall be brought back to the site of work for filling the trench. In case the presence of water is likely to create unstable soil conditions, a well point system erected on both sides of the trench shall be employed to drain the immediate area of the sewer trench prior to excavation operation. A well point system consists of a series of perforated pipes driven into the water bearing strata on both sides of a sewer trench and connected with a header pipe and vacuum pump. If excavation is deeper than necessary, the same shall be fitted and stabilized before laying the sewer.

3.54.5.3 Shoring

The shoring shall be adequate to prevent caving in of the trench walls by subsidence of soil adjacent to the trench. In narrow trenches of limited depth, a simple form of shoring shall consist of a pair of 40 to 50 mm thick and 30 cm wide planks set vertically at intervals and firmly fixed with struts. For wider and deeper trenches, a system of wall plates (Wales) and struts of heavy timber section is commonly used. Continuous sheeting shall be provided outside the wall plates to maintain the stability of the trench walls. The number and the size of the wall plates shall be fixed considering the depth of trench and type of soil. The cross struts shall be fixed in a manner to maintain pressure against the wall plates, which in turn shall be kept pressed against the timber sheeting by means of timber wedges or dog spikes. In non-cohesive soils combined with considerable ground water, it may be necessary to use continuous interlocking steel sheet piling to prevent excessive soil movements by ground water percolation and extend the piling at least 1.5 m below the trench bed. In case of deep trenches, excavation and shoring may be done in stages.

A mechanized shoring is presented in Figure. 3.49

Figure 3.49

Work in execution by latest technology for sewer laying in India (Mumbai). Steel anchors have grooves for slotting precast RCC slabs or other sheeting and the anchors are held in place by steel adjustable struts. Specific advantage is easiness of pulling out the anchors, sheeting and struts by deploying mechanical equipment.

Source: Bihar Urban Development Corporation Detailed Project Report for Saidpur Sewer Network - Patna (Package 6)

3.54.5.4 Underground Services

All other services like pipes, ducts, cables, mains and other services exposed due to the excavation shall be effectively supported.

3.54.5.5 Dewatering

Trenches for sewer construction shall be dewatered for the placement of concrete and laying of pipe sewer or construction of concrete or brick sewer and kept dewatered until the concrete foundations, pipe joints or brick work or concrete have cured. The pumped-out water from the trenches shall be disposed off in existing storm water drainage arrangement nearby.

In the absence of any such arrangement, the pumped water may be drained through completed portion of sewer to a permanent place of disposal. Where a trench is to be retained dry for a sufficient period to facilitate the placement of forms for sewer construction, an under drain shall be laid of granular material leading to a sump for further disposal. Precautions are to be taken to arrest potential floating of the laid sewers, arising out of induced buoyancy during rainy season.

3.54.5.6 Foundation and Bedding

Where a sewer has to be laid in a soft underground stratum or in a reclaimed land, the trench shall be excavated deeper than what is ordinarily required. The trench bottom shall be stabilized by the addition of coarse gravel or rock. In case of very bad soil, the trench bottom shall be filled in with cement concrete of appropriate grade. In the areas subject to subsidence, the pipe sewer should be laid on suitable supports or concrete cradle supported on piles.

In the case of cast-in-situ sewers, an RCC section with both transverse and longitudinal steel reinforcement shall be provided when intermittent variations in soil bearing capacity are encountered. In case of long stretches of very soft trench bottom, soil stabilization shall be done either by rubble, concrete or wooden crib.

3.54.5.7 Tunnelling

Tunnels are employed in sewer systems when it becomes economical, considering the nature of soil to be excavated and surface conditions with reference to the depth at which the sewer is to be laid. Generally, in soft soils the minimum depth is about 10 m. In rocks, however, tunnels may be adopted at lesser depths. In busy and high activity zones, crowded condition of the surface, expensive pavements or presence of other service facilities near the surface sometimes, make it advantageous to tunnel at shallower depths. Each situation has to be analysed in detail before any decision to tunnel is taken.

3.54.5.8 Shafts

Shafts are essential in tunneling to gain access to the depth at which tunnelling is to be done to remove the excavated material. The size of shaft depends on the type and size of machinery employed for tunneling, irrespective of the size of the sewer.

3.55 METHODS OF TUNNELLING

The tunneling methods adopted for sewer construction can be classified generally as auger or boring, jacking and mining.

a) Auger or Boring

In this method, rigid steel or concrete pipes are pushed into the ground to reasonable distances and the earth is removed by mechanical means from the shaft or pit location. Presence of boulders is a serious deterrent for adoption of this method, in which case it may be more economical to first install an oversize lining by conventional tunnelling or jacking and fill the space between the pipe and lining with sand, cement or concrete.

b) Jacking

In this procedure, the leading pipe is provided with a cutter or edge to protect the pipe while jacking. Soil is gradually excavated and removed through the pipe as successive lengths of pipes are added between the leading pipe and the jacks and pushed forward taking care to limit the jacking up to the point of excavation. This method usually results in minimum disturbance of the natural soils adjacent to the pipe. The jacking operation should continue without interruption as otherwise soil friction might increase, making the operation more difficult.Jacking of permanent tunnel lining is generally adopted for sewers of sizes varying from 750 to 2,750 mm, depending upon the conditions of soil and the location of the line.

The pipes selected should be able to withstand the loads exerted by the jacking procedure. The most common pipes used for this are reinforced concrete or steel.

c) Mining

Tunnels larger than 1.5 m are normally built with the use of tunnel shields, boring machines or by open face mining depending on the type of material met with. Rock tunnels normally are excavated open-face with conventional mining methods or with boring tools. These are used as a safety precaution in mining operations in very soft clay or in running sand especially in built up areas. In this method, a primary lining of adequate strength to support the surrounding earth is installed to provide progressive backstop for the jacks which advance the shield.

As the excavation continues, the lining may be installed either against the earth, filling the annular space by grouting with pea gravel or the lining may be expanded against the earth, the latter eliminating the need for any grouting. Boring machines of different types have been developed for tunnel excavation in clay and rock and are equipped with cutters mounted on a rotating head, which is moved forward.

The excavated earth is usually carried by a conveyor system. Some machines are also equipped with shields. Though the machines are useful in fairly long runs through similar material, difficulties are encountered when the material varies. Open face mining without shields are adopted in particular instances such as in rock. Segmental support of timber or steel is used for the sides and the top of the tunnel.

3.56 LAYING OF PIPE SEWERS

In laying sewers, the centre of each manhole shall be marked by a peg. Two wooden posts 100 mm x 100 mm and 1,800 mm high shall be fixed on either side at nearly equal distance from the peg or sufficiently clear of all intended excavation. The sight rail when fixed on these posts shall cross the centre of manhole. The sight rails made from 250 mm wide x 40 mm thick wooden planks and screwed with the top edge against the level marks and shall be fixed at distances more than 30 m apart along the sewer alignment. The centre line of the sewer shall be marked on the sight rail. These vertical posts and the sight rails shall be perfectly square and planed smooth on all sides and edges. The sight rails shall be painted half-white and half-black alternately on both the sides and the tee heads and cross pieces of the boning rods shall be painted black. When the sewers converging to a manhole come in at various levels, there shall be a rail fixed for every different level.

The boning rods with cross section 75 mm x 50 mm of various lengths shall be prepared from wood. Each length shall be a certain number of metres and shall have a fixed tee head and fixed intermediate cross pieces, each about 300 mm long. The top edge of the cross pieces shall be fixed at a distance below the top edge equal to, the outside diameter of the pipe, the thickness of the concrete bedding or the bottom of excavation, as the case may be. The boning staff shall be marked on both sides to indicate its full length.

The posts and the sight rails shall not be removed in any case until the trench is excavated, the pipes are laid, jointed and the filling is started.

When large sewer lines are to be laid or where sloped trench walls result in top-of-trench widths too great for practical use of sight rails or where soils are unstable, stakes set in the trench bottom itself on the sewer line, as rough grade for the sewer is completed, would serve the purpose.

3.56.1 Stoneware Pipes

The stoneware pipes shall be laid with sockets facing up the gradient, on desired bedding. Special bedding, hunching or encasing may be provided where conditions so demand (as discussed in Section 3.50). All the pipes shall be laid perfectly true, both to line and gradient, IS 4127. At the close of each day’s work or at such other times when pipe is not being laid, the end of the pipe should be protected by a close fitting stopper.

3.56.2 RCC Pipes

The RCC pipes shall be laid in position over proper bedding, the type of which may be determined in advance, the abutting faces of the pipes being coated by means of a brush with bitumen in liquid condition. The wedge shaped groove in the end of the pipe shall be filled with sufficient quantity of either special bituminous compound or sufficient quantity of cement mortar of 1:3. The collar shall then be slipped over the end of the pipe and the next pipe butted well against the “O” ring by appliances to compress roughly the “O” ring or cement mortar into the grooves. Care being taken to see that concentricity of the pipes and the levels are not disturbed during the operation. Spigot and socket RCC pipes shall be laid in a manner similar to stoneware spigot and socket pipes. The structural requirements as discussed in this chapter and IS 783 may be followed.

3.56.3 Cast-in-situ Concrete Sections

For sewer sizes beyond 2 m internal diameter cast-in-situ concrete sections shall generally be used, the choice depending upon the relative costs worked out for the specific project. The concrete shall be cast in suitable number of lifts usually two or three. The lifts are generally designated as the invert, the side wall and the arch.

3.56.4 Construction of Brick Sewers

Sewers larger than 2 m are generally constructed in brick work. The brickwork shall be in cement mortar of 1:3 and plastered smooth with cement plaster of 1:2, 20 mm thick both from inside and outside. A change in the alignment of brick sewer shall be on a suitable curve conforming to the surface alignment of the road. The construction shall conform to IS 2212 in general.

3.56.5 Cast Iron Pipes

The pipes shall be laid in position with the socket ends of all pipes facing up gradient. When using lead joints, any deviations either in plan or in elevation of less than 11 1/4 degree shall be effected by laying the straight pipes round the flat curve of such radius that the minimum thickness of lead in a lead joint at the face of the socket shall not be reduced below 6 mm. The spigot shall be carefully pushed into the socket with one or more laps of spun yarn wound round it. Each joint shall be tested before running the lead, by passing completely round it, a wooden gauge notched out to the correct depth of lead and the notch being held close up against the face of socket. When using the “O” ring joints, each “O” ring shall be inserted fully and verified by a toll with prior marking of the socket depth, which, when inserted after the “O” ring joint will reveal that the “O” ring has been fully inserted in position. Special precautions by manufacturers, if any, shall also be followed. Flange joints shall be used with appropriate specials and tail-pieces when inserting a fitting like a meter or a valve in the pipeline. IS 3114 should be followed in setting out the sewers.

3.56.6 Ductile Iron Pipes

The same procedures and precautions for laying as in the case of cast iron pipes shall apply here also.

3.56.7 Solid Wall UPVC Pipes

The single most important precaution is to ensure that the excavated trench is not water logged. Where situations imply water logging, it is mandatory to employ a well point dewatering system running 24 hours, 7 days a week to hold the subsoil water at least 50 cm below the bedding elevation. Thereafter, the grade of the trench having been checked, lower the pipe with socket ends facing the up gradient. When a pipe needs to be cut to suit a given distance, the pipe shall be cut perpendicular to its axis, using a firm hand held saw. Then bevel the cut end by a bevelling tool or power tool to the same angle as in the original uncut pipe and mark the insertion line freshly using an indelible black paint to retain the guide limit for insertion. Carefully remove any loose soil from the socket and do not remove the “O” ring from its housing. Check by hand whether the “O” ring is seated uniformly. Thereafter, place the pipe spigot end near the socket.

3.56.8 Solid Wall HDPE Pipes

Unlike in the case of CI, DI, UPVC pipe sewers, the HDPE sewers are normally butt welded and pre-assembled on ground and only then lowered inside the trench spanning manhole to manhole. The butt-welding shall follow the manufacturer's recommendations. Where flanged joints are needed for attaching or inserting fittings and specials like valves, the free end of the HDPE pipe shall be butt -welded with a standard flange and thereafter the flanged jointing can be made. However, in the case of such pipes, the uplift during high groundwater conditions above the pipe level is a problem specifically in high ground water and coastal areas. The concrete surrounds or venteak piles shall be used to hold these in place in such conditions, where ground water can rise above the sewer.

3.56.9 Structured Wall Pipes

The IS 16098 (Part-1), IS 16098 (Part-2) and EN 13476 also cover the performance requirements for the respective materials. These pipes are manufactured with externally corrugated wall or with T-beam type of wall with hollows between the webs of the T beams. These are laid in almost the same way as the UPVC pipes. These outer-ribbed wall pipes are jointed with “O” rings after due cleaning of dust, etc., using push-tight method and these rings help in preventing the escape of the contained fluid.

3.56.10 Double Wall Corrugated Polyethylene Pipes

Please refer section 3.12.9.3.

3.56.11 Relative Limitations in Pipe Materials in Some Situations

The merits and demerits of different pipe materials are covered in Section 3.12 and their laying is covered in this Section 3.56.

It will be useful to keep in mind that sewers pass through a whole length of roads in a habitation and varying soil conditions, bedding conditions, locations, etc., will be encountered at various places. Hence, a particular pipe material may be suitable in a particular location but may require some other material at some other locations.

A guide for this is presented in Appendix A.3.10. This may be referred to during field execution of the sewer pipes and necessary local adjustments can be made.

3.57 LOAD CARRYING MECHANISM OF THE PIPES

The non-metallic and non-concrete sewer pipes behave integral with the surrounding soil when it comes to structural behaviour. As loads are superimposed, the pipe cross section may tend to deflect by marginal reduction in vertical diameter. This may induce an increase in horizontal diameter, but this increase will be resisted by the lateral soil pressure and eventually there arises a near uniform radial pressure around the pipe and a compressive thrust. This is illustrated in Figure 3.50 overleaf.

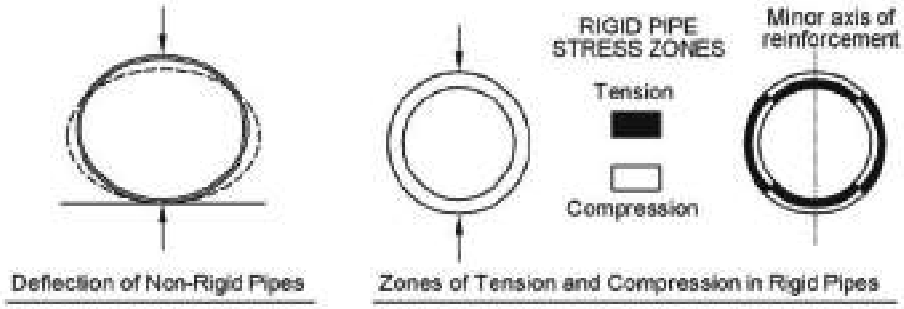


Figure 3.50 Mechanism of resisting superimposed loads by rigid and non-rigid piping

Thus, adequate backfilling in layers and compaction in each layer is of paramount importance. On the contrary, the metallic and concrete pipes, being stiffer than the surrounding soil, they carry a substantial portion of the applied load but the shear stress in the haunch area can be critical specifically, when haunch support is inadequate. Thus the load carrying mechanism of both these classes of pipe materials are dependent on the haunch supports and proper backfilling.

3.58 JOINTING OF SEWERS

Joints of pipe sewers may generally be any of the following types:

i) Spigot and socket joint (rigid and semi flexible)
ii) Collar Joint (rigid and semi flexible)
iii) Cast Iron detachable joint (semi flexible)
iv) Coupling joint (semi flexible)

Cement joints are rigid and even a slight settlement of pipes can cause cracks and hence leakage. To avoid this problem it is recommended that semi flexible joints be used.

3.58.1 Stoneware Pipes

All the pipe joints shall be caulked with tarred gasket in one length for each joint and sufficiently long to entirely surround the spigot end of the pipe. The gasket shall be caulked lightly home but not so to occupy more than a quarter of the socket depth. The socket shall then be filled with a mixture of one part of cement and one part of clean fine sand mixed with just sufficient quantity of water to have a consistency of semi-dry condition. A fillet shall be formed round the pint with a trowel forming an angle of 45 degrees with the barrel of the pipe IS 4127. Rubber gaskets may also be used for jointing. A method of relatively easier checking of the grade of SW pipe sewer line is followed by the CMWSSB. In this method, two tight strings connected to the crown and one horizontal diameter edge as shown in Figure 3-51 (overleaf) are used to judge and adjust the grade, which is much faster and more precise than the boning rod method which becomes cumbersome.

3.58.2 Concrete Pipes

Concrete spigot and socket pipes are laid and jointed as described above for glazed stoneware spigot and socket pipes with yarn or rubber gasket and cement.

Source: CMWSSB, Chennai, 2012

Figure 3.51 A simpler approach to initial laying and aligning the SW sewers by the two tight twines one on crown and one on mid diameter are used

Asbestos cement pipes are joined by coupling joints or CI detachable joints.

Large size concrete sewers have ‘ogee’ joints in which the pipe has mortise at one end and a tendon to suit at the other end. They are jointed with cement or asphalt. A concrete collar sufficiently wide to cover and overlap the joint is fixed on it.The collars shall be placed symmetrcally over the end of two pipes and the annular space between the inside of the collar and the outside of the pipe shall be filled with hemp yarn soaked in tar or cement slurry tamped with just sufficient quantity of water to have a consistency of semi dry condition. This is well packed and thoroughly rammed with caulking tools and then filled with cement mortar (1:2) prop. The joints shall be finished off with a fillet sloping at 45 degrees to the surface of the pipe. The finished joints shall be protected and cured for at least 24 hours. Any plastic solution or cement mortar that may have squeezed in shall be removed to leave the inside of the pipe perfectly clean. For more details of jointing procedure, reference may be made to IS 783.

3.58.3 Cast Iron Pipes

For CI pipes several types of joints such as rubber gasket known as Tyton joint, mechanical joint known as screw gland joint and conventional joint known as lead joint are used. For details refer to CPHEEO Manual on Water Supply and Treatment and relevant Indian Standards.

3.58.4 Ductile Iron pipes

The same procedures and precautions as in the case of cast iron pipes shall apply here too.

3.58.5 Solid Wall UPVC Pipes

Just before jointing, the lubricating material supplied by the pipe manufacturer shall be uniformly applied around the spigot end and onto the “O” ring surface to be in contact with the spigot end after jointing.

Do not remove the O ring for doing this. Thereafter use the lateral force by pushing the socket end of the pipe to be inserted by placing a wooden plank across its face and using a crowbar plunged and anchored into the soil as a level. Do not try to hit the pipe socket. When the insertion mark is reached at the face of socket, stop the work.

3.58.6 Solid Wall HDPE Pipes

The jointing is by welded heat fusion of the pipe cut surfaces to be jointed. The temperature and the time of contact are generally specified by the manufacturer.

3.58.7 Structured Wall Pipes

These are mainly by “O” ring gaskets inserted in a spigot-socket arrangement. In the case of T-Beam wall type pipes, these ends are made integral with the pipe. In the case of externally corrugated pipes, once the pipe ends are positioned and verified for alignment, lubricate the “O” ring in the correct slit as indicated by the manufacturer and push the coupling to its designated location. The “O” ring is fixed into these circular recesses and the piping is slid over it using a separate coupling, which slides over the “O” ring and brings about the jointing. Structured wall pipes are laid and jointed between them or between structured wall and solid wall type cells. The precautions will be to make sure that pipe ends and couplings are cleaned free of extraneous matter and test slide the coupling to mark the pipe ends at half the coupler length to ascertain the lengths of the pipe inside the coupling.

3.58.8 Double Walled Corrugated Pipes

These are jointed the same way as externally corrugated structured wall piping.

3.59 PRECAUTIONS AGAINST UPLIFT

Other than the metallic and concrete pipe sewers, the uplift during high groundwater conditions above the pipe level is a problem specifically in high ground water locations, water logged locations and coastal areas. The concrete surrounds or venteak piles shall be used to hold these in place in such conditions, where the ground water can rise above the sewer.

3.60 THE WATER JETTING ISSUES

With the Honourable courts levying huge penalties if man entry is practiced for sewer cleaning and with the commitment of the ULB to do away with this practice, the mechanical methods of sewer cleaning have gained momentum and is being practiced more widely in recent years. These machines, which are popularly known as jet rodders, jet the water or secondary treated sewage into the sewers by a jack-hammer action at high pressures as in Figure 3.10.

The ability of the stoneware, cast iron, concrete, HDPE/PE/PP/PVC sewer pipes to withstand the pressures to establish the permissible pressure ratings has to be evolved in India. However, an available literature is from the Sewer Jetting Code of Practice first published in 2001 in UK and which provides guidance on jetting pressure for different types of sewer pipes, as in Table 3-27 overleaf.

Table 3.27 Maximum jetting pressure in case of different types of pipes

No.	Maximum Jetting Pressure	Concrete	Clay	Plastic	Bricks/fibre
1	Meter of Water	3450	3450	1800	1030
2	BAR	345	345	180	103

Source: Water Research Centre, 2005

Care must be exercised in the field when applying the pressures to clean the sewers since the pressures stated in Table 3.27 are the maximum pressures.

3.61 TESTING OF SEWER LINES

3.61.1 Water Test

Each section of sewer shall be tested for water tightness preferably between manholes. To prevent change in alignment and disturbance after the pipes have been laid, it is desirable to backfill the pipes up to the top, keeping at least 90 cm length of the pipe open at the joints. However, this may not be feasible in the case of pipes of shorter length, such as stoneware and RCC pipes. With concrete encasement or concrete grade, partial covering of the pipe is not necessary.

In case of concrete and stoneware pipes with cement mortar joints, pipes shall be tested three days after the cement mortar joints have been made. It is necessary that the pipelines be filled with water for about a week before commencing the application of pressure to allow for the absorption by pipe wall. The sewers are tested by plugging the ends with a provision for an air outlet pipe with stop-cock in the upper end. The water is filled through a funnel connected at the lower end provided with a plug. After the air has been expelled through the air outlet, the stop-cock is closed and water level in the funnel is raised to 2.5 m above the invert at the upper end. Water level in the funnel is noted after 30 minutes and the quantity of water required to restore the original water level in the funnel is determined. The pipe line under pressure is then inspected while the funnel is still in position. There shall not be any leaks in the pipe or the joints (small sweating on the pipe surface is permitted). Any sewer or part thereof not meeting the test shall be emptied and repaired or re-laid as required and tested again.

The leakage or quantity of water to be supplied to maintain the test pressure during the period of 10 minutes shall not exceed 0.2 litres/mm dia. of pipes per kilometre length per day.

For non-pressure pipes, it is better to observe the leakage for a period of 24 hours if feasible. The test for exfiltration for detection of leakage shall be carried out at a time when the groundwater table is low.

For concrete, RCC and asbestos cement pipes of more than 800 mm dia. the quantity of water inflow can be increased by 10% for each additional 100 mm of pipe dia.

For brick sewers, regardless of their diameters, the permissible leakage of water shall not exceed 10 cubic meters for 24 hours per km length of sewer.

3.61.2 Air Testing

Air testing becomes necessary particularly in large diameter pipes when the required quantity of water is not available for testing. As per the ASTM C28-80, vitrified clay pipes testing is specified as applying air pressure to 2.8 m water column and held for 2 to 5 minutes when all plugs are checked and the exact point of leakage can be detected by applying soap solution to all the joints in the line and looking for air bubbles. Thereafter, the air supply is disconnected and the time taken to drop from 2.5 m to 1.7 m water column for every 30 m is noted to be in conformity with Table 3.28.

Table 3.28 Minimum test times per 30 m of vitrified clay sewer line for air testing

Diameter, mm	minutes	Diameter, mm	minutes	Diameter, mm	minutes
100	0.3	400	2.1	750	4.8
150	0.7	450	2.4	800	5.4
200	1.2	500	3.0	900	6.0
250	1.5	600	3.6	950	6.6
300	1.8	700	4.2	1,070	7.3

The longer lengths and hence fewer joints of sewer pipelines when laid with RCC and double walled HDPE pipes must be able to easily withstand the above testing and hence, the same test conditions are retained for these sewers also. A typical arrangement is shown in Figure 3.52.

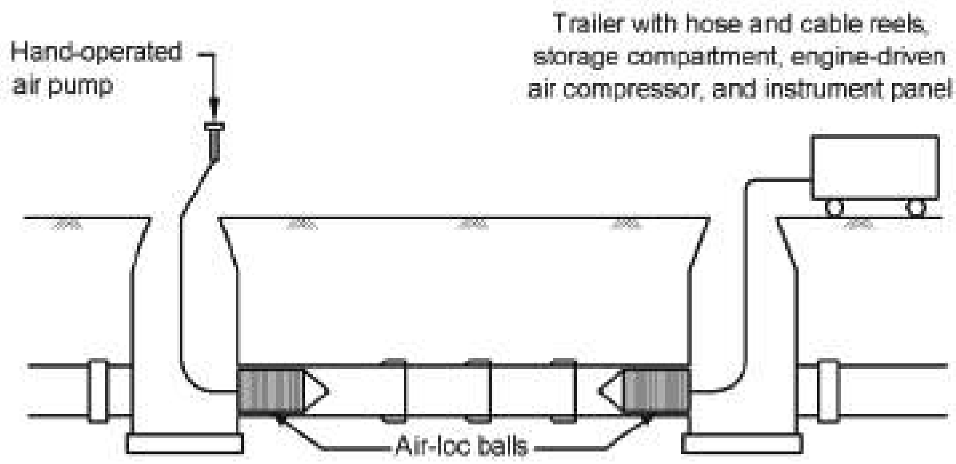


Figure 3.52 Typical arrangement for low pressure air testing of sewer pipeline

3.62 CHECK FOR OBSTRUCTION

As soon as a stretch of sewer is laid and tested, a double disc or solid or closed cylinder, 75 mm less in dimension than the internal dimension of the sewer shall be run through the stretch of the sewer to ensure that it is free from any obstruction.

3.63 BACKFILLING OF THE TRENCHES

Backfilling of the sewer trench is a very important consideration in sewer construction. The method of backfilling to be used varies with the width of the trench, the character of the material excavated, the method of excavation and the degree of compaction required. In developed streets, a high degree of compaction is required to minimize the load while in less important streets, a more moderate specification for back fill may be justified. In open country, it may be sufficient to mound the trench and after natural settlement, return to re-grade the areas.

No trench shall be filled in unless the sewer stretches have been tested and approved for water tightness of the joints. However, partial filling may be done keeping the joints open to avoid any disturbance. The refilling shall proceed around and above the pipes. Soft material screened free from stones or hard substances shall first be used and hand pressed under and around the pipes to half their height. Similar soft material shall then be put up to a height of 30 cm above the top of the pipe and this will be moistened with water and well rammed. The remainder of the trench can be filled with hard material, in stages, each not exceeding 60 cm. At each stage, the filling shall be well rammed, consolidated and completely saturated with water and then only further filling shall be continued. Before and during the backfilling of a trench, precautions shall be taken against the floatation of the pipeline due to entry of large quantities of water into the trench causing an uplift of the empty or the partly filled pipeline. Reference may be made to section 3.46 for more details in this regard. Upon completion of the backfill, the surface shall be restored fully to the level that existed prior to the construction of the sewer.

3.64 REMOVAL OF SHEETING

Sheeting driven below the spring line of a sewer shall be withdrawn a little at a time as the back-filling progresses. Some of the backfilled earth is forced into the void created by withdrawing the sheeting by means of a water jet. To avoid any damage to buildings, cables, gas mains, water mains, sewers, etc., near the excavation, or to avoid disturbance to the sewer already laid portions of the sheeting may be left in the trenches.

3.65 SEWER REHABILITATION

Disrepair of sewers renders them leaky; and, as a result, they carry large volumes of infiltration water. They, most often get, blocked and sometimes collapse. The expenditure of excavating and then replacing a portion of badly functioning sewer is prohibitive. It is, therefore, economical to repair and rehabilitate the system as such. Therefore, continuing sewer maintenance efforts have to be designed with a view to preventing unnecessary deterioration of the sewer system. Any maintenance programme that may be adopted depends on the nature of the problem, necessity of maintaining the flow while the repair is being carried out, the expected traffic disruption that may be caused, safety aspects that need be addressed, and the cost that has to be borne.

It is necessary to clean the sewer lines before embarking on a visual inspection. This is commonly done by flushing the sewer by using a fire hose, connected to a hydrant, which discharges into a manhole.

However, caution is to be applied to avoid backups into the surrounding buildings that are connected to the system. Another method to clean the sewers is by using a soft rubber ball that is inflated to match the diameter of the pipe and later being pulled by a cord via the reach of the line between manholes. Power rodding machines or power winches (to pull a bucket through the line) can also be used. Moreover, it is to be looked into that the collected debris is disposed of properly. Inspections, after cleansing operations, are made during low-flow periods using flashlights. Use of closed-circuit television system (even making a photographic or videotape record) gives accurate location of leaks, root intrusions and any structural problems. A common method for sealing leaks in otherwise structurally sound pipelines comprises chemical grouting, the grout is applied internally to joints, holes, and cracks. In smaller or medium sized lines, inflatable rubber sleeves are generally pulled through, while in large sized lines workers place a sealing ring manually over the defective joint and, the grout is pumped through a hand held probe. However, as a safety measure, the air in the sewer must be tested for carbon monoxide, hydrogen sulphide, and explosive gases before allowing entry to workers.

Crown corrosion can cause structural damage to sewers. Large sewers, suffering this damage can be strengthened by applying a lining of gunite, a mixture of fine sand, cement, and water. It is applied internally by means of pneumatic spraying. Quite long lengths of concrete sewers are effectively rehabilitated with gunite lining. To renew on extensively cracked sewer lines a procedure known as slip lining is adopted. It comprises pulling a flexible plastic liner pipe into the damaged pipe and then reconnecting all the individual service connections to the liner. It may be necessary sometimes, to fill the narrow annular space between the lines and the existing pipe with grout preventing relative movement. However, it may be pointed out that multiple excavations are required to reconnect each service line to the new liner. In a relatively new and sophisticated method, namely, Inversion lining, a flexible liner is used. This line expanding to fit over the pipe geometry is thermally hardened and the procedure avoids excavations for service line connections.

Concrete manholes may also suffer sulphuric acid corrosion. Severe cases may need total replacement of the manhole. For less severe cases, the deteriorated material is removed using water or sand blasting, or mechanical tools, and then special chemical preparations are applied to stabilize the remaining material. Next, high strength patching mortar is used in filling in the irregularities in the internal surface; and lastly a lining or a coating has to be applied.

Manholes are sometimes subject to surface water inflow and/or ground water infiltration, and it is an unacceptable situation. This circumstance can arise due to holes in the manhole cover; spaces between the cover and the frame; and poor sealing of the frame of the cover. Frames can be resealed using hydraulic cement, and water-proof epoxy coating. Sometimes, the manhole frame and cover are raised, and the exposed portion is coated with asphalt or cement. One more method consists in installing a special insert between the frame and the cover and it does not allow water and grit to enter the manhole while allowing gas to escape through a relief valve.

Infiltration of groundwater through the sidewall of a manhole and its base, or around pipe entrances is solved by chemical grouting; being a less costly method compared to lining or coating, it also needs no preparatory restoration of the surface. Further, cracks and opening get sealed by pressure injection of the gel or foam (grouting materials).

House (service) connections and smaller diameter pipes, join the lateral sewer line in the street with the building that the sewer line serves. These house lines are also known as building sewers or service laterals, and can be as long as 30 m. These can develop defects like cracks and open-jointed pipes, causing considerable infiltration of groundwater. The total length of service connections can often be greater than the length of the main sewers. Therefore, the maintenance of these lines is also equally important. Chemical grouting and inversion lining procedures are often helpful.

Sewers, which are determined to be critical after inspection, have to be taken up for rehabilitation. Sewer rehabilitation is necessitated either to improve the hydraulic performance of the existing line or due to danger of the sewer line deteriorating further and leading to eventual collapse or failure.

3.65.1 Methods

Sewer rehabilitation may be carried out by renovation or by renewal of the sewer. When the condition of the sewer is improved either to increase its carrying capacity or to increase its life, it is known as renovation. When the sewer line is reconstructed or replaced to the same dimensions as existing, it is known as renewal.

3.65.2 Sewer Renovation

While preparing the DPRs for a habitation where a sewerage system is already in place, it is equally important to consider and provide for renovation of old sewers as well especially, when the old and augmented systems will be functioning contingent upon each other. In the renovation of sewers, the original sewer fabric is utilized and improvements are carried out; the various methods utilized are:

a) Stabilization where painting or chemical grouting of the joints is carried out

b) Pipe linings in which pipes of slightly smaller diameter than the sewer are inserted

Pipes may be of Glass Reinforced Plastic (GRP) and HDPE which can be butt fusion welded. The in-situ tube, manufactured of polyester felt and impregnated with a resin mixed with a special catalyst is tailored to suit the internal diameter/dimension of the pipe. The in-situ tube is inserted from any manhole, opening, etc. During insertion, the tube turns inside out so that the polyurethane side forms the inside surface of the pipe. Water is pumped into the tube to a predetermined head and the tube travels down the pipe to be repaired. As the in-situ tube travels, the pressure of the water firmly presses the resin impregnated side against the pipe wall. When the in-situ tube reaches the downstream manhole, addition of water is stopped and the water heated to cure the resin. The result is a cast in-situ pipe within a pipe. An alternative method of pulling the tube in and then inflating it is also used for small diameter pipes. Recent development is the use of photo curing resins, i.e., curing by light.

c) Segmental linings of glass reinforced cement, GRP, resin concrete and precast gunite are used when man entry is possible

d) When linings are used, annulus grouting is necessary in majority of the cases for a satisfactory performance.

e) The places where sewer network is crossing a canal/distributary or a natural drain, the site conditions need to be assessed and analyzed carefully considering various available options. At the locations where sewer network is crossing a natural drain, the depth of sewer is kept in such a way that it crosses below the bed level and can be laid through open trenches. The places where large diameter network pipes are crossing a canal or distributary the crown of the pipe is kept more than “D” m below the bed level, where “D” is the diameter of the pipe so as to ensure the pipes can be laid using trenchless technology without disturbing the canal above. The criteria for selection of trenchless technology based on the diameter of pipe is presented in Table 3.29.

Table 3.29 Criteria for selection of trenchless technology

No.	Diameter of Pipe	Suitable Trenchless Technology
1	< 1,000 mm	Guided Boring
2	1,000 mm – 1,500 mm	Pipe Jacking
3	> 1,500 mm	Tunnel Boring

f) Different trenchless technologies are used for different diameters, material of construction of pipe and site conditions. These are explained in the subsequent sections.

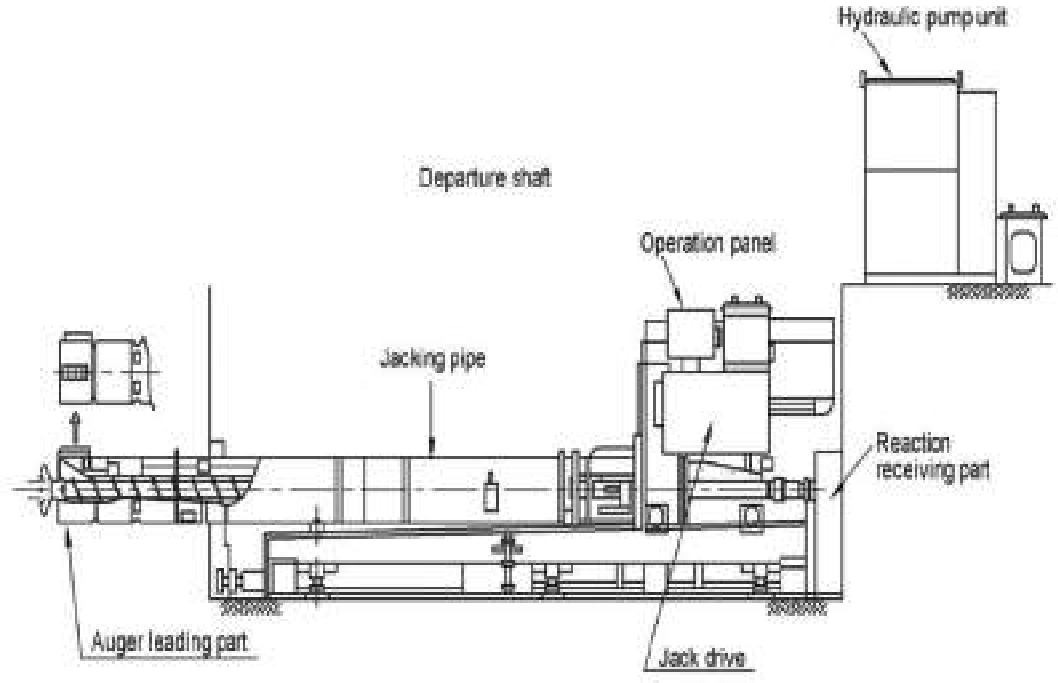
g) Guided Auger Boring

Auger boring is a technique for forming a horizontal bore under a crossing, using a cutting head and auger flights. The auger boring equipment consists of a cutting head attached to a helically wound auger flight. The rotation action of the auger flight simultaneously rotates the cutting head and removes the excavated soil from the bore. The auger flight is contained within a steel casing.

In auger boring, the auger rotates inside the casing as it is jacked. Hence, there is a danger that it may damage any interior coating or liner that may be in the pipe. The standard casing material used with auger boring is steel. Presently, most of the rail road and highway specifications require the use of steel casing with auger boring. The cutting head and auger are rotated from the drive pit by a transmission or power unit. Most auger boring systems include pipe-jacking equipment, which allows the casing to be moved forward as the cutting head advances. Once the casing has been installed, the product pipe can be inserted. The size of pipe that can be installed by this method ranges from 100 mm to more than 1,500 mm. However, the most common size range is 200 mm to 900 mm. A typical guided auger boring is shown in Figure 3.53 overleaf.

h) Pipe Jacking

Pipe jacking is a trenchless construction method, which requires workers inside the jacking pipe and is generally started from an entry pit and can be done manually or by using machines. However, it is accomplished with workers inside the pipe. The excavation method varies from the very basic process of workers digging the face with pick and shovel to the use of highly sophisticated tunnel boring machines.



Source:JSWA, 1991

Figure 3.53 Guided auger boring

Since the method requires personnel working inside the pipe, the method is limited to personnel-entry-size pipes. Hence, the minimum pipe diameter recommended by this method is 1060 mm outside diameter.

Irrespective of the method, the excavation is generally accomplished inside an artificial shield, which is designed to provide a safe working environment for the people working inside and to allow the bore to remain open for the pipe to be jacked in place. The shield is guidable to some extent with individually controlled hydraulic jacks.

The first step in any pipe jacking operation is site selection and equipment selection as per the site requirements. A pipe-jacking project should be planned properly for a smooth operation. The site must provide space for storage and handling of pipes, hoisting equipment for the pipe, spoil storage and handling facility, etc. If adequate space is available, a big jacking pit is preferred so that longer pipe segments can be jacked and the total project duration is reduced.

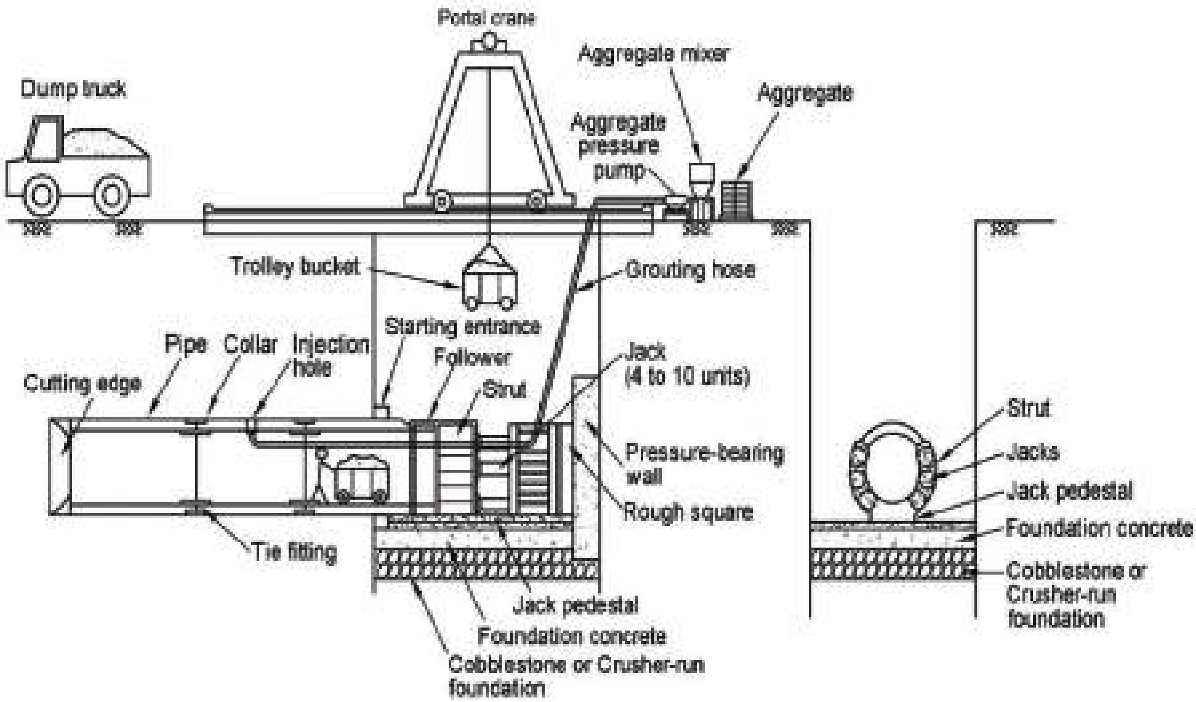
The jacking pit size is a function of pipe diameter, length of pipe segment, shield dimensions, jack size, thrust wall design, pressure rings and guide rail system. The space available at the site governs the selection of all the above components. The jacking pit should be shored and braced unless it is very shallow and high strength clay. It can be shored with timber, steel piling or shaft liner plates. Due to the jacking forces required to push large diameter pipes through the ground, the jacking pit design and construction are critical. The pit embankment supports must be properly designed and constructed. It is critical that pit floor and the thrust reaction structure be designed to withstand the weight of the heavy pipe segment repeatedly placed on it as well as the continuously exerted jacking loads as the operation is being conducted.

Preparation of the floor of pit, i.e., soil, stone or concrete slab will be determined by the length, size and/or duration of the job. The final alignment and grade will depend largely on the initial setup. Therefore, it is advisable to set up a concrete slab foundation for large jobs, which are likely to take a long time. The pit should have space for personnel to walk on both sides of the pipe. It is important that the pit should be dry and continuous dewatering provisions should be made.

One of the major factors that affect pipe jacking is the jacking force required to push the pipe inside the soil. Every effort is made to minimize the thrust. Application of bentonite to the outer skin of the jacking pipe reduces the friction between the jacking pipe and the soil, and reduces the thrust requirements.

The use of intermediate jacking stations (IJS) is common to control or increase the jacking forces. There is no limit to the number of IJS that can be installed in a line. The IJS permit the pipe to be thrust forward in sections rather than the total length being thrust forward from the jacking pit.

The manual pipe-jacking method is suitable for diameter up to 1500 mm. For large diameters manual jacking is not advisable as grade and alignment maintenance may not be possible in such cases. For such works, mechanical techniques like utility tunnelling / Tunnel Boring Machine (TBM) may be chosen. A typical trenchless pipe jacking method is shown in Figure 3.54.



Source:JSWA, 2011

Figure 3.54 Trenchless pipe jacking

i) Tunnel Boring / Utility tunnelling / Trenchless Technology

Tunnel boring is pipe-jacking method; but in this method, instead of manual excavation, highly sophisticated tunnel boring machines are used for excavation. It is generally used for diameter higher than 1,500 mm and where proper slope / gradient is required.

3.65.3 Illustrative Example

Illustrative example for structural design of buried sewer is given in Appendix A.3.8.

3.66 STORMWATER RELATED STRUCTURES

These are devices meant to transmit the surface runoff to the sewers in the case of combined system and form a very important part of the system. Their location and design should therefore be given careful consideration.

Storm water inlets may be categorized under three major groups viz. curb inlets, gutter inlets and combination inlets, each being either depressed or flush depending upon their elevation with reference to the pavement surface.

The actual structure of an inlet is usually made of brickwork. Normally, cast iron gratings conforming to IS 5961 shall be used. In case there is no vehicular traffic, fabricated steel gratings may be used. The clear opening shall not be more than 25 mm.

The connecting pipe from the street inlet to the main street sewer should not be less than 200 mm in diameter and should have sufficient slope. Maximum spacing of inlets would depend upon various conditions of road surface, size and type of inlet and rainfall. The maximum horizontal spacing of 30 m is recommended.

3.66.1 Curb Inlets

Curb inlets are vertical openings in the road curbs through which the storm water flows and are preferred where heavy traffic is anticipated.

They are termed as deflector inlets when equipped with diagonal notches cast into the gutter along the curb opening to form a series of ridges or deflectors. This type of inlet does not interfere with the flow of traffic as the top level of the deflectors lie in the plane of the pavement.

3.66.2 Gutter Inlets

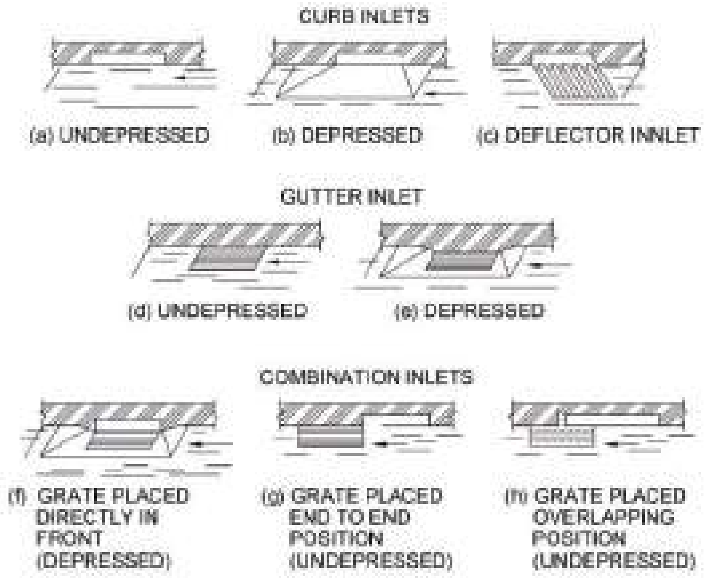
These consist of horizontal openings in the gutter, which is covered by one or more gratings through which the flow passes.

3.66.3 Combination Inlets

These are composed of a curb and gutter inlet acting as a single unit. Normally, the gutter inlet is placed right in front of the curb inlets but it may be displaced in an overlapping or end-to-end position. Figure 3.55 (overleaf) shows the different types of inlets.

3.66.4 Catch Basins

Catch basins are structures meant for the retention of heavy debris in storm water which otherwise would be carried into the sewer system. Their use is not recommended since they are more of a nuisance and a source of mosquito breeding apart from posing substantial maintenance problems.



Source: CPHEEO, 1993

Figure 3.55 Different types of inlets

Where a main sewer is laid and the sewer network is not yet laid, the dry weather flow from the open drains may be connected to the sewers by making a provision for a catch basin and overflow weir.

3.66.5 Flap Gates and Flood Gates

Flap gates or backwater gates are installed at or near sewer outlets to prevent backflow of water during high tide or at high stages in the receiving stream. Such gates should be designed so that the flap should open at a very small head differential. With a properly operated flap gate, it is possible to continue to pump a quantity equivalent to the sanitary sewage flow from the combined sewer to the treatment plant even though flood conditions prevail in the stream at the sewer outlet.

In case of a sea and estuary outfall, the outfall sewer should be able to discharge at full rate when the water level in the estuary or sea is ¾th the mean annual tide level. Adequate storage to prevent backflow into the system due to the closure of these gates at the time of high tides is also necessary if pumping is to be avoided. To control the flow from the storage tank, flood-gate or penstocks are provided which can be opened and closed quickly at the predetermined states of tide. The gates are generally electrically operated and are controlled by a lunar clock.

Many flap or backwater gates are rectangular and may consist of wooden planks. Circular or rectangular metallic gates are commercially available. Flap gates may be of various metals or alloys as required by the design conditions.

Flap gates are usually hinged by a link-type arrangement that makes it possible for the gate shutter to get seated more firmly. Hinge pins, linkages and links should be of corrosion resistant material. There should be a screen chamber to arrest floating undesirables on the upstream side of the flap gate. The maintenance of flap gates requires regular inspection and removal of debris from the pipe and outlet chamber, lubrication of hinge pins and cleaning of seating surfaces.

3.67 OUTFALL SEWERS

The aspects to be considered in the design of a sewer outfall are listed as under:

1) Location to avoid unpleasant sight and offensive smell

2) Protection of the mouth of sewer if it empties into a river against swift currents, water traffic, floating debris, heavy waves, or other hazards which might damage the structure; and

3) Prevention of backing-up of water into the sewer if the outfall is having a flat grade

3.68 CROSS INFRASTRUCTURE WORKS

Section 3.52 shall be mandatorily followed.

3.69 CORROSION PREVENTION AND CONTROL

3.69.1 General

Corrosion is the phenomenon of the interaction of a material with the environment (water, soil or air) resulting in its deterioration. There are many types of corrosion, the major types being galvanic, concentration cell, stray current, stress and bacterial. Sewage collection and treatment systems are more prone to corrosion in view of the nature of the sewage. Since sewage contains solids which are more likely to cause abrasion in sewers, pumps and their components, thus removing the protective coating and accelerating the corrosion process, corrosion control becomes all the more important in sewerage systems. It is particularly acute in areas where sewage strength is high, sulphate content of water is substantial and average temperature is above 20ºC. The corrosion problem in sewerage systems can be categorized as (1) Corrosion of sewers and (2) Corrosion of treatment systems.

3.69.2 Corrosion of Sewers

The most widely used materials for sewers are reinforced concrete, stoneware, asbestos cement and cast iron. The development of plastics, fibre glass and other synthetic materials has increased the choice of piping materials. For gravity sewers the usual practice is to use vitrified stoneware pipes for smaller sizes and cement concrete pipes for larger sizes. For pumping mains, CI pipes are generally used. Factors such as climate and topography, high temperature, flat grades and long length of sewers may favour the development of highly septic, sulphide containing sewage in the sewer line. Industrial wastes may aggravate these problems by the introduction of high concentration of pollutants and/or large volumes of hot water that accelerate chemical and biological reaction rates. Concrete sewers are the worst affected because of sulphides in sewage.

3.69.3 Corrosion due to Biological Reactions

Hydrogen Sulphide may be produced biologically in sewers by (1) the hydrolysis of organic compounds containing sulphur and (2) by reduction of sulphates. Sewage contains a variety of sulphur bearing organic compounds (usually at concentration between 1 to 5 mg/l) and inorganic sulphates, which find their way through drinking water, industrial water or sea water intrusion.

Hydrogen sulphide in sewer is usually produced by bacteriological reduction of sulphates. Hydrogen sulphide gas by itself is not injurious to cement concrete, unless it gets readily oxidized by dissolved oxygen or by several bacterial species.

Oxygen is normally present in the air between the crown and the sewage, H_2S, a prerequisite for sewer corrosion and CO_2, are usually present in the sewer air. In the presence of air, H_2S gets oxidized to sulphuric acid and this sulphuric acid reacts with the cement constituents of concrete. In fact, it reacts with the lime in the cement concrete to form calcium sulphate, which in turn, reacts with the calcium aluminates in the cement to form calcium sulpho-aluminates.

Expansion caused by these reactions results in spalling of the surface of the concrete, thereby exposing underlying layers of concrete to further attack. If the corrosion products adhere to the surface of the concrete, then a certain measure of protection against further acid attack is provided. Sulphuric acid, in fact, does not and cannot penetrate into normal concrete.

Acid attack therefore takes place at the surface only. The most outstanding character of this form of corrosion is the fact that it only occurs above the water line in the sewer. In other words, it is the crown portion of the pipe, which gets corroded and this phenomenon is referred to as crown corrosion. Due to this corrosion, the reinforcement gets exposed and the sewer gets damaged. In general, synthetic material pipes are not directly affected by biological corrosion.

3.69.4 Factors Influencing Sulphide Generation

The factors that influence sulphide generation in sewers include: (i) temperature of sewage, (ii) strength of sewage, (iii) velocity of flow, (iv) age of sewage, (v) pH of sewage, (vi) sulphate concentration and (vii) ventilation of the sewer.

3.69.4.1 Temperature

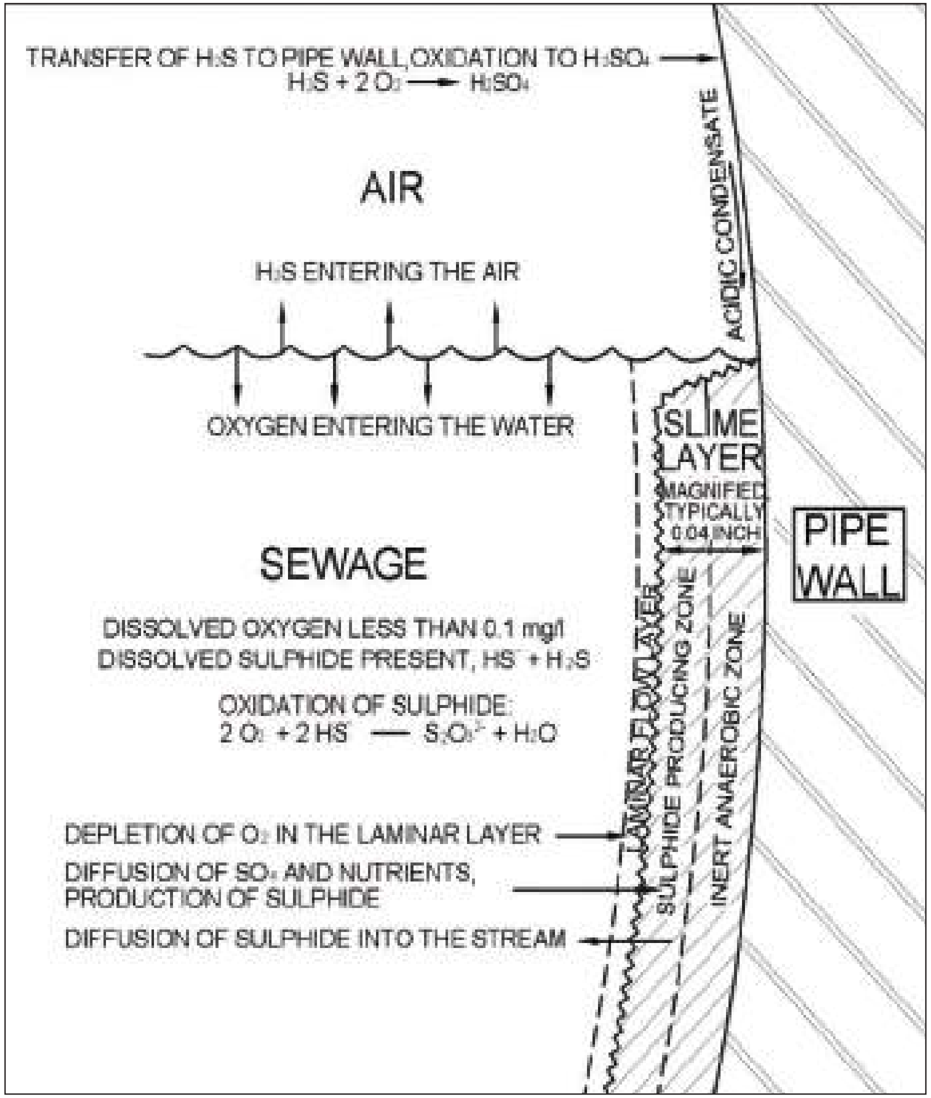
Since sulphide generation is a biological phenomenon, it is obvious that sewage temperature influences the rate of sulphide generation. Temperature below 20°C generally will not cause any appreciable sulphide build up. From 20°C to 30°C, the rate of sulphide generation increases at about 7% per °C rise in temperature and is maximum at 38°C.

3.69.4.2 Strength of Sewage

A high concentration of organic strength (BOD) in sewage will lead to an increased rate of sulphide generation as in Figure 3.56 and Figure 3.57 overleaf.

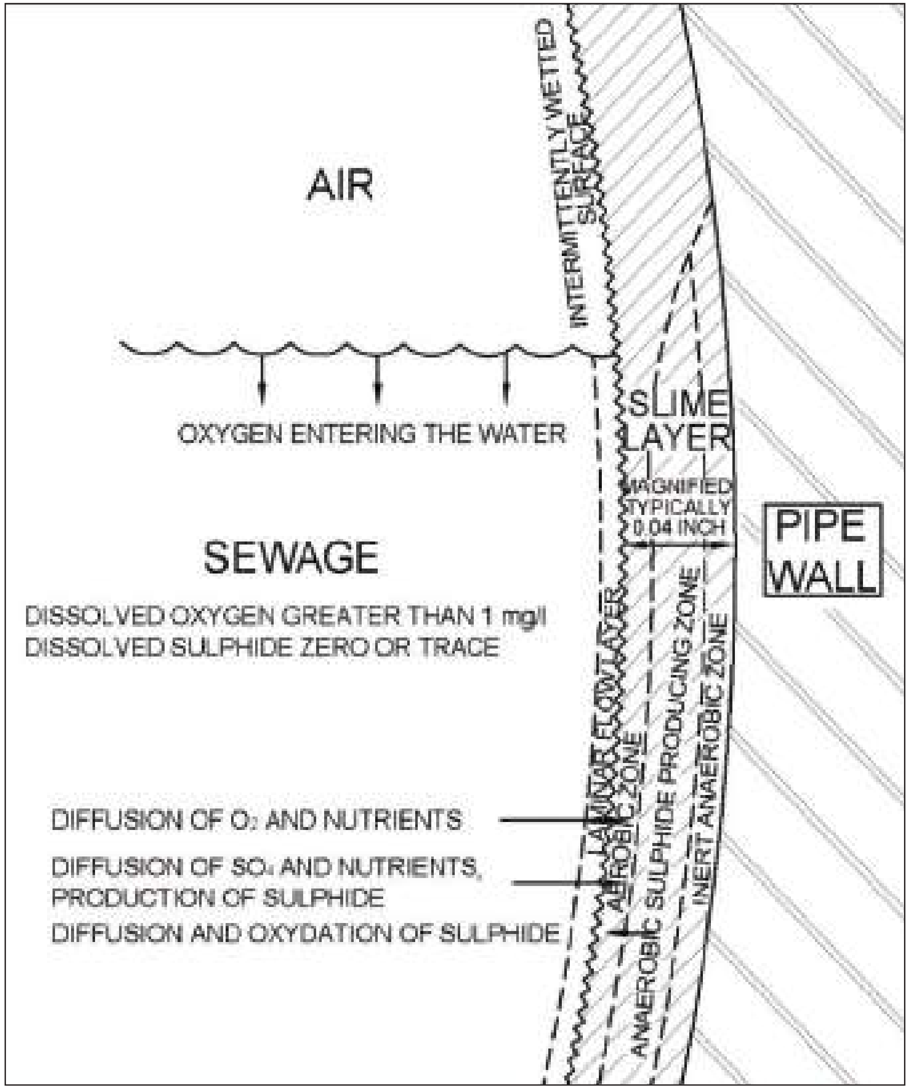
For any specified sewage temperature and flow condition in a sewer, there is limiting sewage strength, usually less than 80 mg/l of BOD, below which a build-up of hydrogen sulphide will practically cease.

However, it is possible in a long force main or at other locations where oxygen is shut off from the sewage for a few hours, that sulphide build up may occur even with low values of BOD.



Source: USEPA, 1974

Figure 3.56 Sulphide gas equilibrium in negligible oxygen conditions in sewers

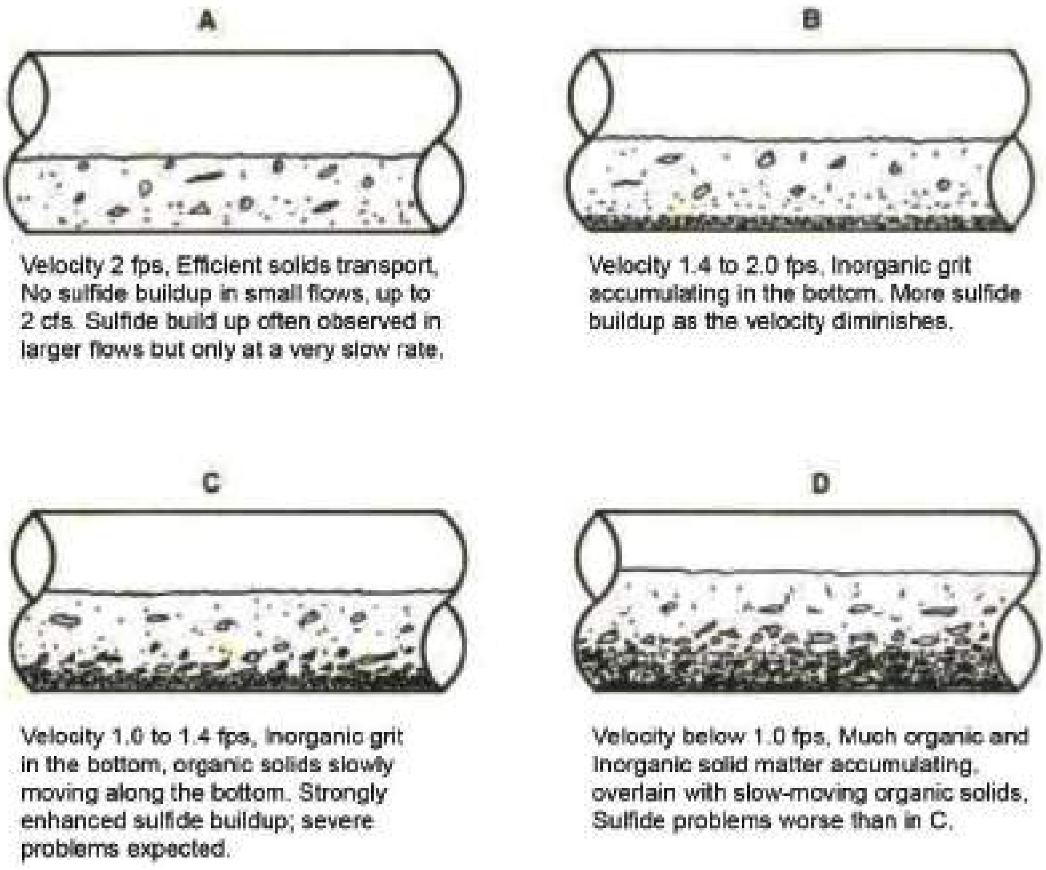


Source: USEPA, 1974

Figure 3.57 Sulphide gas equilibrium in appreciable oxygen conditions in sewers

3.69.4.3 Velocity of Flow

The velocity should be both self-oxidizing and self-cleansing. If the velocity of flow is great enough to keep the submerged surfaces of the sewer free from slimes, no generation of H_2S will occur. Whenever the velocities are too small, the organic materials get settled out and undergo anaerobic decay and release the sulphide, which later combines with the moisture and forms sulphurous and sulphuric acid. The effect of velocity and relative sedimentation of organics and grit is shown in Figure 3.58.



Source:USEPA,1974

Figure 3.58 Solids accumulation at invert of mains at various velocities

This incidentally brings out the fact that under prolonged conditions of absence of the minimum velocities and the absence of the high velocities associated with the peak flow conditions at least once a day, the effective area of the pipe is progressively reduced. This affects the gravity sewers by backing up of the sewage upstream and reduced pumping in pumping mains resulting from higher heads needed to pump the same volumes of sewage in reduced bores of the pipe.

The velocity necessary to prevent the build-up of sulphides in flowing sewage corresponding to different values of the effective BOD (BOD_T) are shown in Section 3.15.2.

In determining the velocity to be used in design, the effective BOD should be calculated for the period of the year, which gives the maximum value.

3.69.4.4 Age of Sewage

The oxidation-reduction potential of sewage which in turn is influenced by the age of sewage, seems to be one of the important factors contributing to sulphide build up in the lower reaches. When septic sewage is discharged from a collecting system, an Imhoff tank, or from a septic tank into an outfall, it should be treated before it goes into the sewer. When outfall grades are steep, the problem is particularly acute since high turbulence can release the sulphides causing odour and corrosion problems. Long detention times in forced mains greatly influence the generation of sulphides.

The possible sulphide build up in a filled pipe can be roughly estimated as:

$$\Delta C_s = 0.066\, t\, BOD_T \frac{(1 + 0.0004\, d)}{d} \qquad (3.47)$$

where,

C_s : Increase of Sulphide concentration in the force main in mg/l

t : Detention time in the main in minutes

d : Pipe diameter in mm

3.69.4.5 Hydrogen Ion Concentration

Sulphide producing organisms are known to have a considerable adaptability so that pH value is not likely to have much effect on the rate of generation in sewers within the pH of 6 to 8. If the pH value is above 9.0 or below 5.5, sulphide generation will be affected.

3.69.4.6 Sulphate Concentration

The more the concentration of Sulphate, the more is its reduction to H_2S.

3.69.4.7 Ventilation

Ample ventilation through sewers will help in carrying away the generated H_2S, supply additional oxygen to the sewage and keep the walls free of moisture and reduce the tendency for sulphuric acid formation and attack of concrete.

Ventilation is particularly important in locations of turbulent flow, either by better natural ventilation or by forced ventilation by fans, one or more of the necessary factors for optimal bacterial activity can be made limiting. However, it is often very difficult and expensive to provide enough ventilation to prevent corrosion.

3.69.5 Sulphide Control Procedures

The following are some of the criteria that may be taken into account in preventing or controlling sulphide build up and consequent odour and/or corrosion.

3.69.5.1 Design of Sewers for Sulphide Corrosion Issue

In the design of sewer systems, consideration should be given to the desirability of maintaining velocities sufficient to avoid sulphide build up and of minimizing pressure lines and points of high turbulence. The designer should take into consideration topography, grades of sewers, ventilation, materials of construction, sewage temperature and strength, etc.

Some of the design features that should be considered are described below.

One of the important factors in the control of H_2S is the velocity of flow and BOD. Please refer Sections 3.15.2 for BOD to prevent H_2S in sewers. The limiting velocities for prevention of sulphide generation vary with temperature and effective BOD. The velocities given in Section 3.15.1 are believed to be the minimum that should be used. An allowance of 25% in the velocity should be made as a factor of safety and if industrial wastes are present with a higher content of dissolved organic matter, it may be necessary to increase this allowance to 50%. Where it is impractical to provide a sewer gradient in design to give these limiting velocities, other means of controlling sulphide generation should be considered. Velocities giving high, single point turbulence, may however, result in sulphide release and severe odour and/or corrosion.

Except in the cases where sewage is quite weak and in a fairly well aerated condition, high sulphide generation because of large slime areas can be expected in completely filled sewer lines. Force mains, therefore should be kept to a minimum or velocities must be adequate at all flowing times.

Since biological activity is concentrated largely in the slime layer, it increases with an increase of the wetted perimeter. The oxygen uptake is proportional to the surface width of the stream. Therefore, it follows that deep flow in a pipe is more conducive to sulphide generation than shallow flow. Accordingly, where sulphide generation is a critical consideration, a larger pipe is always better than a smaller one for any given slope and sewage flow.

Turbulence caused by high velocities for short distances or improper design of junction manholes permitting sewage lines to intersect at right angles or at different elevations should be avoided as turbulence can cause excessive release of H_2S even where sewage contains only a small amount of dissolved sulphides.

Concrete with a low water-cement ratio of suitable workability, thorough mixing, proper placing and sufficient curing is preferred for sewers.

3.69.5.2 Control of Sewage Character for Sulphide Corrosion Issues

Trade wastes containing dissolved sulphides should not be allowed into the sewers. High sulphate concentrations arising from the discharge of tidal or sea-water to the sewer should be controlled. The oxidation-reduction potential of the sewage can be increased and the rate of generation of H_2S slowed down by steps, which include the partial purification of sewage allowed into the sewers by sedimentation or by high rate treatment on filters. Effective BOD of sewage depends upon sewage strength and temperature. By reducing sewage strength and/or temperature, effective BOD as well as minimum velocity required can be reduced.

Strength of sewage can be reduced in some cases by diluting sewage with unpolluted water. It must be realized, however, that dilution reduces the waste-carrying capacity of the sewer.

Where velocities are inadequate to control the formation of H_2S or where completely filled lines are encountered as in force mains, supplemental aeration by the use of compressed air may be desirable. Air injection would prevent hydrogen sulphide building up and in any case will greatly reduce generation.

Air addition at about 10 lpm for each cm of pipe diameter is necessary. Care must be taken to prevent the formation of air pockets in such lines, since experience has shown that some H_2S will form on the walls at the points of such air pockets and corrosion will occur.

3.69.5.3 Cleaning of Sewers for Sulphide Corrosion Issues

Removal of slime and silt has the effect of reducing sulphide generation. Periodic cleaning of sewers by mechanical or chemical means is necessary. Any partial blocking of the sewer by debris will result in retardation of flow and consequent anaerobic decomposition of deposited sludge. Periodic mechanical cleaning and flushing of sewers can reduce average sulphide generation by 50 %. A good continuing programme of mechanical cleaning is probably the foundation for any control programme.

Sulphuric acid is effective in reducing slimes. Intermittent use of sulphuric acid was found to be useful in removing slimes on the submerged walls. Caution must be exercised in the use of sulphuric acid for the purpose of acidification since iron sulphide, that may be present on sewer walls, may cause an initial release of H_2S sufficient to be fatal to any worker inside the sewer. The shift of pH value also changes all the ionized sulphide (in the flow) to H_2S.

Slaked lime, $Ca(OH)_2$ is probably more suitable for chemically treating the slime since no corrosion damage will result from it and sulphide release will not occur. It has been found that if the slimes are subject to lime slurry of about 8,000 mg/L for 45 minutes, they will be inactivated for periods of 3 to 14 days depending upon flow and sewage characteristics.

3.69.5.4 Chlorination for Sulphide Control Issues

Chlorine has been successfully used in controlling sulphide generation for many years. Chlorine is effective in three ways (i) it destroys sulphides by chemical reaction, (ii) it reduces biological activity and produces mild oxidizing compounds in the sewage and (iii) it destroys the slimes.

An approximate dosage of 10 to 12 mg/L of chlorine is sufficient. When excess chlorine is applied, it leaves the sewage in an oxidized state, and prevents the re-appearance of sulphide for some distance downstream.

3.69.6 Materials of Construction for Sulphide Corrosion Issues

When corrosion cannot be prevented by design, maintenance or control of wastes entering the sewer, consideration must be given to corrosion resistant materials such as vitrified-clay or to protective linings of proven performance.

Plastic pipes may also be used if accepted in all other respects. It is possible that super sulphated metallurgical cement, pozzolana-portland cement mixtures or portland cement low in tricalcium aluminate may be more resistant to attack than normal portland cement.

On concrete pipe, extra wall thickness (sacrificial concrete) sometimes is specified to increase pipe life in the event corrosive conditions develop. On reinforced concrete, this takes the form of added cover over the inner reinforcing steel.

Another method of modifying the composition of concrete is by the use of limestone or dolomite aggregate in the manufacture of the pipe materials. The use of such aggregates increases the amount of acid-soluble material in the concrete, which prolongs the life of the pipe in corrosive environments. The rate of acid attack of limestone or dolomite aggregate pipe may be only about one fifth as great as when granite aggregate is used.

Unfortunately, not all limestone and dolomite aggregates exhibit the same resistance to this form of corrosion. Accordingly, tests should be made before limestone or dolomitic aggregate is used. Aluminous cement has initial resistance to acid attack. Its corrosion products are also not extensive. Therefore, it may have limited use in sewer structures.

3.69.7 Sewer Protection

Protection of sewer structures by lining or coating against H_2S attack can also be considered if other methods of control are impracticable.

3.69.7.1 Liners

A plastic polyvinyl chloride sheet, having T-shaped protections on the back, which key into the pipe wall at the time of manufacture, is one of the successful lining materials. Vitrified clay of low porosity has also been used as a liner. In regions where high sulphides and high production of H_2SO_4 can be expected, the problems remain.

Cement mortar joints are subject to attack. Bituminous joints are emulsified and dissolved by soaps, oil and grease. Acid-proof cement joints offer the best protection, but they are costly. Some type of plastic coatings and/or linings for sewers and other structures have proved moderately successful, given continued inspection and maintenance.

The function of these linings is to isolate the concrete from the corrosive atmosphere. To be effective, the lining including joints must be sealed completely to protect the sewer system throughout its expected life.

The interior of cast iron and ductile iron pipe usually is lined with cement mortar. Steel pipe sometimes is lined similarly. Smooth-walled steel pipe also may be protected by cementing plasticized polyvinyl chloride sheets to the pipe and sealing the joints.

Corrugated metal pipe may be coated inside and out with bituminous material. For added protection, asbestos fibres may be embedded in the molten zinc before it is bituminous-coated (asbestos bonded). Such coatings should be of impermeable material of sufficient thickness and free of flaws such as pin-holes.

3.69.7.2 Protective Coatings

Any protective coating used should possess the following qualities; (i) it should be resistant to acid attack, (ii) it should bond securely to the concrete, (iii) it should be economical and durable, (iv) it should be resistant to abrasive action by flow of sewage, and (v) when applied, it should be thin enough to fill all pores and irregularities in the surface. The coating should be continuous with no pin holes or other breaks. Figure 3.59 presents a RCC sewer pipe with coating.

Figure 3.59 RCC Sewer pipe with protective coating

The effectiveness of a coating thus depends on its inherent resistance to acid attack and on its ability to form impervious membrane. In practice, no coating can be applied without discontinuity. Inspection and maintenance must be periodical.

Plastic-based paints and coal tar epoxy coatings have proved to be good.

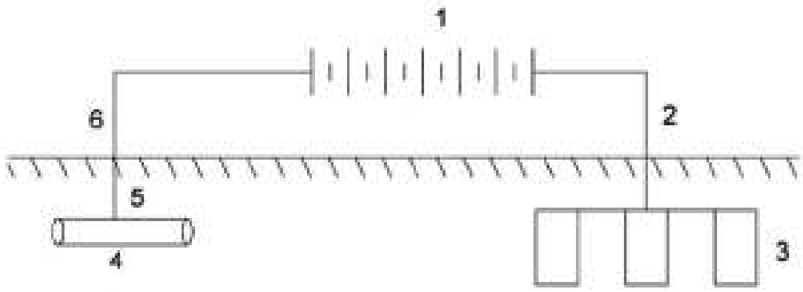
3.69.7.3 Cathodic Protections

Cathodic protection is the application of electricity from an external power supply or the use of galvanic methods for combating electrochemical corrosion.

Cathodic protection should be used as a supplement and not as an alternative technique to other methods of protection. It may be a more suitable and expeditious method of protection for existing pipelines.

a) Basic Principle

The basic principle is to make the entire surface of the equipment cathodic, thus affording protection, since corrosion takes place only at the anodic surface. This can be achieved by connecting it to a DC source. In this case, the anode consists of specially earthed electrodes. The general arrangement in a cathodic protection assembly is shown in the Figure 3.60 overleaf.



Source: CPHEEO, 1993

Figure 3.60 General arrangement of cathodic protection

The current from the positive pole of the DC source flows through the conductor 2 into the earthed anode 3 and then into the soil. From the soil the current flows to the surface of the pipe 4 to be protected and flows along the pipe to the drainage junction point 5, the conductor 6 and back to the negative terminal of the current source. Thus, the entire surface of the underground pipe or equipment becomes cathodic and is protected from corrosion, while the earthed anode gets corroded. The anode, is usually scrap metal e.g., old tubes, rails, etc. Other metals, which are resistant to attack by surrounding soil like special alloys or graphite, are also used. The conductivity of the protective coating has a direct influence on the length of the protected section of the pipe. The required power increases with increasing conductivity of the coating.

b) Preliminary Investigations

The existing pipeline has to be inspected to ascertain the sections that require protection. Other basic information required are as follows:

a) Plan and details of the pipelines (showing branch connections, diameter, length and wall thickness) and location plan of the section to be protected along with

i) Data on soil resistance along the section to be protected at the intervals of at least 100 m as well as the earthing points

ii) Information on the availability of sources of electricity, amperage, voltage, DC/AC (phase) in the vicinity and spaces for housing current supply and controls

iii) Data on the conductivity or resistivity of the existing protective insulation; and

iv) Condition of the pipeline, if it is already in use

c) Power requirements

With the above data, minimum current density and maximum protection potential can be worked out. The capacity of the current source for a cathodic protection system depends on (1) length of the section to be protected (2) type and state of the coating of the pipeline (3) diameter of the pipe (4) wall thickness of the pipe (5) conductivity of the soil and (6) design of anode earthing. The power requirements vary from 0.4 to 10 kilowatts in most cases. The possible current sources are DC Generator, converter-rectifier, storage batteries of dry or acid type. The pipeline should be at least 0.3 V negative to the soil.

d) Anodes

The main power loss occurs in the anode earthing. The earthing can be carried out by any metal (pure or scrap) of any shape and carbon forms like coke or graphite. When tubes are used, the earthing can be either horizontal or vertical. Near the earthing zone, soil treatment can be done to reduce soil resistance by adding salts like sodium chloride, calcium chloride or moistening the soil, the former being better and long lasting. Carbon or graphite electrodes have longer durability than metal electrodes.

e) Other facilities

A cathodic protection station should provide space for housing the equipment, installation of current sources, supply and distribution zones, equipment for check measurements, construction of earthing structures and facilities for carrying out operational tests.

3.69.7.4 Protection by Sacrificial Anode

Sacrificial anodes serve the same purpose as the cathodic protection system but do not require external electric power supply. The required current is supplied by an artificial galvanic couple in which the parts to be protected, usually iron or steel, is made as the cathode by choosing the other metal having the higher galvanic potential, as the anode. Zinc, Aluminium and Magnesium (with sufficient purity) or their alloys, which are higher up in the galvanic series must be used for this purpose. Sheets of zinc suspended in a coagulation basin are an example. A single protector anode will not be sufficient and it will be necessary to install a number of such anodes generally spaced at 4 to 6 m in the pipeline or the structures to be protected.

The performance and service life of anodes depend mostly on the nature of soil or water surrounding them. Use of fill materials in the soil such as clay and gypsum powder results in low resistance of anode earthing and yields a high current. The costs of protection by galvanic anode would be appreciably higher in the case of pipeline networks in big towns since it would be necessary to suppress incidental contacts.

For the application of galvanic protection, the resistance of the soil should be less than 12,000 ohm-cm. A higher resistance of the circuit can achieve neither the required current density, nor the reduction of the pipe to soil potential. In such cases, cathodic protection by means of external power supply offers better protection.

The following measures are also of interest in minimizing corrosion:

i) Minimizing point of high turbulence within the system thus resulting in less sulphide generation

ii) Designing wet wells to preclude surcharge of tributary lines which also result in less sulphide generation

iii) Provision of forced ventilation at a point where air may be depleted seriously of its oxygen

iv) Using a coating of another metal such as zinc, galvanized iron or using paints appropriately

v) Gas Scrubbing

vi) Providing inside sleeving or lining of suitable type of plastic materials

The problem of sewer corrosion due to hydrogen sulphide production and its control is a serious one to the sewage conveyance system. Prevention of H_2S generation by proper design and continued cleaning of sewers seems to be the best available method.

3.70 CONNECTION OF HOUSE SEWER TO PUBLIC SEWER

The earlier practice has been to connect house sewers to public sewers using the typical Y branch or T branch depending on the depth of public sewer. The reason for this is that stoneware pipes had these specials and can be inserted wherever needed while laying the sewers. Most of the problems of sewer blocks are traceable to solid materials getting stuck at “T” or “Y” junction in house services, requiring most times even cutting open the roads. It is henceforth proposed to discontinue this practice. The house-service sewer connections shall be effected only in manholes. In case of old sewers, a new manhole shall be inserted for this purpose. The material of the house service sewer shall be either conventional salt glazed stoneware or UPVC rigid straight pipes of 6 kg/cm2 pressure class in manufacture and as per IS 15328 with solvent cement joints.

The minimum earth cover above the crown of the sewer shall be mandatorily 90 cm and where this becomes impossible; the property owner shall be directed to depress his terminal chamber to comply with this minimum earth cover of 90 cm, as the public manhole shall start at its crown at 100 cm below ground level (see also section 3.20.3). Where such sewers cross the electricity power cables, the specifications of IS 1255 clause 6.3.3 and clause 6.3.3.1 shall be mandatorily followed without any exception. All such house service sewers shall be only above the electricity power cable and the minimum clearance shall be 30 cm all-round the electricity power cable. The electric power cable itself shall be covered all around by 15 cm riddled soil and further protected on top by tiles, bricks or slabs. Hence, the total minimum clearance will be 30 cm + 15 cm = 45 cm.

The house owner shall be mandated to possess a “kraite” a type of non-corroding, sufficiently flexible but rigid type of less than 10 mm diameter rod, which he/she shall use to rod the house-service sewer freely up to the manhole. The labour of the local body shall not be deployed for any removal of obstructions in the house-service sewer. Typically, it is possible to effect six service connections to a manhole. Without exception, the provision of terminal chamber inside the property premises shall be mandatorily followed.

3.71 SPACING OF MANHOLES

Sewers are known to get choked resulting in sewage overflow from upstream manholes. The non-invasive and non-destructive cleaning is by equipments like jetting machines, bucket machines, rodding machines etc. At the same time, however, house-service sewers are also known to get choked and sewage backs up into the houses. The reasons in this case are

(a) the choking of public sewers and manholes in road portion or

(b) obstructions in the house sewer itself due to extraneous material pushed into it by the residents.

The problem at

(a) can be relieved when the public sewer is cleared up.

(b) however requires clearing the house-service sewer.

In the older practice of house service sewers joining the public sewer through "Y" or "T" junctions, this is difficult and invariably, the road is dug up at the junction to break the house service sewer, clear it up and join back by covering with a curved tile or sleeve etc. Even then, the choking can recur and the practice is to be repeated resulting in the weakening of the service sewer itself to withstand the load from the road.

The Chennai Metropolitan Water Supply and Sewerage Board (CMWSSB) have been successfully implementing for over two decades, the practice of connecting all house service sewers to manholes and not public sewers. The precaution used is such connections are effected below the corbel portion of the manholes. Typically, a manhole takes six such connections, three from each side of the road, from properties opposite to and on both sides of the manhole. The clearing of the house-service itself seldom arises because even the extraneous matter pushed in by the residents gets "dropped" into the manhole and if at all noticed, the simple rodding of the house sewers by bamboo splits or flexible rods succeeds in clearing the blockages and drop them into the manholes. Thus, the problem of the road getting dug up frequently causing nuisance in the public is completely avoided.

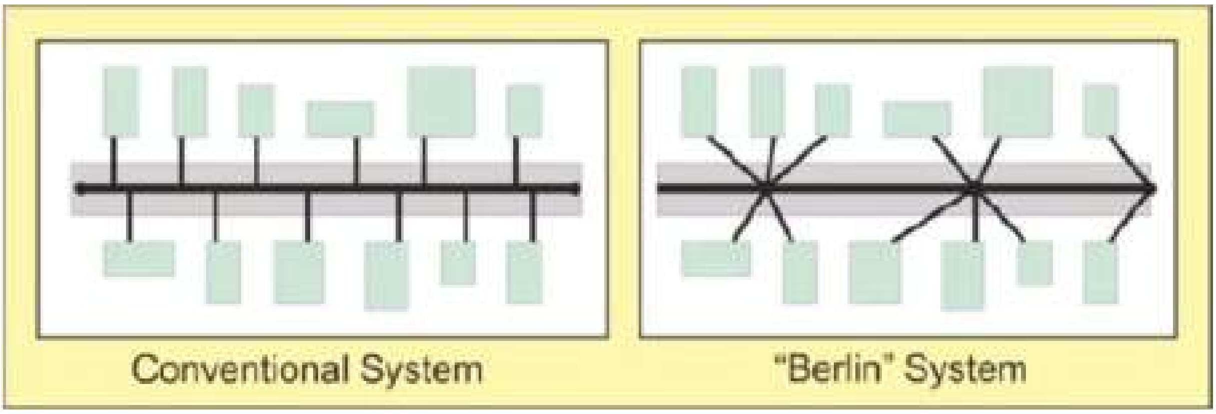
From this point of view, it becomes necessary to limit the length of such house service sewers. Considering that typically properties are developed on plots of about 10 m width, a spacing of 30 m between manholes permits both the objectives of easy cleaning of public sewer stretches and eliminating the avoidable road digging to clear obstructions in house service sewers. Accordingly, the spacing of manholes shall be retained at 30 m as in the existing manual and in case of economically weaker sections, the spacing can be narrower, commensurate with the width of the plots so that all connections are made to the manholes without undue lengths.

In addition, there will be additional manholes at changes of directions and in the case of commercial structures like meeting halls, marriage halls etc as the case may be. Very wide plots should be encouraged to avail manholes at each end of the plot by meeting the cost of the extra manhole. In the case of gravity outfall sewers with no house service sewers, the spacing can be at 60 m besides at every change of direction and drops. For insertion of the house service sewers into the manholes, it is necessary to have a precast ring section below the corbel portion with holes at 45 degrees to the public sewer line to facilitate insertion of three house service sewers on each side of the road.

Usually the house service sewers shall be 110 mm or 160 mm UPVC 4 kg / sqcm (as detailed in sewer laying section). Accordingly, the height of the ring shall be 250 mm and 300 mm to permit filling of the annular the interspaces between the sewer and the opening with cement concrete and at least 50 mm of RCC annular fill around the inserted house service sewer respectively. Without exception, the provision of terminal chamber inside the property premises shall be mandatorily followed.

3.71.1 In Public Sewers

The CMWSSB are connecting all the house service sewers directly below the corbels of manholes for over two decades. Typically, a manhole takes six such connections, three from the properties on each sides of the public sewer alignment. The clearing of the house service itself seldom arises because even the extraneous matter pushed in by the residents gets “dropped” into the manhole. Where needed, simple rodding of the house sewers by bamboo splits or flexible rods from the terminal chamber of the property clears the blockages and drop it into the manholes. Thus, the problem of the road getting dug up is completely avoided. Such a system is also referred to as the “Berlin System” as in Figure 3.61 as cited by the EPA in its publication quoted here.



Source: EPA, 2010

Figure 3.61 Connection system between house service sewer and public sewer

Considering that the typical road frontage of plots in ULBs at about 10 m and the road widths being two lanes with 6 m to 8 m width, a spacing of 30 m between manholes permits restricting the lengths of house sewers between 3 m in the shorter perpendicular direction and 15 m in the longer hypotenuse direction. These are reasonable for maintenance and cleaning. Accordingly, the spacing of manholes shall be 30 m. In the case of economically weaker sections, the spacing can be lesser, commensurate with the width of the plots. The standard provision of additional manholes at changes of directions will continue. In the case of meeting halls, marriage halls etc as also very wide plots these should be encouraged to avail manholes at each end of the plot by their meeting the cost of the extra manhole.

3.71.2 In Outfall or Trunk Gravity Sewers

In outfall or trunk gravity sewers with no house service connections, the diameter of the sewers will also be larger as 1.5 m in diameter. The Japanese manual specifies as in Table 3.30 overleaf.

The WEF states the spacing of manholes as “Manholes are provided in sewer systems to help maintain and clean sewer pipes. Typically, they are provided at intersections of two or more mainline sewers, at changes in direction of sewer lines, and at regular intervals along a mainline. Manholes are typically spaced approximately 300 feet (91.4 meter) apart, but can be less than 100 feet (30 meter) or as far apart as 500 feet (152 meter) (EPA)”.

Table 3.30 Spacing of manholes as per Japanese Manual

Sewer Diameter (mm)	600 or less	1000 or less	1500 or less	1650 or more
Maximum Manhole Space (m)	75	100	150	200

Source: JSWA

Considering the involved aspects of costs, frequency of cleaning and financial sustainability of sewer cleaning equipment by the ULBs, the following in Table 3.31 is now recommended. It is always possible to insert a manhole later on in between the existing manholes if another sewer or house service sewer is to be connected at that time.

Table 3.31 Spacing of manholes in gravity sewers not receiving house service sewers

Sewer Diameter (mm)	Up to 600	600 to 900	900 to 1200	1200 to 1500
Maximum Manhole Space (m)	60	90	120	150
Maximum Manhole Space (m)	30 m or it can be less than 30 m depending on the street			

Where sewer diameters exceed 1500 mm, the possibility of using egg shaped sewers made out of pre-cast RCC sections made out of sulphate resisting cement and duly plastered on both sides should be explored for easier execution of work and better control over the flushing and cleaning at low flows in the bottom egg shaped segment. The geometry of the egg shaped sewer and its hydraulic properties at full flow are shown in Figure 3.62. The characteristics of flow shall be referred to from standard texts.

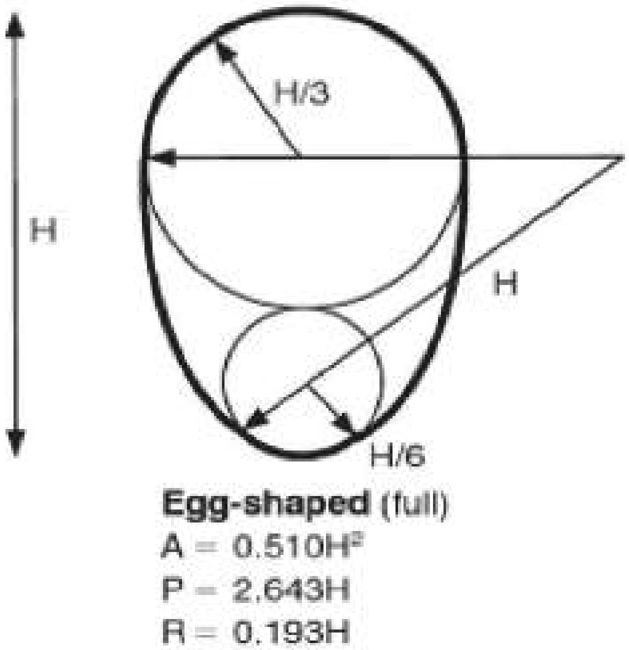


Figure 3.62 Geometry and hydraulics of egg shaped sewer at full flow

3.72 LIFT STATIONS IN GRAVITY SEWERS

There are cases where high water table conditions or rocky strata pose considerable difficulties in the design and provision of conventional gravity sewerage system in that excavations amidst sub soil water or rocky terrain is not only difficult but also is frowned upon by the public when the works drag on and on in the middle of the road. Such situations can be easily got over by restricting the depth of sewers to a practicable limit and diverting the flow into a pavement submersible pump station with a lockable control panel there itself. This is similar to the pillar boxes of the electricity board and the delivery main can lift the flow to the downstream manhole at the conventional 0.9 m depth to invert.

With the availability of quite a few manufacturers of sewage-submersible pump sets in the country it should be possible to implement this instead of struggling with deep sewers in such areas for years together and more importantly compounding the problems of O&M and the repairs at these depths perpetually. These submersible pump stations can be operated by mercury float switches and powered by dedicated feeder lines from the local electrical authority similar to the lines given to the hospitals, etc. These pump sets can also be connected to solar panels. The pump pit can be covered with pedestrian grade walkway slabs, which are of RCC and with adequate lifting arrangements to permit the lowering and lifting the submersible pump sets. More details on lift stations are available in Chapter 4.

CHAPTER 4 DESIGN AND CONSTRUCTION OF SEWAGE PUMPING STATIONS AND SEWAGE PUMPING MAINS

4.1 GENERAL CONSIDERATIONS

Pumping stations handle sewage either as in-line for pumping the sewage from a deeper sewer to a shallow sewer or for conveying to the STP or outfall. They are required where sewage from low lying development areas is unable to be drained by gravity to existing sewerage infrastructure, and / or where development areas are too remote from available sewerage infrastructure to be linked by gravity means.

4.1.1 Design Flow

Refer Chapter 3 of this manual.

4.1.2 Location and Configuration

The proper location of the pumping station requires a comprehensive study of the area to be served, to ensure that the entire area can be adequately drained. Special consideration has to be given to undeveloped or developing areas and to probable future growth. The location of the pumping station will often be determined by the trend of future overall development of the area. The site should be aesthetically satisfactory. The pumping station has to be so located and constructed such that it will not get flooded at any time. The storm-water pumping stations have to be so located that water may be impounded without creating an undue amount of flood-damage, if the flow exceeds the pumping station capacity. The station should be easily accessible under all weather conditions. Pumping stations are typically located near the lowest point in a development. However, the siting and orientation of each pumping station shall be considered individually and based on the following criteria:

- Local topography as slope of the ground and above and below ground obstructions
- Proposed layout of the particular development and of future developments
- Proximity of proposed and/or existing sewerage infrastructure
- Size and type of the pumping station
- Access considerations for O&M needs including operators health and safety issues
- Visual impact, particularly the vent tube, odours, noise problems, etc.,
- Availability of power, water, etc.,
- Vulnerability of the site for inundation
- Compatibility to neighbouring residences by suitable dialogues.

Of these, the inundation is the key and can result in major environmental and health problems in case raw sewage is flushed to the surface due to flooding of the wet well, or because of failure of the system due to a partially/fully submerged switchboard. Inundation may also result in severe scouring around structures, particularly around the wet well, valve chamber, and possibly cause damage to the critical components such as the electrical switchboard. Accordingly, the designer shall establish the levels of the top of the wet well wall, top of valve chamber walls and top of the plinth supporting the electrical cubicle, so that those structures cannot be inundated by a flood of a 1 in 100-year recurrence interval.

Preferred method will be the formed ground level to be at the 1 in 100 -year flood level and building plinth and top of wet wells etc. shall be at 0.45 m above.

Ditch drain shall be mandatorily provided all around and if it is not possible to drain by gravity to the nearby natural drain. Drain pump sets shall be installed with 100% standby to pump out rain water and connected to the standby power. Rain-water harvesting shall not be provided in sewage pumping stations to avoid ground water pollution by raw sewage due to accidental spillage.

Minimum number of wet wells shall be two, irrespective of the volume of sewage to be pumped out and the structures shall be as far possible circular in plan to facilitate simpler and economical construction, besides the possibility of removing accumulated grit from one of the wells at a time without interrupting the pumping out.

4.1.3 Measures for Safety and Environment Protection

1. Railing shall be provided around all manholes and openings where covers may be left open during operation and at other places, where there are differences in levels or where there is danger for people falling.
2. Guards shall be provided around all mechanical equipment, where the operator may come in contact with the belt-drives, gears, rotating shafts or other moving parts of the equipment.
3. Staircases shall be provided in preference to ladders, particularly for dry well access. Straight staircases shall be provided as against spiral or circular staircases or steps. The steps to be provided in the staircase shall be of the non-slippery type.
4. Telephone is an essential feature in a pump-house, as it will enable the operator to maintain contact with the main office. In case of injury, fire or equipment-difficulty, the telephone will provide facility to obtain proper assistance as rapidly as possible.
5. Fire extinguishers, first-aid boxes and other safety devices shall be provided at all SPS.
6. A system of colours for pipes shall minimize the possibility of cross-connections.
7. To prevent leakages of explosive gases, the wet well should not be directly connected by any opening to the dry well superstructure.
8. All electrical equipment and wiring should be properly insulated and grounded and switches and controls should be of non-sparking type. All wiring and devices in hazardous areas should be explosion-proof.

9. All pumping stations should have potable water supply, washroom and toilet facilities and precautions taken to prevent cross connections.

10. Hoisting equipment shall be provided for handling of equipment and materials, which cannot be readily lifted or removed by manual labour. Hence, in large pumping stations, gantries of adequate capacities shall be provided to lift the pumps, motors and large piping.

11. Fencing shall be provided around the pumping station to prevent trespassing.

12. The station should be landscaped to make it blend with the surroundings and to add to the aesthetic effect, particularly when residential areas are in the near vicinity of the station.

13. Adequate lighting is essential at the plinth and all locations of the pumping station and glares and shadows shall be avoided in the vicinity of machinery and floor openings.

4.1.4 Design Suction Water Level

The suction elevation should be preferably below the invert of the incoming sewer to facilitate air passage through the sewer in the reaches closer to the pump station. A preferable drop of 50 cm to 100 cm below the invert of the incoming sewer is desirable to safeguard against problems of choking of sediments in sewers due to stagnations.

4.1.5 Design Discharge Level

The water surface elevation in the receiving structure decides the static lift when compared to the suction level. However, friction losses and free-fall at receiving chamber are to be added to this to get at the design discharge level. As a rule, if needed this has to be increased such that the hydraulic grade line does not cut the longitudinal section of the ground level along the pumping main. This is achieved by raising the discharge elevation by means of a raised delivery line ending up in a goose-neck before dropping the flow into the receiving chamber such that the hydraulic grade line moves upwards in its terminal end and thus becomes free of the ground level.

The hydraulic grade line shall be at least 1 m above the highest ground level or the top most crown of the pumping main.

4.1.6 Selection of Power Source

The power source will be the local electricity grid. A dedicated feeder from the nearby substation is recommended and in large pumping stations two such independent dedicated feeders from two different substations is to be considered. Drawing off a nearby power cable is permissible for small pumping stations handling less than 1 MLD of DWF.

4.2 SCREEN AND GRIT CHAMBER

4.2.1 Gate

It is necessary to insert a penstock gate at the entry of the sewer into the wet well. The gate shall close by lowering the gate by either hand driven or motorized gear wheel.

4.2.2 Screens

These are needed to trap the floating matters like sachets, plastic milk packets, grocery bags, etc., which otherwise can lump in the impeller. The travelling mechanized endless screen is recommended so that man entry is totally avoided. For this purpose, it is necessary to restrict the width of flow to a rectangular profile in plan with the upstream length as at least three times the width and downstream length as at least two times the width. It is difficult to design and construct such a rectangular structure at deep depths. Hence, the recommended procedure is to construct the circular well first and fill up the arc sections with partitioned mass concrete to get at the rectangular passage. The design is invariably governed by equipment manufacturers who use the DWF and peak flows as the basis. In large pumping stations, it pays to have two successive screens: one coarse and the other fine, the idea being to have a back-up, in case one of them is in downtime. In small stations where the depth of incoming sewer is just about 3 m or so, a hand operated screen facility can be provided as in Figure 4.1.

The screen champer consists of two individual screens hung from a common wire rope gliding over a pulley lined with Teflon to avoid friction and avoid need for oil or grease to get over the friction. When one screen is in operation, the other is in raised position to facilitate cleaning. This relative movement can be got either by manually rotating the pulley wheel or mechanically doing this through a motor and limit switch. Each screen has an L shaped tray with perforated sheet at the bottom and when raised, the cleaning between the screens by a manual rake disturbs the screenings which will fall into the tray from where it is scooped out by a push of the spade over it and emptying directly into the trolley at ground level.

Figure 4.1 Typical Hand operated Screen Facility at Shallow Sewers in Pumping Stations

4.2.3 Amount of Screenings

Refer Section 5.6 of Chapter 5.

4.2.4 Configuration, Number of Grit Chambers and Method of Degritting

Grit shall be removed at the SPS to safeguard the same from causing wear to the pump impeller and inside of especially RCC pumping mains. In case of HDPE and PVC pipeline, the material of the wall does not succumb to erosion as long as velocities are between 1 m/s and 3 m/s and moderate grit content can be even pumped out directly to the STP. For almost all other pipelines the grit will erode the wall thickness and the pipes may collapse after some time. All the same, it is best to remove the grit before pumping.

The grit well shall be an independent well upstream of the wet well. A reliable grit removal system shall be a simple submersible pump set. The system shall be designed such that the floor of the wet well is depressed below the level of the incoming and outgoing sewers. The depression shall be minimum 0.6 m deep at one end and 1.0 m at the opposite end and such end finished flat for 1 m diametrical distance to house submersible pump sets. These pump sets shall be operated at the beginning of each eight-hour shift to pump out the grit laden sediments to a filtering masonry unit at GL and its filtrate let back into the grit well. The filtering masonry unit shall follow the designs of the sludge drying beds as in chapter 5 of this manual. The pump out rate shall be equal to the volume of the depressed portion pumped out in 10 minutes. The filtrate will be returned by gravity to the wet well. The pumped out sewage grit mixture can also be put through vortex separators (see Chapter-5 sub section on grit) installed above ground level and the grit collected in a trolley and the overflow degritted sewage can flow back to the well itself. The grit at the bottom can be further handled in screw classifiers like in detritors and elevated to fall into a stationary trailer. The system will be an enclosed and compact system eliminating human contact. There are many such integrated systems but these are patented. Hence specifying design criteria is difficult. However, these can be procured on competitive basis.

4.2.5 Amount of Grit

Refer section 5.6 in Chapter 5 of this manual.

4.2.6 Treatment and Disposal of Screenings and Grit

If land area is available, the screenings can be segregated to remove non-biodegradable components and blend it with grit and compacted. If not, it can be made a secure fill into an elevated HDPE container to be transported to the STP as and when it fills up. The removed plastics will be disposed into the municipal solid wastes.

4.3 MACHINERY ROOM

This room will house both switches, for switching ‘on’ and ‘off’ and the control gear and shall be a dedicated room with no other occupant and well ventilated with two entries/exits. The height of the room shall be the typical 4.5 m and the roof shall be a permanent structure of masonry. Timber products shall be totally avoided in the construction and fixtures and furniture in this building.

4.4 MEASURES AGAINST ODOUR

The best method is to provide good aromatic plants around and not trees. Artificial room sprays can be used, but not inside the electrical control room.

4.5 PUMPS

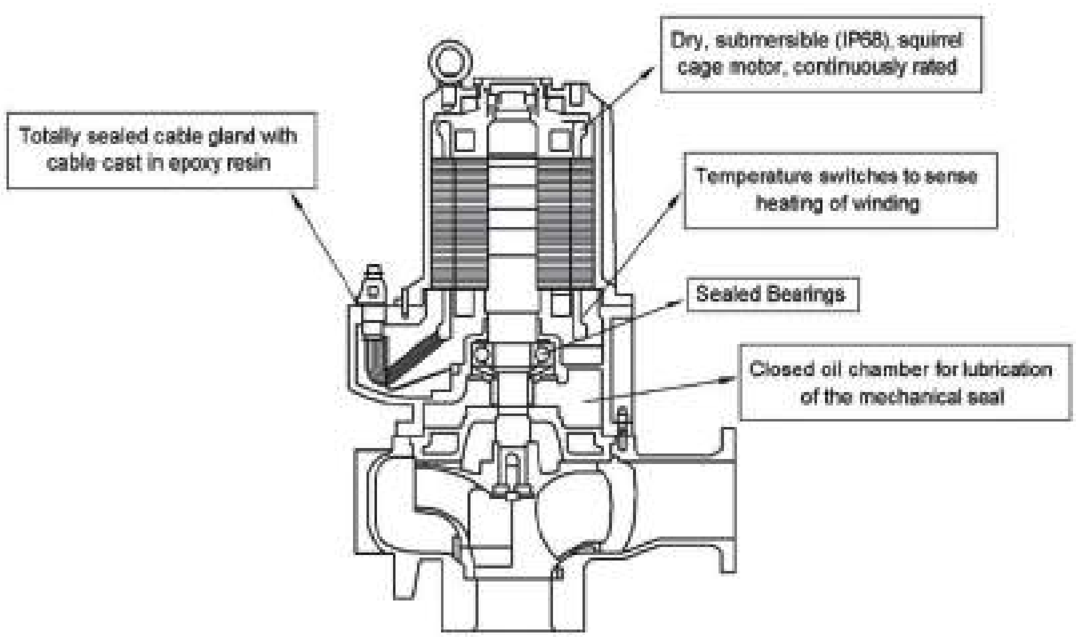
4.5.1 Types of Pumps

Historically, the sewage pumps are of two types, namely, horizontal axis driven with impeller rotating in the vertical plane or vertical axis driven with impeller rotating in the horizontal plane. Both these are centrifugal pumps. The vertical axis driven pump has the advantage that the pump can be at a lower elevation and the motor can be at a higher elevation and connected by a vertical shaft, which permits the pump at the floor of the wet well and the motor on top of it above the MFL. If horizontal axis driven pump is used, the motor and pump are close coupled and can be installed in dry well at the same depth of wet well. The suction pipe driven through the wall or erected above the MFL and the vacuum pump set is used to create the vacuum in the pumping arrangement called negative suction with priming. In general, this negative suction and its vacuum priming are to be avoided altogether in sewage.

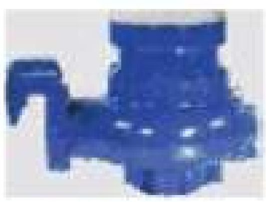
The later entrant of submersible pump sets are with integral motor and pump in the same casing and the assembly is water tight to the motor compartment and functions on vertical axis. Unlike the individual motor coupled pump sets, where the heat of the motor is dissipated by the air circulation brought about by the fan driven by impeller shaft blowing the air over the motor surface, the submersible pump sets require to be kept submerged in sewage at all times and the cooling of the motor is achieved through the surrounding sewage.

The recent entrant of immersible pump sets is with a seal of oil around the motor and which takes care of its cooling. Thus, theoretically, it is possible to pump out the wet well contents to almost the mid height of the pump and this saves considerable construction costs. These are otherwise similar to the submersible pump sets.

A typical section of a submersible pump set is shown in Figure 4.2.



TYPICAL SECTIONAL VIEW OF SUBMERSIBLE SEWAGE PUMP



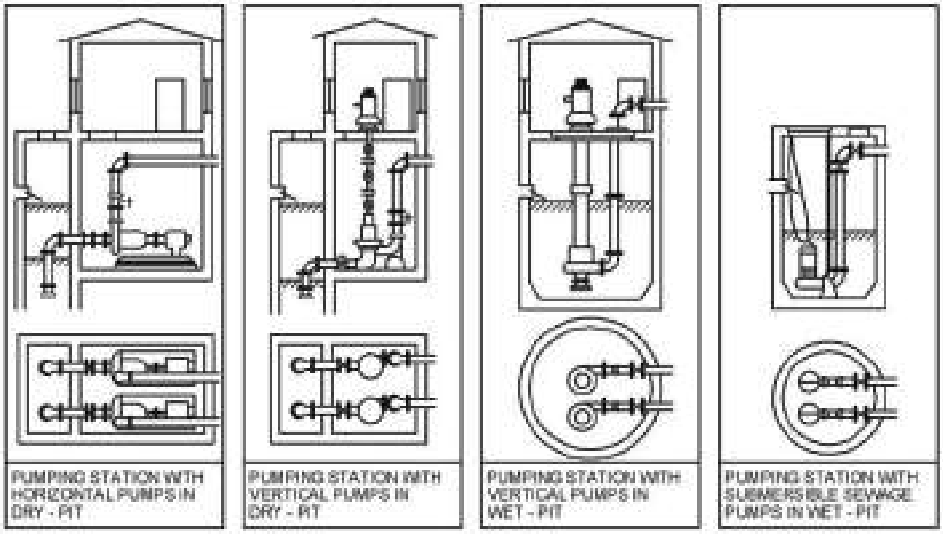
SELF-LOCKING CLAMP TO SEAL THE PUMP DELIVERY TO THE PIPE BRANCH

Figure 4.2 Arrangement of internals in a submersible pump set

In all cases, the free passage between the impeller and casing is called the ball passing size and is to be preferred as minimum of 80 mm.

4.5.2 Types of Pump Stations

This is shown in Figure 4.3 and is self-explanatory.

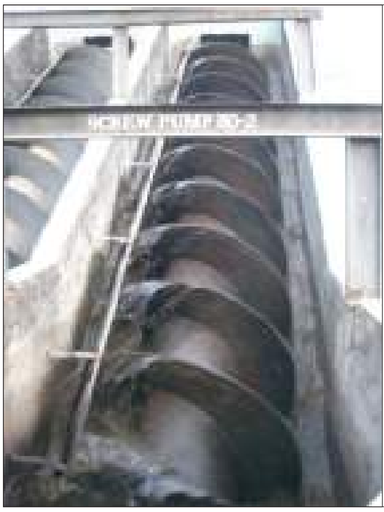


Source: CPHEEO, 1993

Figure 4.3 Typical dry-well and wet-well installations

4.5.3 Screw Pump Stations

There is yet another type known as immersible pump set where the cooling is made by an oil chamber filled with specific oil around the motor in the same arrangement as the submersible pump set and in this case, there is no need to keep the minimum depth of sewage submergence. There is also the Archimedean screw pump set which is shown in Figure 4-4 and which can only be used for lift stations as the delivery will be at atmospheric conditions .



The open impeller screw picks up and lifts the sewage on the screw and discharges at the top of the screw. The rotational speed is generally about 20 to 30 rpm. The motor is mounted at the top with a gearbox kept inclined and thus driving the impeller axially. Depending on the speed of rotation the quantity lifted varies. There is no piping or valve. The material is carbon steel. Photo courtesy CMWSS Board, Koyambedu STP-Chennai

Source: CMWSSB, Chennai

Figure 4.4 Archimedean screw pump.

4.5.4 Number of Pumps

The capacity of a pump is usually stated in terms of Dry Weather Flow (DWF), estimated for the pumping station. The general practice is to provide 3 pumps for a small capacity pumping station comprising (a) 1 pump of 1 DWF, (b) 1 of 2 DWF and (c) 1 of 3 DWF capacity. For large capacity pumping station, 5 pumps are usually provided, comprising (d) 2 of 1/2 DWF, (e) 2 of 1 DWF and (f) 1 of 3 DWF capacity, including standby.

Alternatively, the number of pumps can also be chosen to be in multiples of DWF flow and provide a 100% standby capacity for peak flow. This will permit easier inventories, cannibalization and uniformity in electrical control systems and switchgear except that the civil structure may need a larger footprint. In this alternative, it is also possible to defer the actual pump installations till the commensurate volume of sewage arises in due course.

A combination of vertical submersible screw, centrifugal impeller pump and a vortex inducing arrangement at the pump pit floor is stated to induce a spin of the incoming sewage depending on the flow rate and thus producing a flow variation commensurate to the incoming flow variation, while the pump is in constant speed of rotation. Right now, it is a patented make. It will be useful to take up pilot project of this and such other technologies and evolve a system so that it can help reduce the costs of civil works and multiples of pump set machinery for future pump stations.

4.5.5 Selection of Pump Stations

Where suction lifts are about 3 m to 5 m only, the horizontal foot mounted centrifugal pump stations should be explored in view of the ease of repairs from local resources and the fact that motors and pumps can be independently taken out for repairs. In addition, where space limitations are constricting, submersible pump stations could be preferred. Archimedean pumps are rugged in operation, but have a limitation on the efficiency, which is only about 25% and are to be preferred in dealing with high volumes of raw sewage to be lifted over a short height. As otherwise, their application in sewage is very little except return sludge in STP where they are ideal.

4.6 Wet Well

Wet well pumping stations usually contain larger pumping units than those required for submersible type pumping stations. The pumping units are installed in the dry well whilst the sewage is stored in the adjacent wet well. To ensure that the centrifugal pumps are always primed, the pumps are located below the level of sewage in the wet well. In respect of submersible pump sets, the top of the pump set shall be such that the pump set is fully submerged at minimum level of sewage flow and the required wet volume is satisfied by the volume of wet well below the invert level of the incoming sewer and an additional allowance of 50 cm below it. Conventional stations are often equipped with multi-stage pumps.

Wet wells shall be designed and constructed to be as hazard free as possible, and corrosion resistant materials shall be used throughout. No junction boxes shall be installed in the wet well. Float cables and bubbler tubes shall be placed in a covered case that shall extend from the control panel to the wet well.

These shall be in 2 parallel wells, each catering to 50% of the volume or in case of large flows, a single well with two compartments which can be hydraulically connected by a penstock in the partition wall.

4.6.1 Structure

Sewage pumping-station wet wells shall be constructed of brickwork duly plastered or reinforced concrete and shall be circular. Wet wells that are installed below the groundwater table shall be adequately designed to prevent uplift pressure without the use of hydrostatic pressure relief valves. Wet well size and depth shall be as required to accommodate the influent sewer, provide for adequate pump suction pipe or pump submergence as recommended by the pump manufacturer and to provide adequate volume to prevent the frequent start and stop of pumps. Partitioning the wet well to help accommodate future growth requirements may be practiced.

4.6.2 Interior Linings and Waterproofing for Old Wells

Wet well interior walls shall be lined with a material that is suitable for prolonged immersion in sewage. The lining shall be completely resistant to hydrogen sulphide and sulphuric acid. The liner shall be easily cleanable and sufficiently durable so that it can be washed with a high-pressure water hose and shall be light in colour. Wet wells that are anticipated to be below the groundwater table shall also have a waterproofing system installed on the exterior of the wet well. Regardless of the elevation of the water table, all joints in the concrete and all penetrations through the concrete shall be grouted with non-shrink grout on both sides of the joint or penetration.

4.6.3 Floor Slopes

In the case of wet well and dry well type with horizontal food mounted centrifugal pumps in the dry well, the floor should have benching like a hopper with a minimum slope of 1 vertical to 1 horizontal to enable suspended solids to drain into the hopper and pumped out without depositing on the entire flow. In the case of submersible pump / immersible pump, the floor shall be horizontal to permit easy installation of present and future pumps.

4.6.4 Lighting

The interior of pump stations, whether at grade or below grade, shall have a lighting system specifically designed to provide illumination best suited for the station layout, which may include suspended, wall, or ceiling mounted. Energy efficient fluorescent fixtures are preferred. Lighting shall be at levels adequate for routine service inspections and maintenance activities.

4.6.5 Ventilation

Pump stations shall be provided with a separate ventilating system and shall be sized to provide a minimum of 10 air changes per hour. Ventilation systems shall be capable of matching inside air temperature to outside air and shall be automatic. Ventilation shall be accomplished by the introduction of fresh air into the pump station under positive pressure. The air shall be filtered to remove particulates inside the pumping station.

4.6.6 Wet Well Design Criteria

Size of the wet well should be based on the following:

1. Flow from proposed development and any associated future development
2. Capability to receive flows from surrounding areas as determined by the authorities

The volume of wet well is given by

$$V = T \times Q/4 \qquad (4.1)$$

where,

V :	Effective volume of wet well (in cubic meters)
T :	Time for one pump cycle (in minutes)
Q :	Pumping rate (cubic meters per minute)

The value of T is related to the number of starts per hour and it is not advisable to exceed 6 starts per hour. Accordingly, the value of T in the design is to be taken as 10 minutes for smaller pump capacities but 15 minutes is desirable and hence, the denominator in the equation is to be used as a value of 4.

Ideally, this volume has to be provided below the invert of the lowest incoming sewer in accordance with section 4.1.4 of this manual. However, it may not always be possible especially in large sized pumping stations. In such a case, the volume in the sewers calculated at 0.8 times their total volume can be considered as the extended wet well volume. This is illustrated in Table 4.1.

Table 4.1 Illustration of wet well volume in sewer systems

Given,	
Pumping capacity at peak flow	= 42 cubic meters per minute
Volume required	= 15 × 42 / 4 = 158 m^3
Possible depth below invert of sewer	= 2 m
Area needed	= 158 / 2 = 79 m^2
Diameter needed	= SQRT (4×79 /3.14) = 10 m
The depth of the wet well required is also governed by the depth of submergence needed for the submersible pump set. This is governed by the height of the submersible pump set and the floor clearance. Assuming the height of the pump set as 1.2 m and floor clearance as 0.3 m the minimum depth of the floor of the wet well below the invert of the sewer shall be 2 + 1.5 = 3.5 m.	

It may be difficult to construct wet wells of 3.5 m deep below invert of incoming trunk sewers which themselves may be at a depth of about 5 m to 6 m below ground level. Moreover, designing and constructing the wet wells to be checked for cracking stress in high water table areas may be not only difficult but may give way to infiltration which will be a challenge to control later on. Thus, it becomes a problem of obtaining sufficient wet well volume at reasonable cost.

A solution to this is proposed in Chapter 6 titled pumps and pumping stations of the book "Wastewater Engineering, collection, treatment and disposal" by Metcalf & Eddy, TMH edition, 1974, which states, "The effective volume V of the wet well between "on" and "off" float switch settings includes the storage in the incoming sewers. If the "on" switch setting is below the sewer invert, no storage is available. This setting is wasteful of both the storage capacity and head available in sewers of appreciable size. It also wastes power and may require higher head pumps and larger horsepower motors. Where it is necessary to rely on the storage in the sewers to obtain adequate wet well volume for control, backwater curves should be computed to obtain the effective volume between the various float switch settings. This may amount to 50% of the volume in the wet well itself. Some stations have been built with practically no wet well and in the case of really large stations, the wet well volume within the station approaches insignificance. In these cases, the only volume available for control is the storage volume in the sewers."

It is also a matter for consideration to move on to immersible pump sets in future where the submergence in sewage is not needed and the motor winding cooling is provided by an internal oil chamber around it in the example above, this will mean reducing the height of wet well below the incoming sewer by 1.2 m.

An ideal type of wet well design can be as shown in Figure 4.5 overleaf.

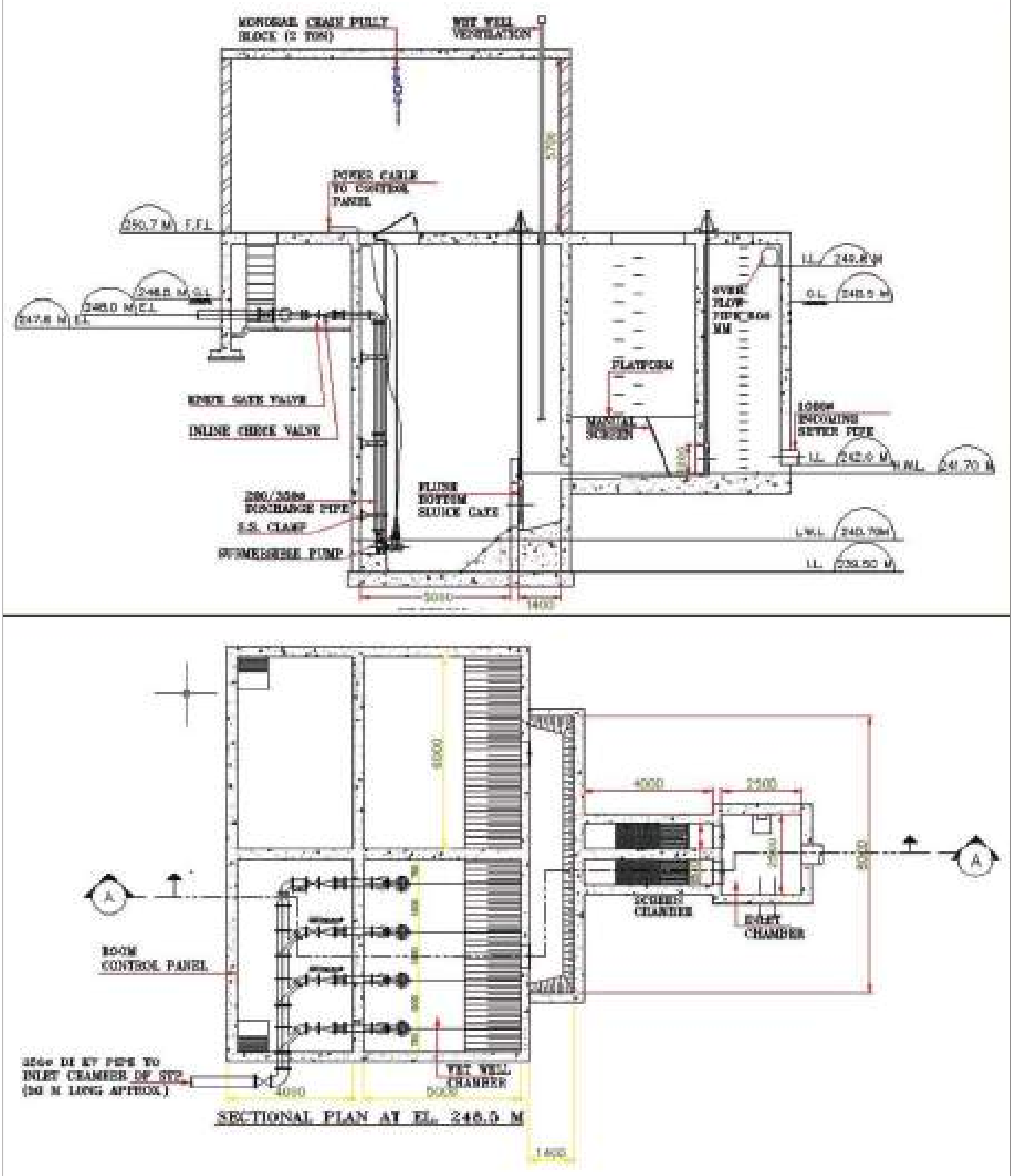
Following points should be considered while designing the wet well pumping system:

- Normal operating volume shall prevent any one pump from starting more than 3 times per hour.
- Level control is to be provided by ultrasonic level controller or submersible transducer.
- Provide high water and low water alarm activated by ultrasonic or submersible level control system and backup float switches.
- Locate level switch where flow from the inlet pipe will not interfere with the float.
- Design electrical service to handle the ultimate capacity of the pump station.

4.6.7 Structural Design Criteria

In respect of civil structural design, all wet wells shall be designed to withstand soil water pressure as though it is at ground level itself irrespective of actual water level, to take care of contingencies of flooding and marooning of the stations. In addition, the stability of the base slab shall be checked for resisting moment by considering the weight of the slab alone and neglecting the weight of the sidewalls. Pressure relief valves for soil water uplift should not be encouraged in wet wells and IS: 456 & IS: 3370 shall be followed. All civil works in contact with sewage shall be constructed with either brick work or RCC and in both cases sulphate resistant cement alone shall be used. In RCC, Fusion Bonded Epoxy coated reinforcement steel having not less than 175 to 300 microns shall be used for reinforcement.

Epoxy coating over the inner face of the screen well / grit well 1 m above the maximum sewage water level is recommended.



Illustrative wet well design for submersible pump set

The top of the pump set shall not be exposed to the atmosphere and shall be always submerged. The required wet well volume shall be provided preferably between the invert of the sewer and the top of the submersible pump set. This is achieved in this drawing. The wet well volume for large stations can also be compensated by the volume in sewers obtained at 0.8 of diameter and backwater curve.

Figure 4.5 Typical arrangements of a submersible pump set wet well

4.7 PUMP BASICS

Even though submersible pumps have become a generic name, the fact remains that the basics of pumps as applicable to centrifugal pumps apply to these also and accordingly, these are discussed herein.

4.7.1 Centrifugal Pumps

These are by far the most widely used in the country in the past in sewage pumping and are generally classified as radial flow, mixed flow and axial flow pumps based on the specific speed of the pump (n_s), which is obtained from the following formula:

$$n_s = \frac{3.65 n \sqrt{Q}}{H^{0.75}} \tag{4.2}$$

where,

n : Speed of the pump in rpm

Q : Flow rate in m^3/s

H : Head of the pump in m

The specific speed of the pump is akin to a shape number and forms the basis for the design of the impeller of a centrifugal pump. The shape of the impeller is identifiable by the relative proportions of the inlet size, outlet width and the outside diameter. Broader inlet size and outlet width are logical for larger flows. For higher head-to-speed ratio the impeller would be logically narrower than broader. Therefore, the specific speed is larger and the shape is broader. This is proportional to the flow-rate and inversely proportional to the head-to-speed ratio.

In a narrow and tall impeller, the flow through the impeller will be radial, i.e., across a plane perpendicular to the axis of rotation. Hence, these are called as radial flow pumps and are pumps of low specific speed, generally between 40 and 150.

In a broad and short impeller, the flow through the impeller will be partly radial and partly axial. Hence, these are called as mixed flow pumps and are pumps of specific speeds in the range from 150 to 350. If the impellers in the pumps has specific speeds higher than 350, the flow is more or less parallel to the axis of rotation and hence these pumps are called as the axial flow pumps. In a double-suction pump, the impeller is actually a composite impeller, with two identical flow-passages combined back to back. Each side is practically an independent impeller and each such impeller handles only half the flow. So, the specific speed for such pumps is calculated by taking only half the flow. By this, the specific speed of a double-suction pump is only 70% of what the specific speed would have been with a single-suction design.

Generally, pumps of low specific speed can work with more suction-lift than the pumps of higher specific speed. With the pumps of very high specific speed as that of the axial flow pumps, not only that they would not work with any suction-lift, instead they would need positive suction head or minimum submergence for trouble free working.

It is always advisable to avoid suction-lift for any centrifugal pump. In the SPS, the pumps are installed either to work submerged in the wet well itself like vertical pumps with motor above GL and the pump in the wet well connected by a rotary shaft or installed in the dry well at such a level that the impeller will be below the level of the liquid in the wet well.

The power-characteristics of the centrifugal pumps are also related to the specific speed. The radial flow pumps with low specific speed have such power-characteristics that the required input power to the pump increases as the capacity, i.e., the flow-rate of the pump increases.

In radial flow pumps, the power demand is minimum with zero flow, i.e., with the delivery valve closed. Since the pump should be started with the pump exerting the minimum load on the driver/ motor, the radial-flow pumps should be started with the delivery valve closed.

The power-characteristics of the mixed flow pumps are almost flat or with very little gradient so, the mixed flow pumps can also be started in the manner similar to that for the radial flow pumps. However, in the case of axial flow pumps, the power needed to put into the pump is maximum at zero flow. These pumps should hence be started with the delivery valve fully open.

The impellers of centrifugal pumps have vanes, which are either open or have shrouds. Open impellers have no shrouds. Semi-open impellers have only a back shroud. Enclosed impellers have both the front and the back shrouds. Axial flow pumps would have only the open impellers. The mixed flow pumps, especially of the higher specific speed would be generally semi-open. However, the impellers of radial and mixed flow pumps can be constructed in all the three types.

The centrifugal pumps are used more commonly for clean and clear liquids. The enclosed impellers are the most common in construction. The impellers are constructed of the semi-open or open type depending on the size of the solids and the consistency of solids to be handled. For handling large-size solids, the impellers are also designed with fewer vanes, which would however have less efficiency.

In the case of high head pumping, the total head is shared by more than one impeller in the multi-stage pumps. With very high head, for a single-stage pump the specific speed may become less than 40 and in turn so low that even the radial flow design would be too narrow. By making the head to be shared by more than one impeller, the specific speed for each impeller will be better. On the other hand, high head would be beyond the range of a single-stage, high specific speed mixed flow or axial flow pump.

Multi-staging would make the head attainable, as is typically seen in vertical turbine pumps. In multi-stage construction, the flow out of one impeller is carried to the suction of the next impeller, with some conversion of the kinetic energy into pressure-energy, in a bowl or a diffuser.

In single-stage pumps, the energy conversion is achieved in a volute casing around the impeller.

For ease of access to the internals the volute casing is often made of the axially split type. This facilitates accessing all the rotating parts for cleaning or repairs, without disturbing the fixation of the pump with the adjoining suction and delivery piping.

4.7.2 Computation of the Total Head of Pumping

The total head of pumping has to be calculated taking note of four factors.

Firstly, the differences between the static levels of the liquid in the suction sump, i.e., the wet well and the highest point on the discharge side makes the potential or static head.

Secondly, the rate of flow and the size of the discharge-mouth determine the velocity at the point of discharge and in turn the kinetic or the velocity head.

Thirdly, the difference in the pressures on the liquid in the suction sump and at the point of delivery makes the pressure head. On the suction side, the liquid in the wet well is open to the atmosphere, but on the delivery side when delivering into a closed conduit sewer, there would be a potential head at the point of delivery, against which the pump will have to deliver. Therefore, the delivery pressure will be higher than atmospheric. The pressure-differential will make the pressure head.

Lastly, the pump has to generate as much head as is needed to compensate for the frictional losses across the pipes, valves, bends and all such appurtenances both on the suction and delivery sides. This makes the frictional head.

With the pumps running, if the discharge of the pumps is more than the inflow, then the level of the liquid in the wet well would keep falling. By this, the potential head component in the total head would keep increasing. Converse will be the case when the inflow is more than the discharge by the pumps.

Throttling of the delivery valve causes a change in the rate of flow and in turn a change in the velocity head which varies in square proportion of the velocity, because the velocity head is computed as $V^2/2g$.

The frictional losses also vary in square proportion of the velocity or flow-rate. The formula for calculating the friction loss will be the Hazen Williams formula as in section 3.16.2 of this manual.

There will be losses in fittings of the pipe line which can be calculated as a function of the velocity head and as in Table 4-2 overleaf.

A typical calculation of the total friction factor for fittings is shown in Appendix A.4.1.

4.7.3 System Head

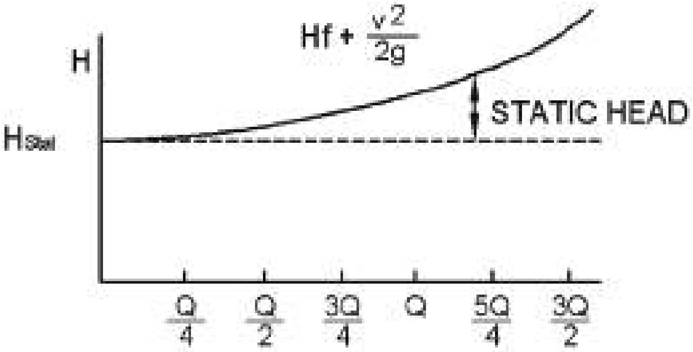
At the stage of planning, the method of computing the total head of pumping should be to estimate it over a range of flow-rates, for different variations in the static levels and for different options of piping sizes and layouts. This obtains the system head curve, as illustrated in Figure 4-6 overleaf.

With an increase only in the potential head, the new system head curve will be a curve shifted parallel upwards, as shown in Figure 4.7 overleaf.

For a smaller size of piping, the parabolic portion in the system head curve will be steeper, as shown in Figure 4.8.

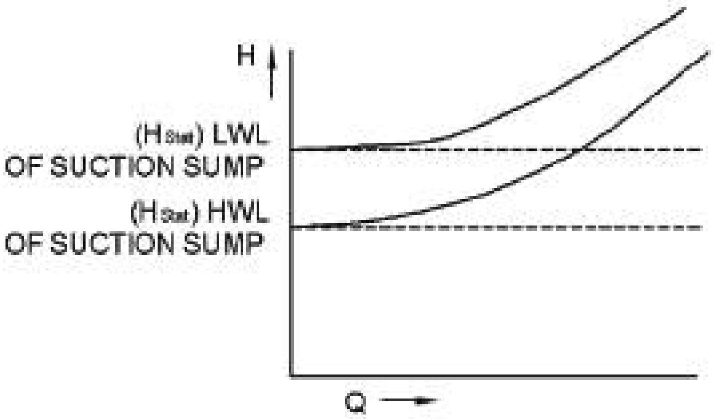
Table 4.2 Friction factor for fittings in pumping mains

No.	Type of Fittings	Factor
1	Sudden contraction	0.5
2	Entrance shape well rounded	0.5
3	Elbow 90 degrees	1.0
4	Elbow 45 degrees	0.75
5	Elbow 22 degrees	0.5
6	Tee 90 degrees	1.5
7	Tee in straight pipe	0.3
8	Gate valve open	0.4
9	Valve with reducer and increaser	0.5
10	Globe valve	10.0
11	Angle	5.0
12	Swing check	2.5
13	Venturi meter	0.3
14	Orifice	1.0



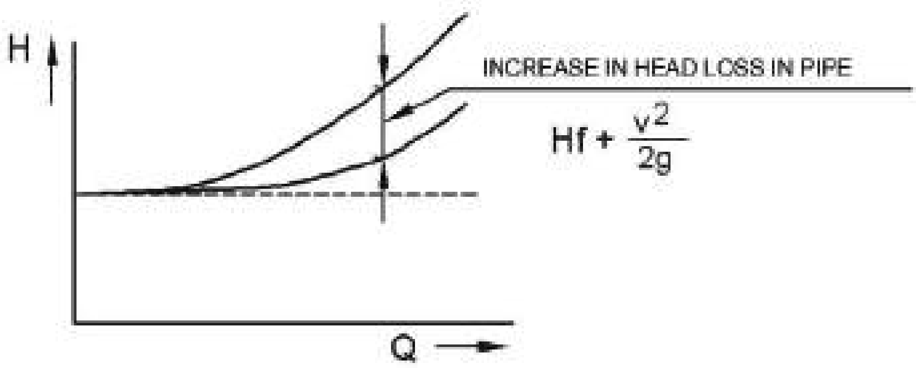
Source:CPHEEO, 1993

Figure 4.6 System head curve for a pumping system



Source:CPHEEO, 1993

Figure 4.7 System head curves for LWL & HWL in suction sump



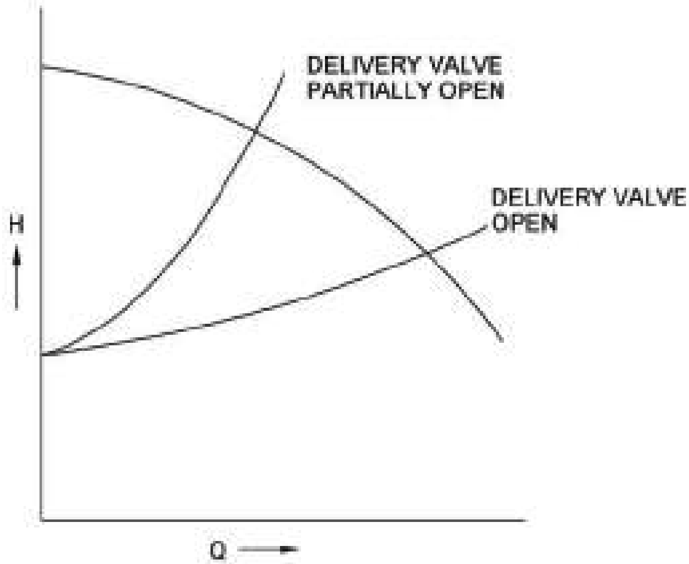
Source:CPHEEO, 1993

Figure 4.8 System head curves with change in pipe sizes

From the system head curves, one knows what the total head would be for an average operating condition, which then can be specified as the total head of pumping.

4.7.4 Operating Point of a Centrifugal Pump

The Head-Discharge (H vs. Q) characteristics of a centrifugal pump are a drooping parabola, with the pump discharge being less when the head is more. When the pump is put into a system, it meets the head as demanded by the system. The system demand is as per the system-head curve. The head met by the pump is as per its H-Q curve. For example, by throttling the delivery valve to close, the system head curve would become a steeper parabola and would intersect the H-Q curve of the pump at a point of higher head and less discharge, thus becoming the new operating point of the pump. This is illustrated in Figure 4.9.

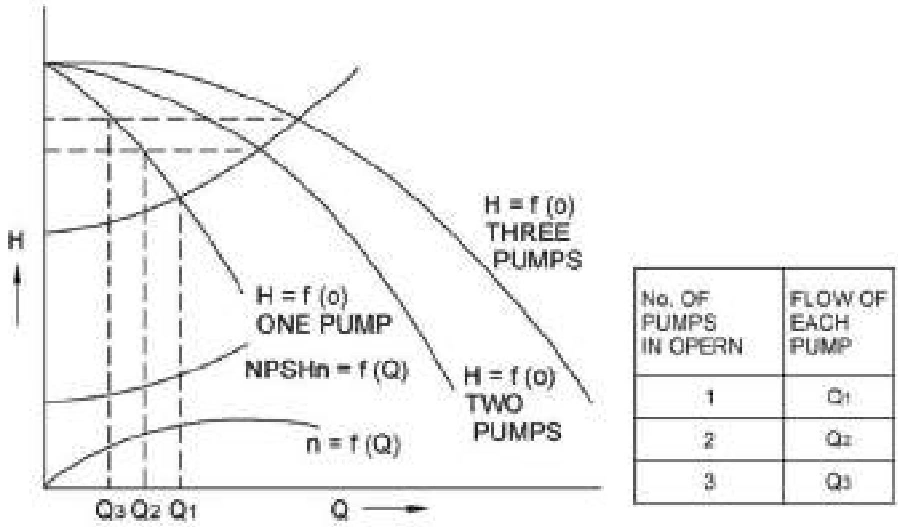


Source:CPHEEO, 1993

Figure 4.9 Change in operating point by operation of delivery valve

4.7.5 Parallel Operation

When more than one pump would be discharging into a common closed conduit or header, the performance characteristics of the pumps suffer mutual influences. Pumps discharging into a common closed header/conduit are said to be running in parallel. The flow obtained in the header is what is contributed by all the running pumps together. The combined characteristics of pumps running in parallel are obtained by reading against different heads; the values of the Q obtainable from the individual pumps and plotting the addition of the Q-values against respective heads, as illustrated in Figure 4.10.



Source: CPHEEO, 1993

Figure 4.10 Operation of pumps in parallel

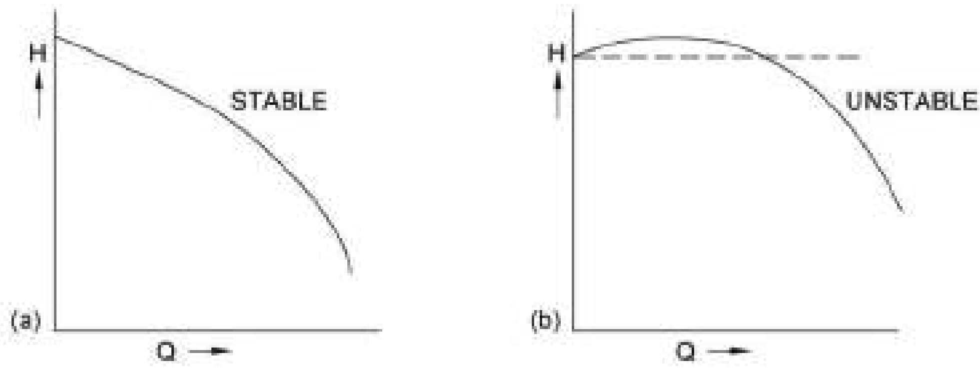
The operating point of parallel operation is the point of intersection of the combined H-Q curve with the system head curve. Because the point of intersection on the combined characteristics is at a head higher than that at the point of intersection on the H-Q curve of a single pump, the discharge at the operating point will be the intersection on the H-Q curve of a single pump.

From this, it is clear that two identical pumps put into parallel operation will give discharge less than double the discharge of only one pump operating. One must study what combination of pumps of different H-Q characteristics can give such combined characteristics as to have an intersection on the combined characteristics at the desired double discharge.

As seen from Figure 4-10, if there are two identical pumps running in parallel, individual pump would be contributing a discharge Qp. If one of the pumps would trip, the system would have only one pump running and giving a discharge Q1, which is more than Qp. At higher discharge, the pump would draw more power, which should not overload its motor. While putting the pumps into parallel operation, it is sound logic to provide that the discharge Qp in parallel operation would be somewhat to the left of the discharge at the best efficiency-point (b.e.p.) of the pump. This will aid in the event of tripping of any other pump, the higher discharge such as Q1 of the running pump will only be nearer to its b.e.p.

4.7.6 Stable Characteristic

It is possible that on the H-Q curve of a centrifugal pump, the shut-off head will not be the maximum head, as shown in Figure 4.11.



Source: CPHEEO, 1993

Figure 4.11 Stable and unstable characteristics of centrifugal pumps

Such H-Q curve is called unstable, because at heads higher than the shut-off head, the discharge of the pumps keeps hunting between two values, causing the pump's performance to be unstable. Such instability is prone to cause the pump to suffer vibrations. This becomes more hazardous in parallel operation, because the hunting of flow of the unstable pump causes the other pumps also to experience continuous change in their share and in turn hunting, instability and vibrations. Pumps to be put into parallel operation should hence be only of stable H-Q curve or care should be taken that the system head will definitely be safely less than the shut-off head of the pump with unstable curve.

4.8 CAVITATION IN PUMPS

The flow must reach the eye of the impeller with such absolute pressure-head, that it will be higher than the vapour-pressure and the net positive suction head required ($NPSH_r$) by the pump. The absolute pressure-head of the flow, as it reaches the eye of the impeller can be found by deducting from the pressure on the liquid in the suction sump. It is atmospheric in the case of an open sump such as the wet well, firstly, the static head between the liquid level in the suction sump and the centre-line of the pump, if the pump's centre line is above the liquid-level, i.e., if there is a suction-lift. If the centre-line of the pump is below the liquid level, i.e., if the suction is flooded, the static head will have to be added and not deducted. Next, the velocity-head, appropriate to the suction-size will have to be deducted.

In addition, the frictional losses up to the eye of the impeller will also have to be deducted. Even if the flow has a positive absolute pressure, after all the deductions, while reaching the eye of the impeller, the flow suffers from shocks, twists, turns and turbulences at the eye of the impeller. This tax on the energy in the flow is called as the net positive suction head required (NPSHr) of the pump. Therefore, the positive absolute pressure of the flow, as it reaches the eye of the impeller should be more than the vapour pressure (Vp) even after providing for NPSH.

This means:

(Pressure head at the eye of the impeller) > ($NPSH_r$ + V_p)

i.e., (Pressure head at the eye of the impeller − V_p) > ($NPSH_r$)

The value in the parenthesis now on the left is termed as the $NPSH_a$, i.e. Net Positive Suction Head available. $NPSH_a$ can hence be derived as follows:

$NPSH_a$ = Pressure on liquid in the suction sump ± Static head between the liquid level in the suction sump and the centre line of the pump − velocity head − frictional losses up to the eye of the impeller − vapour pressure

If the $NPSH_a$ be not greater than $NPSH_r$, vapour bubbles get formed, which while travelling along the flow, being compressible receive energy from the impeller, which builds up the pressure inside them and the resultant compression reduces their volume culminating in the collapse of the bubbles with sudden release of the energy. This causes impact and vibrations. This entire phenomenon is called cavitation and can cause very serious damages. The simple clue to avoid cavitation is to ensure that $NPSH_a$ will be more than $NPSH_r$. The formula given above for $NPSH_a$ suggests many possibilities of keeping $NPSH_a$ as high as possible.

4.9 PRIME MOVERS

Invariably the prime mover is an electrical driven motor. See section in Chapter 5 for further details.

4.10 SURGING OF PUMP AND WATER HAMMER

This may occur where the delivery is for a high head and there is a sudden shut off when the sewage in the delivery main surges back on the pump. In medium situations, this is taken care of by the non-return valves in the delivery main which itself is the surge control device, especially when the dashpot type is used wherein an air cushion is trapped inside a chamber and the surge force is absorbed and the flap does not bang against the valve seating. A sudden closure can lead to bursting of the pipeline and hence the surge analysis has to be made and evaluated for the normal operation of the pump station as well as for a power outage while the pump(s) are running. The modulus of elasticity of the pipe material shall be considered when evaluating water hammer effects and cyclic loadings. At a minimum, the following should be addressed in the surge analysis:

- Transient pressures due to water hammer and its effect on the entire system
- Cyclic loading of the force main
- Investigation of the pipeline profile to determine the possibility of water column separation
- Reverse rotation characteristics of the pumps
- Shut-off characteristics of all proposed pump control valves (if allowed), including check valves.
- Substantiation for the use of surge control valves and other surge protection devices, when necessary, listing recommended size and computed discharge pressures

The potential impact of water hammer shall be evaluated with special consideration given to cyclic loadings that are inherent in sewage force mains. All elements of the piping system must be designed to withstand the maximum water hammer in addition to the static head and cyclic loading. A minimum safety factor of 1.5 shall be used when determining the adequacy of all piping system components with regard to withstanding system pressure. A surge control device in lieu of strengthening piping system components may be used based on the life-cycle cost comparison.

The software for surge analysis is rather complicated and hence it is not promulgated in this manual. Suffice it to state that when the delivery heads exceed 25% of the horizontal length of the delivery pipeline, a surge analysis can be carried out by using the commercially available software or outsourced to institutions of repute.

Water hammer is an internal surge in pressure inside the pumping main when a pump suddenly stops or when a delivery valve in the pumping main is suddenly closed causing a reversal of the flow direction instantly and its forward and reverse oscillation. This phenomenon imparts a higher instantaneous pressure on the pumping main and can cause bursting depending on the magnitude which is almost entirely a function of the static lift. In general, sewage-pumping mains seldom encounter static lifts of more than about 20 m and this will not be a problem.

Moreover, soft-start starters shall be used to ameliorate the situation as also spring-check or dashpot type of non-return valves to be used instead of plain swing-check valves. There are also customized protection systems from appropriate equipment vendors.

4.11 PIPING AND VALVES

The suction and delivery piping of pumping stations are to be chosen between ductile iron and cast iron, in that order and the inside lining shall be with either high alumina cement mortar or polyurea and outside coated with epoxy. Joints shall be of O-ring spigot and socket and valve fixtures shall be through appropriate flanged joints. Next are the RCC pipes with high alumina cement or polyurea lining on the inside and a sacrificial concrete of 15 mm to 20 mm on both the inside and outside and in cases where the soil water has sulphates exceeding the limits for concrete, sulphate resistant cement shall be used in the manufacture itself. The use of MS pipelines is not advocated.

The preferred material of valves is cast iron body with disc of such material as desired by the user agency and relevant BIS code.

4.12 APPURTENANCES

4.12.1 Air-Release and Air/Vacuum Release Valves

Air release and air/vacuum release valves shall be specifically designed for sewerage services and be sized as per the manufacturer’s recommendations. Air release and air / vacuum release valves shall be required at pumps on the discharge pipe as close as possible to the check valve. The air and vacuum release valves will be contained in a vault and vented above ground. A manually controlled isolation valve shall be installed between the force main and the air release or air / vacuum release valves.

4.12.2 Drain Valves

There should be provision of at least one force main dewatering connection at the pumping station and dewatering connections at other major force main low points. Drains shall generally include a plug valve installed on a tee and drain piping to an existing sewer manhole or to a separate manhole that can then be pumped out.

4.12.3 Additional Appurtenances

Additional appurtenances at sanitary sewer pumping stations and force mains should be provided on a case-by-case basis.

4.12.4 Dry Well

This shall be designed in accordance with IS: 456 and IS: 3370 and the precautions stipulated in Subsection 4.6.7 shall apply here also.

4.12.5 Automatic Operation of Pumps and Equipment

Automatic operation of pumps is possible by pre-programmed logic controllers which start the specified pump set once the sewage level reaches a specified height and progressively brings in more pumps into operation and the same in reverse order with dropping of sewage levels. The input to this is the float switch with mercury contact in sealed float, which gets tilted to horizontal and floats when sewage level reaches the float and thereby closes an electronic circuit inside the float which generates a standard signal of 4 mA to 20 mA which is relayed to the control panel for activating the pump. When the sewage level falls, the circuit gets tripped and the signal vanishes and the pump is tripped. The key to the whole issue is to recognize the pre-set programming which may have to be validated for different seasons like monsoon, normal and drought. For this purpose, these controllers are referred to as programmable logic controllers (PLCs). These are custom designed.

4.12.6 Protective Equipment

Refer chapter 5 of this manual.

4.13 AUXILIARY POWER DEVICES

Refer chapter 5 of this manual.

4.14 ALARM SYSTEMS

Alarm is indicated when the pump is running dry and when the motor temperature exceeds the specified limit. In both cases, the method of instant detection is most crucial. The dry running of the pump is detected by the no flow reading in the flow meter. The temperature increase in the motor is detected by the built in temperature sensor which uses the bimetallic properties of dissimilar metals and a set point transducer. In both cases the signal generation is the standard 4 to 20 mA which is relayed to first trip the pump set and simultaneously raises a hooter and visual annunciation by appropriately coloured flashing lamps. Refer chapter 5 of this manual on instrumentation.

4.15 FLOW MEASUREMENT

4.15.1 Magnetic Flow Meters

Magnetic flow meters work on the principle of electromagnetic induction. The induced voltage generated by an electrical conductor in a magnetic field is directly proportional to the conductor's velocity. Thus, the sewage is the conductor and is suitable for all piping like, raw sewage, settled sewage, primary sludge, return activated sludge, waste activated sludge and treated sewage. These are non-invasive and used in almost all pipelines but of course initial calibration is needed. The output is the standard 4 mA to 20 mA signal which is relayed to the central monitoring system.

4.15.2 Ultrasonic Flow Meters

When ultrasonic impulses are released onto a pipe surface carrying sewage, the impulses are deflected along the flow direction based on the velocity of the flow before they impinge on the opposite sidewall of the pipe. The time taken is measured and is correlated to the velocity and then to the diameter of the pipeline and hence the flow rate is arrived at. Like magnetic flow meters these are also non-invasive and used in almost all pipelines but of course, initial calibration is needed. The output is the standard 4 mA to 20 mA signal which is relayed to the central monitoring system.

4.16 CORROSION PREVENTION AND CONTROL IN PUMP SETS

In general, when pipes are flowing full, corrosion does not arise. As such piping in pumping stations, as long as they are of DI or CI, will not exhibit corrosion inherently because they are of such a material and because there is no chances of sulphide corrosion on these metal castings. However, mild steel fixtures will immediately go into corrosion and will be totally avoided. The fasteners shall be of SS under all circumstances.

4.17 REHABILITATION / RECONSTRUCTION OF PUMPING STATION

These arise in contingent situations such as incoming sewage flow exceeding the capacity of the pump sets, or the pump sets are old or the civil works are beginning to crumble. When the inflow exceeds the capacity of existing pump sets, if the increase is marginal, it may be possible to use a variable frequency drive and increase the speed of the pump set, but this may not be a permanent measure. Installation of diesel pump sets in the open area and connecting the pumped sewage to the existing delivery main header is another option, but here again, may be to about the same 10% extra flow only as otherwise the pressure in the delivery main will increase and burst can occur.

A better option will be to switch over to near uniform pumping instead of using peak hour pump sets in the morning peaks whereby the no flow time slots and night-time slots can be brought into play beneficially. In fact, if the pumping is effectively managed in this way, the volume of the entire sewer system itself will buffer the morning peak flows till about noon time for stretched out pumping.

If the civil works start crumbling, the first thing is to construct another independent electrical control panel room and shift all electrical gadgets there. The next is to gunite the outer surface of the walls of the wet well to arrest leakages on both sides.

As for the bottom slab, it is difficult to examine its integrity and if it is only a wet-well with no submersible pump sets, under pinning technique can be used. If the well has installed submersible pump sets, a possibility will be to sink another wet-well and shift the pump sets and then attend to the old well.

4.18 LIFT STATIONS

In locations of high water table and rocky terrain, a typical conventional sewer design and more so its construction poses a series of challenges when depths of excavation exceeds about 3 m. Eventually, the depth of wet-well is also negatively influenced by this issue.

In such situations, it is advantageous to opt for intermediate lift stations, which are like “on line”. In general, these are submersible pump stations, which are interposed in the gravity sewer network.

The procedure is to sink a wet-well on the road shoulder or an acquired plot beyond the shoulder and divert the incoming deeper sewer to it and the submersible pump set therein will lift the sewage and discharge it to the next on line shallow sewer. As the sewer progresses, any number of such lifts can be inserted based on the location. These shall be connected to dedicated electricity feeders as installation and O&M of standby diesel pump sets etc., are not feasible in such locations.

A typical lift station is illustrated in Figure 4.12 overleaf.

4.19 INSTALLATION OF PUMPS

The procedure of installation depends upon whether the pump is to be mounted horizontally or vertically. Most pumps to be mounted horizontally are supplied by the manufacturers as a wholesome, fully assembled unit.

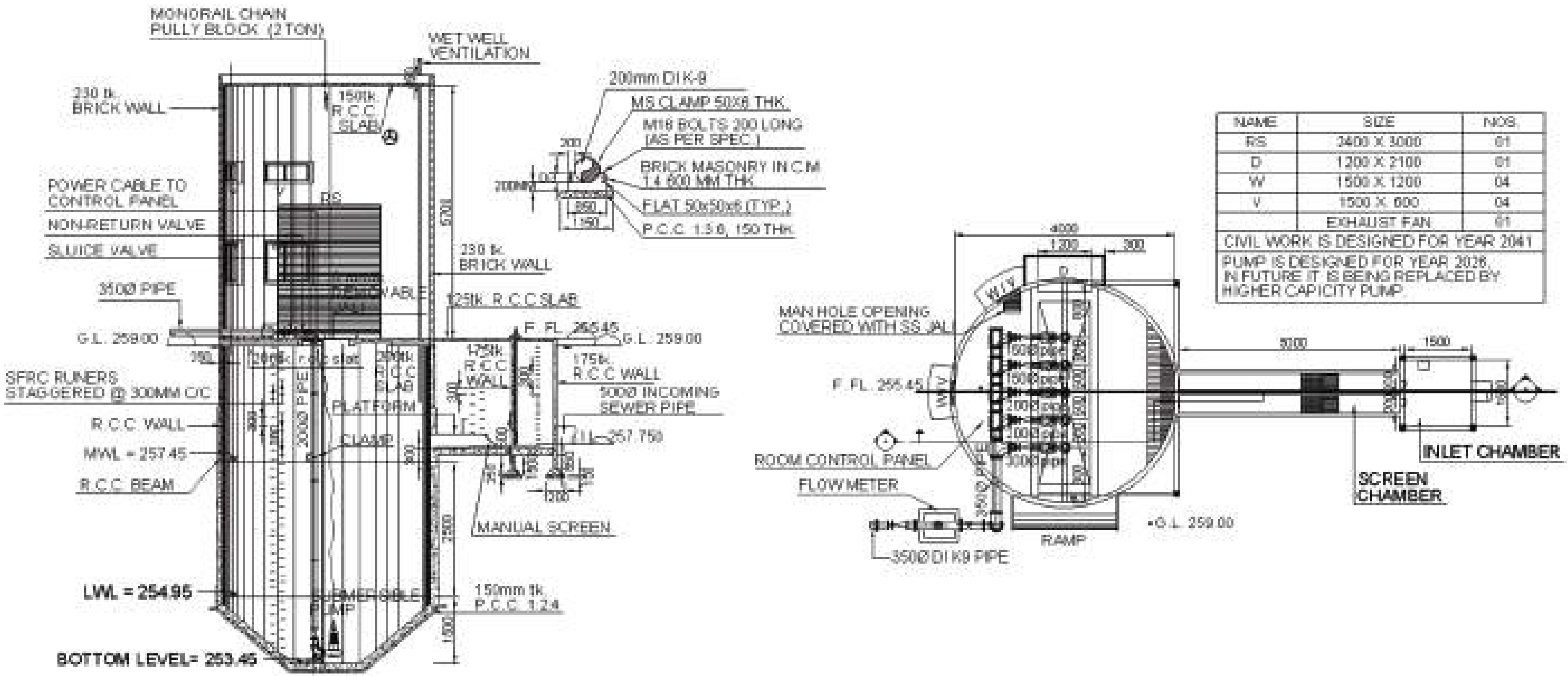
However, pumps to be mounted vertically are supplied as sub-assembled. For the installation of these pumps, the proper sequence of assembly has to be clearly understood from the drawing of the pump manufacturer.

The installation of a pump should proceed through five stages in the following order:

1. Preparing the foundation and fixing the foundation bolts
2. Fixing the pump on the foundation bolts, however resting on levelling wedges, which permit not only easy levelling but also space for filling in the grout later on
3. Levelling
4. Grouting
5. Alignment

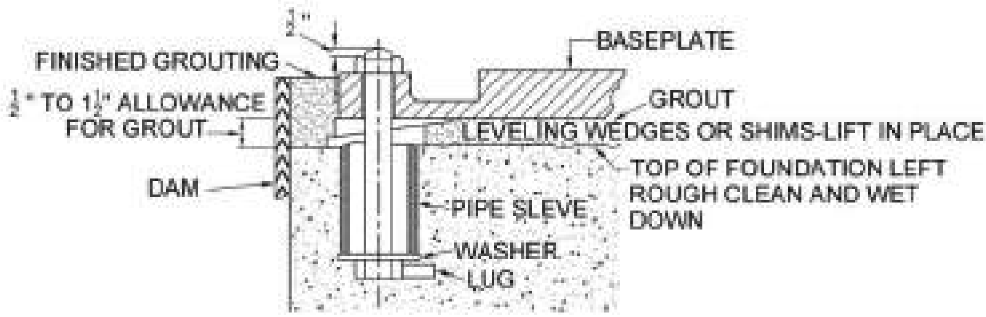
The foundation should be sufficiently substantial to absorb vibrations and to form a permanent, rigid support for the base plate.

A typical foundation is illustrated in Figure 4.13.



These are normally used for lifting the sewage in the sewers at intervals to save the ultimate depth of cut and laying sewers. The wet well is finished as a bowl to collect and pump out the grit.

Figure 4.12 Illustrative drawing of lift stations



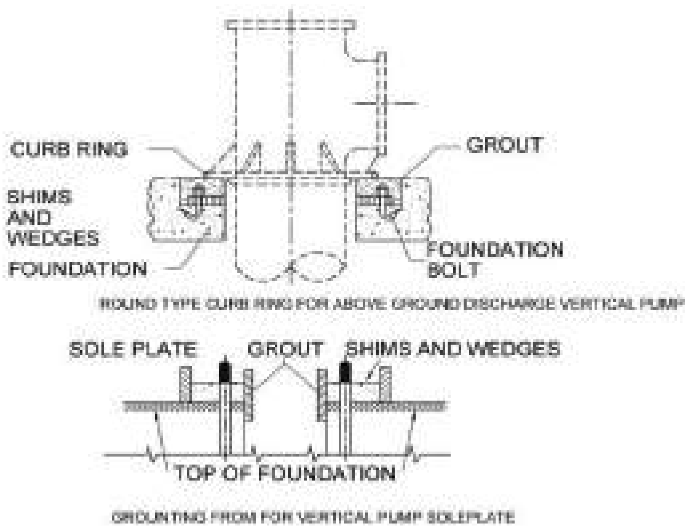
Source: CPHEEO, 1993

Figure 4.13 Typical foundation for a pump

The capacity of the soil or of the supporting structure should be adequate to withstand the entire load of the foundation and the dynamic load of the machinery. As mentioned in clause 6.2.2 and 6.2.3 of IS: 2974 (Part IV), the total load for the pump set and foundation shall include the following:

a) Constructional loads
b) Three times the total weight of the pump
c) Two times the total weight of the motor
d) Weight of the water in the column pipe
e) Half of the weight of the unsupported pipe, connected to the pump-flanges

If the pumps are mounted on steel structures, the location of the pump should be as nearest as possible to the main members (i.e., beams or walls). The sections for structural should have allowance for corrosion also. A curb-ring or a sole-plate with machined top should be used as a bearing surface for the support flange of a vertical wet-pit pump. The mounting face should be machined, because the curb-ring or sole-plate is used to align the pump. Figure 4-14 shows typical arrangement with curb-ring and with sole-plate. Pumps kept in storage for a long time should be thoroughly cleaned before installation.



Source: CPHEEO, 1993

Figure 4.14 Foundation for vertical pumps

Submersible pumps with wet type motors should be filled with water and the opening should be properly plugged after filling the water.

Alignment of the pump sets should be checked, even if they are received aligned by the manufacturer. The alignment should be proper, both for parallelism (by filler gauge) and for coaxiality (by straight edge or by dial gauge). During all alignment-checks, both the shafts should be pressed hard, over to one side, while taking readings. Alignment should be also checked after fastening the piping and thereafter, periodically during operation.

4.20 PUMPING MAINS AND DESIGN APPROACH

These are designed and constructed in the same way as any other water pumping mains. The exception being that the design practice of economical size of pumping mains in conjunction with the electrical energy of the pump sets as used in water pumping mains is not applicable in sewage pumping mains. This is due to varying rates of discharge through the 24 hours like low, average and peak flows through the same main at various parts of the day and night.

4.20.1 Design Formula

The Hazen Williams formula as detailed in Section 3.16.2 of Chapter 3 shall be followed.

There will be pressure losses in fittings which shall be accounted for as in Table 4.2 and illustrated in Appendix A.4.1.

4.20.2 Computation of Pump Kilowatt

This is a function of the static head, friction losses and incidental other losses as illustrated in Appendix A.4.2.

The usual efficiencies of pump sets for estimating the kW requirement can be taken as in Table 4.3.

Table 4.3 Efficiencies of pumps to be adopted for design purposes

No.	Type of Pump Set	Efficiency
1	Horizontal foot mounted centrifugal pump sets	0.85
2	Vertical shaft centrifugal pump sets	0.8
3	Submersible pump sets	0.65
4	Positive displacement pump sets	0.40

In actual practice based on the manufacturer’s pump curves and duty point, the figures may vary and here again, the figures will vary from manufacturer to manufacturer and hence, suffice to state that for design purposes, these figures shall be used.

The kW of a pump shall be calculated as

$$\text{kW required} = Q \times H / 100.5 / ή \qquad (4.3)$$

where,

Q : Discharge in litres per second

H : Total head to be got over in m

ή : Efficiency of the pump

This is usually called the brake horse power. The actual horse power is to include the efficiency of the motor. This is about 0.95 for modern new motors and 0.9 for motors nearing their life cycle of 15 years. Thus, the actual kW needed shall be taken for design purposes as

Actual kW = Brake horse power in kW / 0.9

4.20.3 Velocity Considerations in Design of Pumping Mains

The US EPA suggests that pumping mains designed for velocities between 0.6 to 2.4 m/s are normally based on the most economical pipe diameters and typical available heads. For shorter pumping mains of less than 600 m and low lift requirements of less than 10 m, the recommended design force main velocity range is 1.8 to 2.7 m/s. This higher design velocity allows the use of smaller pipe, reducing construction costs. Higher velocity also increases pipeline friction loss resulting in increased energy costs.

The maximum velocity at peak conditions is recommended not to exceed 3 m/s. In the case of water pumping mains, economical size of pumping mains is calculated by trying out various sizes and finding out the net present value of the capital costs of pipeline and pumping machinery and capitalized electrical energy costs. In the case of sewage, this is not possible because of the complexity of varying pumping rates during lean flow, average flow and peak flows resulting in near impossibility of doing the economical size calculations.

Hence, the rule of thumb is recommended whereby the maximum velocity in peak flow does not exceed 2.7 m/s and the minimum velocity at low flows is not less than 1 m/s.

A judicious selection of the pipe diameter is implied in dealing with sewage pumping mains. The reason for recommending the minimum velocity as 1 m/s is based on the fact that sewage in India invariably brings in considerable grit and even though grit removal is provided in pumping stations, there can be times when either the equipment is under repair or the grit actually passes through at peak flows. When the peak flow tapers off it accumulates in the pipeline and reduces the sectional area and higher velocities are needed if the net pumping flow is the low flow conditions.

A case study of a pumping main evaluated by the WHO/UNDP at Chennai way back in 1979, itself using Fluorometer studies illustrates the theory as in Appendix A.4.3 and is a rare piece of literature. Sewage pumping mains of especially RCC can suffer corrosion by hydrogen sulphide gas, which forms and gets liberated inside these mains due to the velocity conditions.

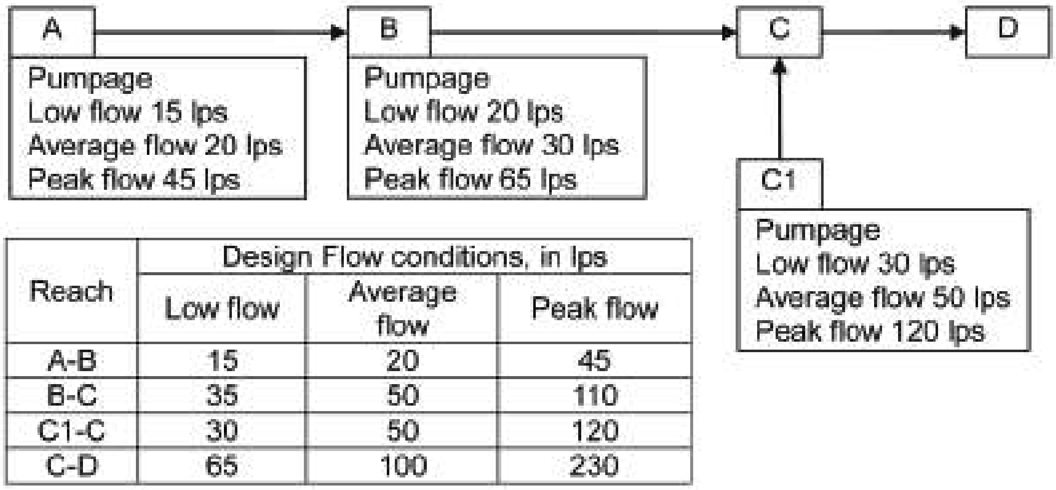
Whenever the velocities are too small, the organic materials get settled out and undergo anaerobic decay and release the sulphide, which later combines with the moisture and forms sulphurous and sulphuric acid.

The effect of velocity and relative sedimentation of organics and grit is shown in Appendix A.4.3. Thus, ensuring of velocities at not less than 0.8 m/s barest minimum and not exceeding 3 m/sec at any time has to be the criterion.

The example in Appendix A.4.4 explains the interpretations of these through the entire 30 years period by considering segments of each 10 years.

4.20.4 Injection and Relay Pumping Mains

Often sewage pumping mains themselves get injected one into another. This is designed on the same principles of design as in Appendix A.4.4 applied to each sequential section starting from the farthest origin of the pumping and add the respective low, average and peak flows for each successive section and arrive at the sizes of pipelines along the “spine” as shown in Figure 4.15.



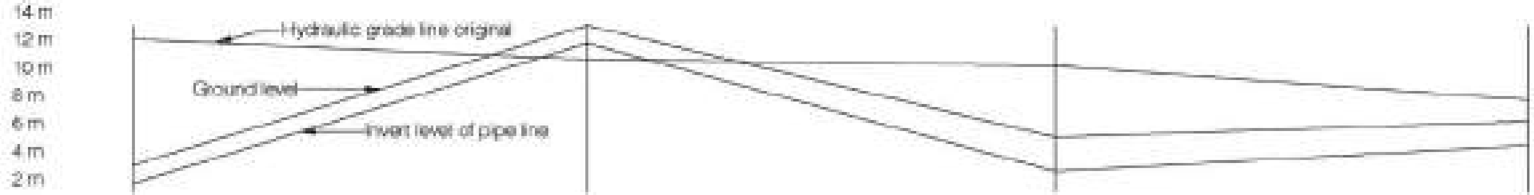
Reach	Design Flow conditions, in lps		
	Low flow	Average flow	Peak flow
A-B	15	20	45
B-C	35	50	110
C1-C	30	50	120
C-D	65	100	230

Figure 4.15 Illustration of pumping main hydraulics on serial pumping mains

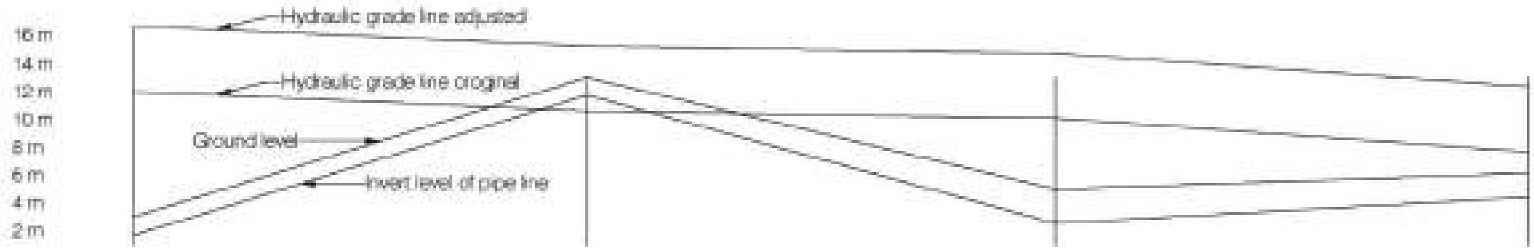
a) The first step is to calculate the friction loss and fittings loss in the “spine” pipe line which will be A-B-C-D with incrementing flows in each segment for a given pipeline diameter by using Appendix 4.4 for each segment and adding up all these from A to D and establish the preferred diameter from velocity considerations.

b) The next step is to plot the hydraulic head line, the ground level and invert level line on Y scale from A to D on a two dimensional scale on Y scale.

c) The next step is to mark the delivery level at D and connect backwards by the losses and verify the hydraulic grade line is above GL by at least 2 m and if not, raise it by 2 m above GL at the crown point. The hydraulic elevations at B, C and D are the delivery levels for pumps at A, B and C1. This is explained in Figure 4.16 overleaf.



Original condition							
Hydraulic Head	13-4=9		11.5-13=(-)1.5		11-6=5		8.5-7=1.5
Hydraulic level	13		11.5		11		8.5
Friction loss in the reach		1.5		0.5		2.5	
Delivery level							6.5
Invert of pipe, m	2.7		10.7		3.55		5.55
Earth cover in the reach, m		1		1		1	
Dia. of pipe in the reach, m		0.3		0.3		0.45	
Ground level, m	4		12		6		7
Location	A		B		C		D



Adjusted condition							
Hydraulic Head	16.5 - 4 = 12.5		15 -13 = 2		14.5 - 6 = 8.5		12-7 =5
Hydraulic level	15 + 1.5 = 16.5		11.5+1.5+2=15m		15 - 0.5 = 14.5		14.5 - 2.5 = 12.0
Friction loss in the reach		1.5		0.5		2.5	
Delivery level							6.5
Invert of pipe, m	2.7		11.7		3.55		5.55
Earth cover in the reach, m		1		1		1	
Dia. of pipe in the reach, m		0.3		0.3		0.45	
Ground level, m	4		13		6		7
Location	A		B		C		D

In the original condition the hydraulic grade line cuts into the ground level at location B by 1.5 m and leading to cavitation in the pipe line.
In the adjusted condition the hydraulic grade line is lifted by the 1.5 m to avoid cavitation and additional 2 m safety is introduced.

Figure 4.16 Illustrative hydraulics of relay pumping mains in Figure 4.15

4.21 ANTI VORTEX

A vortex is a phenomenon whereby when a liquid is sucked into a suction end of the pump set, air is also drawn due to a vortex formation. This can be caused in both vertically downward suction as well as vertically upward suction. The result is the pumped sewage will be having an air-sewage mixture and thus, in fact it will aid imparting oxygen to the sewage which is beneficial. However, the problem is because of the turbulence induced, dissolved gas like sulphide if already present in the sewage can get stripped and the discharged end may have a perceptible concentration which may be offensive. Hence, anti-vortex attachments are normally used in the suction end, which breaks up the formation of the vortex. The simpler version is the attachment of a circular orifice plate of sufficient annulus width for upward suction pipes as in Figure 4.17.

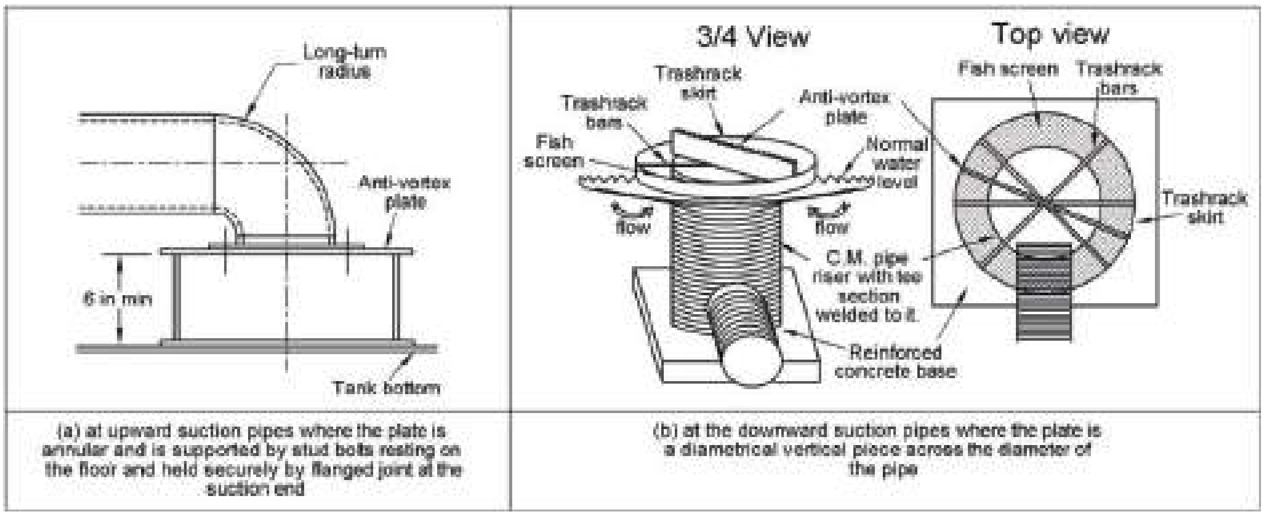


Figure 4.17 Typical anti vortex plate

There are many other variations and can be sourced from the market.

CHAPTER 5: DESIGN AND CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

5.1 GENERAL

Sewage is 99 % water carrying domestic wastes originating in kitchen, bathing, laundry, urine and night soil. A portion of these goes into solution. The remaining goes into colloidal or suspended stages. It also contains salts used in cooking, sweat, bathing, laundry and urine. It also contains waterborne pathogenic organisms from the night soil of already infected persons. The concentrations are mentioned in Table 5.1

Table 5.1 Contribution of human wastes in grams per capita per day

Parameters					Range
1	Biochemical oxygen demand, BOD				45-54
2	Chemical oxygen demand, COD				1.6-1.9 times BOD
3	Total organic carbon, TOC				0.6-1.0 times BOD
4	Total solids, TS				170-220
5	Suspended solids, SS				70-145
6	Grit (inorganic,0.2 mm and above)				5-15
7	Grease				10-30
8	Alkalinity as calcium carbonate ($CaCO_3$)				20-30
9	Chlorides				4-8
10	Total nitrogen N				6-12
11	Organic nitrogen				~0.4 total N
12	Free ammonia				~0.6 total N
13	Nitrate				~0.0-0.5 total N
14	Total phosphorus				~0.6-4.5
15	Organic phosphorus				~0.3 total P
16	Inorganic(ortho- and poly-phosphates)				~0.7 total P
17	Potassium(as potassium oxide K_2O)				2.0-6.0
Microorganisms in 100 ml of sewage					
18	Total bacteria	10^9-10^{10}	22	Protozoan cysts	Up to 10^3
19	Coliforms	10^9-10^{10}	23	Helminthic eggs	Up to 10^3
20	Faecal streptococci	10^5-10^6	24	Virus (plaque forming units)	10^2-10^4
21	Salmonella Typhosa	10^1-10^4			

Source: Arceivala, 2000

Note:

1. The wastewater from toilets is usually referred to as black water and the rest of the wastewater from all other activities is referred to as grey water.
2. As already cited in chapter 1, about 12.6 % of population are still not having toilets and practice open defecation. Their grey water somehow gets into sewers by way of open drains discharging into sewers.
3. Thus, the BOD of raw sewage has to be foreseen realistically because this dictates the cost of the STP almost pro-rata.

continued in next page

4. The difference between total solids and suspended solids is the dissolved solids. When calculating its concentration, the dissolved solids in the freshwater used by the ULB must be added to arrive at the values in raw sewage.
5. The raw sewage pH generally ranges between 6.8 to 8.0 depending on raw water quality.
6. The major nitrogen compound in domestic waste is urea $CO(NH_2)_2$, which is readily hydrolyzed to ammonia (NH_3) and carbon dioxide (CO_2) by the enzyme urease present in sewage. Hence, NH_3 constitutes the major fraction of total nitrogen in domestic sewage.

When the treated sewage is discharged into the rivers, the ratio of the respective flows decides the concentration of these parameters in the blended river water. The quality of surface waters for specified uses are shown in Table 5.2

Table 5.2 Use based classification of surface waters in India
(All values are in mg/l unless otherwise specified therein)

Class	Designated best use	Criteria	Limits
A	Drinking water source without conventional treatment but after disinfection	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	6 or more
		BOD	2 or less
		Total Coliform MPN / 100 ml	50 or less
B	Outdoor bathing (organized)	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	5 or more
		BOD	3 or less
		Total Coliform MPN / 1000 ml	50 or less
C	Drinking water source with conventional treatment followed by disinfection	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	4 or more
		BOD	3 or less
		Total Coliform, MPN / 1000 ml	5000 or less
D	Propagation of wild life and fisheries	pH	6.5 to 8.5
		Dissolved Oxygen (D O)	4 or more
		Free Ammonia	1.2 mg/l or less
E	Irrigation, industrial cooling, and controlled waste disposal	pH	6.0 to 8.5
		Electrical Conductivity, micro mhos/cm	< 2250
		Sodium Absorption Ratio (SAR)	< 26
		Boron	< 2 mg/l

Source: The Environment (Protection) Rules, 1986 in http://cpcb.nic.in/GeneralStandards.pdf

The objective of sewage treatment is to reduce the polluting substances to (a) the standards laid down by the Ministry of Environment and Forests (MoEF) of the Government of India (GOI) and these cannot be relaxed by the State Pollution Control Boards (PCB), but they can prescribe more stringent standards specific to the discharge environment and (b) the specified limits of faecal coliforms laid down by the National River Conservation Directorate (NRCD). These standards are compiled and presented in Table 5.3.

Table 5.3 General standards for Discharge of Environmental Pollutants, Part A: Effluents as per Schedule VI of the Environmental (Protection) Rules 1986 and National River Conservation Directorate Guidelines for Faecal Coliforms, (Values in mg/l unless stated)

No	Characteristics	Standards			
		Inland Surface Water	Public Sewers, (A)	Land for Irrigation	Marine Coastal Areas
1	Colour and odour	(B)		(B)	(B)
2	SS	100	600	200	(C), (D)
3	Particle size of SS	(E)	-	-	(F), (G)
4	pH value	5.5 to 9.0			
5	Temperature	(H)	-	-	(H)
6	Oil and grease	10	20	10	10
7	Total residual chlorine	1.0	-	-	1.0
8	Ammoniacal nitrogen (as N)	50	50	-	50
9	Total Kjeldahl Nitrogen, (TKN) (as N)	100	-	-	100
10	Free ammonia (as NH_3)	5.0	-	-	5.0
11	Biochemical Oxygen Demand	30	350	100	100
12	Chemical Oxygen Demand	250	-	-	250
13	Arsenic (as As)	0.2			
14	Mercury (as Hg)	0.01	0.01	-	0.01
15	Lead (as Pb)	0.1	1.0	-	2.0
16	Cadmium (as Cd)	2.0	1.0	-	2.0
17	Hexavalent Chromium (as Cr 6+)	0.1	2.0	-	1.0
18	Total Chromium (as Cr)	2.0	2.0	-	2.0
19	Copper (as Cu)	3.0	3.0	-	3.0
20	Zinc (as Zn)	5.0	15.0	-	15.0
21	Selenium (as Se)	0.05	0.05	-	0.05
22	Nickel (as Ni)	3.0	3.0	-	5.0
23	Cyanide (as CN)	0.2	2.0	0.2	0.2
24	Fluoride (as F)	2.0	15.0	-	15.0
25	Dissolved phosphates (as P)	5.0	-	-	-
26	Sulphide (as S)	2.0	-	-	5.0
27	Phenolic compounds (as C_6H_5OH)	1.0	5.0	-	5.0
28	Radioactive materials				
	Alpha emitters, micro curie/L	10^{-7}	10^{-7}	10^{-8}	10^{-7}
	Beta emitters, micro curie/L	10^{-6}	10^{-6}	10^{-7}	10^{-6}
29	Bio-assay test	(I)			
30	Manganese (as Mn),	2.0	2.0	-	2.0
31	Iron (as Fe),	3.0	3.0	-	3.0
32	Vanadium (as V),	0.2	0.2	-	0.2
33	Nitrate Nitrogen (as N),	10.0	-	-	20.0
34.	Faecal Coliform, MPN/100 ml for discharge	onto land		into water	
		(J)	(K)	(J)	(K)
		1,000	10,000	1,000	10,000

Explanations of notations are given in next page

A. These standards shall be applicable only if such sewer leads to a secondary treatment including biological treatment system; otherwise the discharge into sewers shall be treated as discharge into inland surface waters.

B. All efforts should be made to remove colour & unpleasant odour as far as practicable.

C. For process wastewater 100 mg/l

D. For cooling water effluent 10% above total suspended matter of influent.

E. Shall pass 850 micron IS Sieve

F. Floatable solids max. 3 mm

G. Settleable solids max. 850 microns

H. Shall not exceed 5°C above the receiving water temperature

I. 90 % survival of fish after 96 hours in 100 % effluent

J. Desirable

K. Maximum permissible

Source: The Environment (Protection) Rules, 1986 in http://cpcb.nic.in/GeneralStandards.pdf

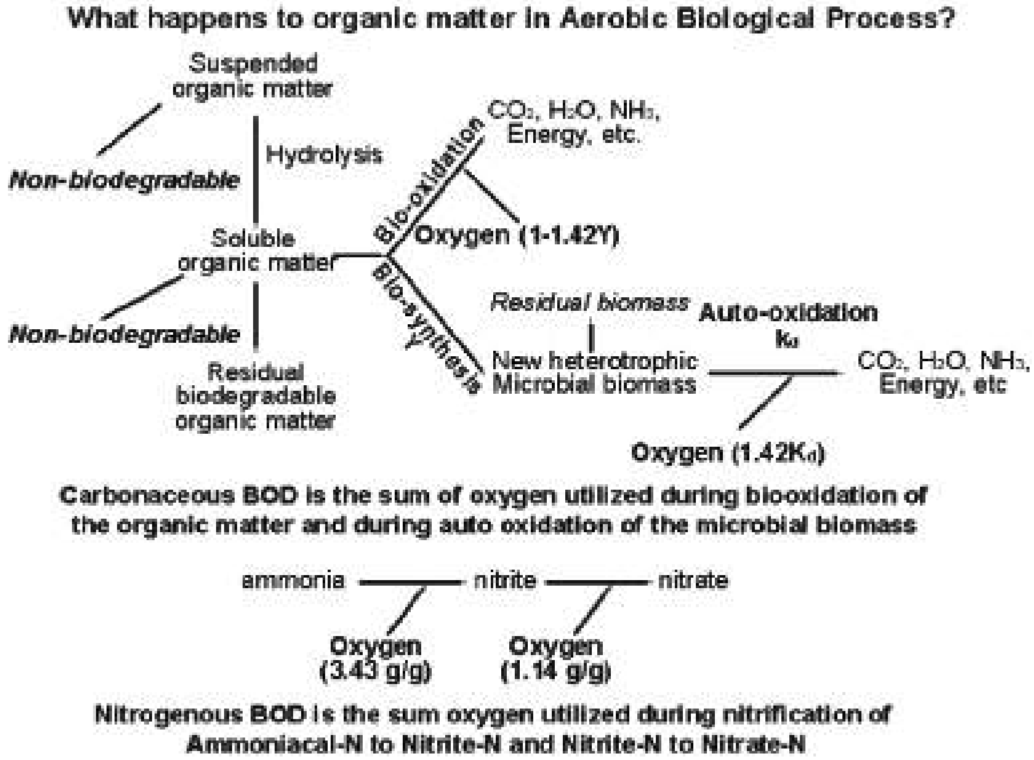
In respect of standards specific for treated sewage discharge into surface waters, the values of BOD not exceeding 20 mg/L and SS not exceeding 30 mg/L have been of historical origin. However, this manual recommends the guidelines for treated sewage if discharged into such surface waters used as a source of drinking water as (1) BOD not exceeding 10 mg/L, (2) SS not exceeding 10 mg/L, (3) Total Nitrogen as N not exceeding 10 mg/L, (4) Dissolved Phosphorous as P not exceeding 2 mg/L and (5) Faecal coliforms not exceeding 230 MPN / 100 ml. More details of these can be seen in Table 5-20.

5.1.1 Recommended Exclusive Discharge Guidelines

The recommended guidelines for treated sewage discharge into surface water which after some travel may be used as a source of drinking are mentioned in Table 5-20.

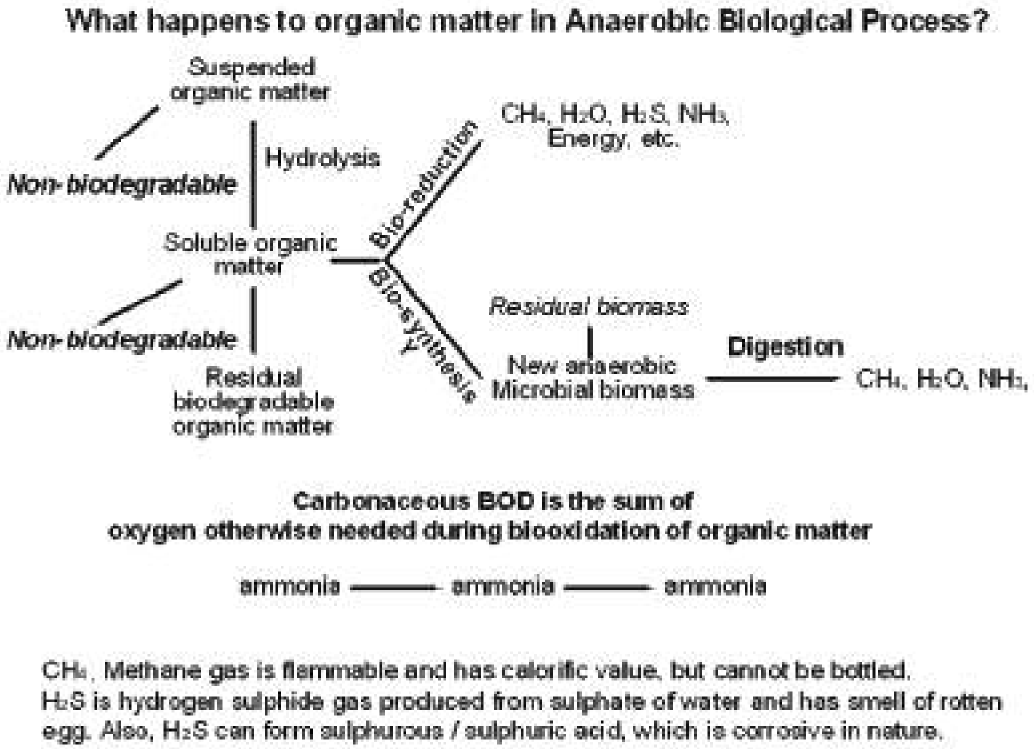
5.1.2 Process of Biological Sewage Treatment

Biological sewage treatment is a process where biological organisms are cultured and allowed to consume the organic matter and multiply their population. This is by a process where the organisms secrete enzymes through their cell walls; and these enzymes solubilize the organic matter and the solution is drawn back into the organisms again through their cell wall. This is the food. The organisms grow and multiply by a process called binary fission whereby each organism splits into two complete new organisms. This is called metabolism. The multiplied organisms are settled out and the clear treated sewage is free from the organic matter. The metabolism can be by (a) aerobic organisms needing oxygen like human beings or (b) anaerobic organisms that do not need oxygen. The pathways are shown in Figure 5.1 and Figure 5.2 overleaf.



Source: Dr. Akepati S. Reddy, Thapar Centre for Industrial Research & Development, Punjab

Figure 5.1 Aerobic metabolism

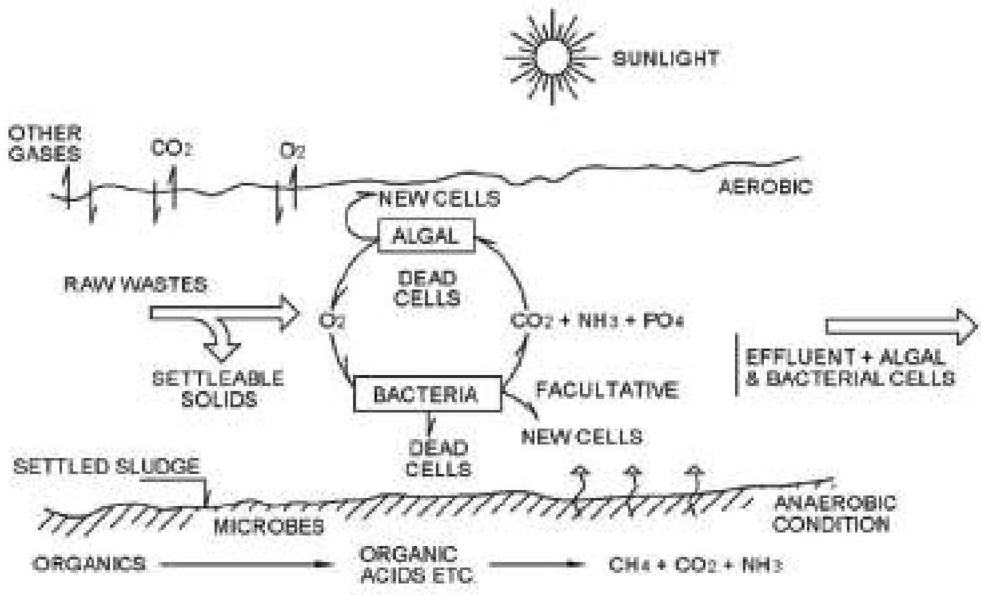


Source: Dr. Akepati S. Reddy, Thapar Centre for Industrial Research & Development, Punjab

Figure 5.2 Anaerobic metabolism

The settled and separated organisms are again put through aerobic or anaerobic processes where their own protoplasm is the reserve food and is referred to as aerobic digestion or anaerobic digestion. The anaerobic digestion is preferred as it yields valuable methane gas, as a source of thermal energy to generate electricity. The digested remains are referred to as digested sludge and can be disposed off as soil filler.

There is also another process of treatment known as facultative where both the aerobic and anaerobic processes occur simultaneously. This is confined to stabilization ponds where the upper portion is aerobic and the settled sludge undergoes anaerobic process at the pond bottom as in Figure 5.3.



Source: CPHEEO, 1993

Figure 5.3 Facultative Metabolism

The digested organisms are removed once in many years when it reaches about 30 % of the liquid height of the pond and are disposed of as soil filler in the dry summer months.

There are also shallow ponds which are fully aerobic and deep ponds which are fully anaerobic.

5.1.3 Design Sewage Flow

Once the DPR is approved, it takes about three to four years to complete the construction. This completed year is referred to as the base year. Hence, the design population and design volume of sewage shall be taken as the values in the base year.

5.1.4 Raw Sewage Characteristics

5.1.4.1 Determination of Influent Raw Sewage Quality

The value of BOD may vary from place to place due to various prevailing socio-economic conditions, etc. The values may be ascertained for the specific situation with suitable documented justification with laboratory analysis data based on the following procedure.

The raw sewage characteristics are a function of level of water supply and per capita pollution load. Thus, the level of water supply plays a major role in deciding the concentration of pollutants. Other significant factors are settlement and decomposition in sewers under warm weather conditions, partially decomposed sewage from septic tanks, lifestyle of the population, etc. The best way to ascertain the sewage characteristics is to conduct the composite sampling once a week for diurnal variation on hourly basis from the nearby existing sewage outfall or drain.

Considering a four-week month, three samples are to be taken on weekdays, whereas the fourth sample is to be taken on an off day i.e. Sunday.

Sampling for water quality should be conducted for at least one month during dry weather to assess pollution load quantitatively and qualitatively.

The samples should be analyzed for the following parameters;

pH, Temperature, Colour, Odour, Alkalinity, TSS, Volatile SS, BOD (Total & Filtered), COD (Total and Filtered), Nitrogen (NH3, TKN, NO3), Phosphorus (Ortho-P & T-P), Total Coliforms and Faecal Coliforms, TDS, Chloride, Sulphates, Heavy Metals (if there is a chance of industrial contamination)

The results arising from these analyses shall be adopted with the approval of the competent authority. In the absence of drain or outfall, the Table 5.4 can be referred for new developments for 135 L/cap /day rate of water supply. Depending on the rate of water supply the concentrations can be forecast. Based on the raw sewage quality monitoring experiences, the following typical concentrations can be taken for design purpose for 135 L/Cap /day water supply.

Table 5.4 Concentration of various parameters in the absence of drain or outfall

Item	Per capita contribution (g / c /d)	water supply (L / c / d)	Sewage Generation 80 % of (3)	Concen-tration (mg/L)
(1)	(2)	(3)	(4)	(5)
BOD	27.0	135	108	250.0
COD	45.9	135	108	425.0
TSS	40.5	135	108	375.0
VSS	28.4	135	108	262.5
Total Nitrogen	5.4	135	108	50.0
Organic Nitrogen	1.4	135	108	12.5
Ammonia Nitrogen	3.5	135	108	32.5
Nitrate Nitrogen	0.5	135	108	5.0
Total Phosphorus	0.8	135	108	7.1
Ortho Phosphorous	0.5	135	108	5.0

Illustration BOD = 27 *1000 (mg) / 135 X 0.8 (litres) = 250 mg/L

The Table 5.1 in the manual is retained as a historical value and the Table 5-4 will be followed for the design of biological STPs.

The main reasons for condensing the parameters in the Table 5-4 are

(a) there is a need for listing briefly the parameters of direct relevance to biological treatment processes for BOD removal.

(b) the parameters like total organic carbon, grit, grease, alkalinity chlorides, nitrite, nitrate, potassium are not influencing the biological treatment processes for BOD removal and

(c) the fact that the reduction or elimination of the organisms like coliforms, streptococci, salmonella, protozoa, helminths and virus in these biological treatment processes are incidental and are not specifically designed for.

In general, the required ratio of BOD:N:P is as follows
For aerobic process, BOD:N:P is 100:5:1
For anaerobic process, BOD:N:P is 300:5:1
If the N and P is less, these are artificially added by making a solution of appropriate fertilizers and adding to the raw sewage.

5.2 UNIT OPERATIONS IN BIOLOGICAL TREATMENT

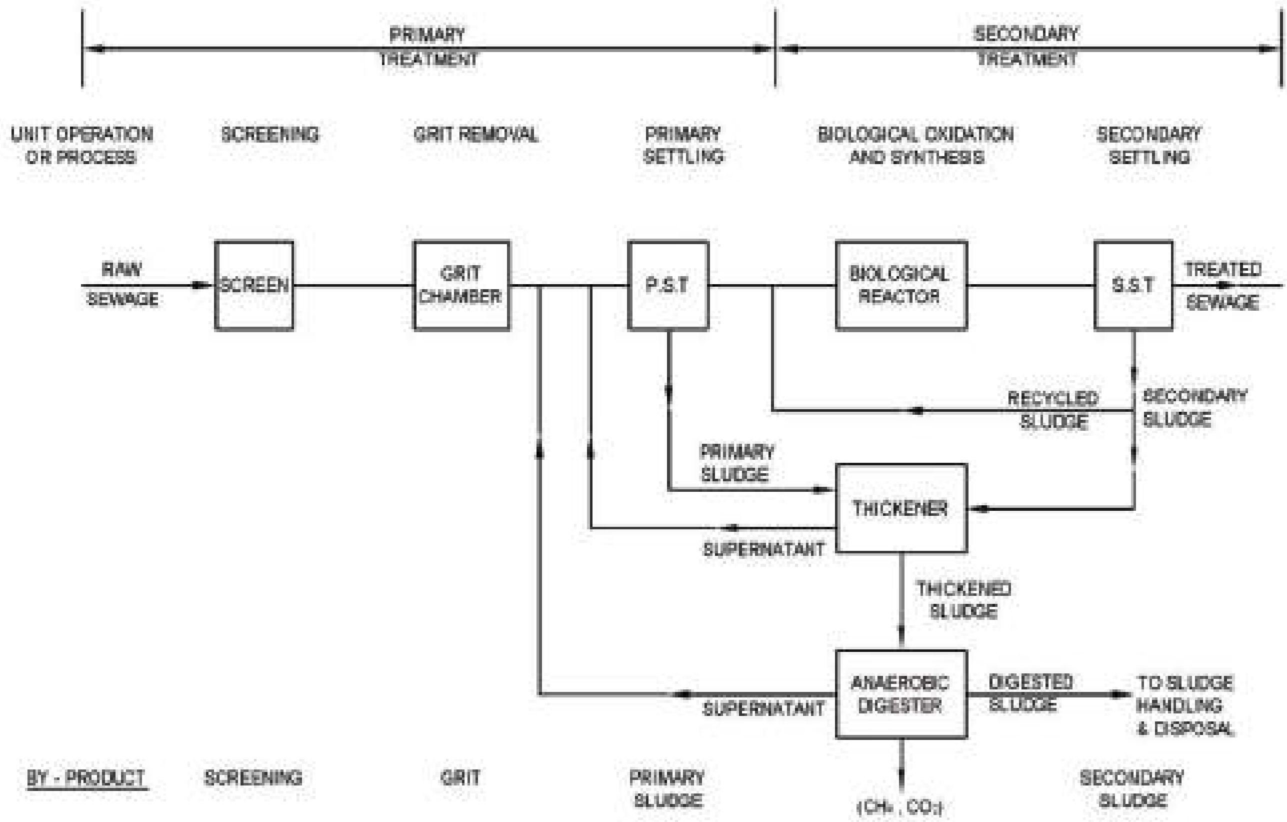
The treatment processes are already explained in Section 5.1. The physical activities used to implement the processes are called unit operations. For example, the physical processes of screening, grit (sand) and suspended solids being settled out are together referred to as primary treatment. The metabolic process is called secondary treatment. Unit operation means the physical activity. For example, simple settling of raw sewage is carried out in primary clarifiers. Pumping air into the sewage for supplying oxygen to the aerobic metabolism is called aeration. Settling of the microorganisms after aeration is carried out in secondary clarifiers. The concentration of settled out organics and microorganisms from primary settling or secondary settling or both together is carried out in sludge thickeners. The anaerobic metabolism of thickened sludge is carried out in sludge digesters. The general sequence is shown in Figure 5.4 and Figure 5.5 overleaf.

5.3 SECONDARY BIOLOGICAL TREATMENT PROCESS

5.3.1 Aerobic Treatment Process

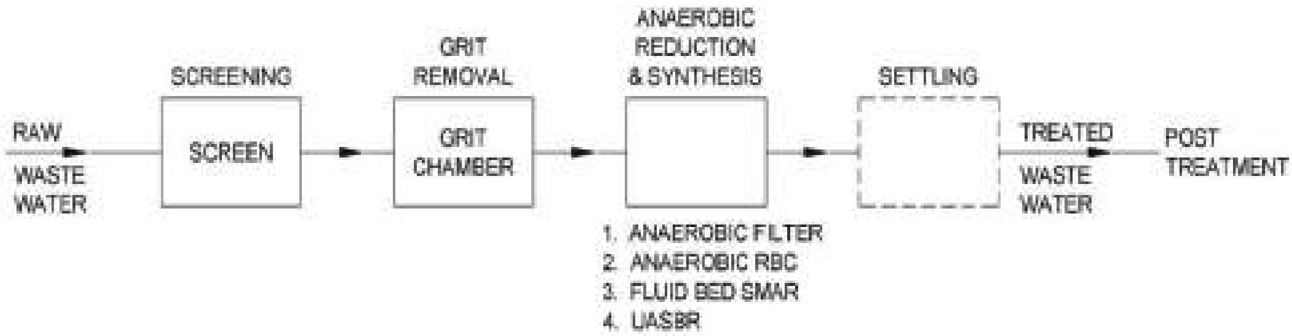
The following treatment processes as also cited in the advisory issued by the Ministry of Urban Development in March-2012 titled “Recent Trends in Technologies in Sewage Treatment” fall under the classification of aerobic treatment

- Activated Sludge Process (ASP)
- Sequencing Batch Reactor (SBR)
- Moving Bed Bio Reactor (MBBR) / Fluidized Aerobic Bioreactor (FAB)
- Membrane Bio Reactor (MBR)



Source: CPHEEO, 1993

Figure 5.4. Unit operations in aerobic mechanized biochemical sewage treatment process (Secondary Treatment can also be extended aeration and without digester)



Source: CPHEEO, 1993

Figure 5.5 Process flow sheet of conventional anaerobic sewage treatment

- BIOFOR – Biological Filtration and Oxygenated Reactor (BIOFOR)
- High Rate Activated Sludge BIOFOR-F Technology
- Submerged Aeration Fixed Film (SAFF) Technology
- Fixed Bed Biofilm Activated Sludge Process (FBAS)
- Fixed media like Rotating Biological Contactor (RBC)
- Oxidation ditch (O D)

5.3.1.1 Activated Sludge Process

There are two variations of this process namely,

(a) the conventional process for removal of BOD and SS alone and

(b) additionally incorporation of biological nitrification & denitrification for removal of nitrogen in the same process.

Within the conventional process, there are other variations as in Figure 5.6. In the case of very small STPs bleeding excess sludge will be a hydraulic challenge and hence mixed liquor can be wasted intermittently.

The conventional system represents the early development of the ASP which is more than 100 years old. (See Box No. 5.1 page 5-219, about Edward Ardern & William T. Lockett, the inventors)

Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives. In step aeration, settled sewage is introduced at several points along the tank length which produces a uniform oxygen demand throughout.

In tapered aeration, air supply is tapered to match the needs from the deeding point of sewage to its exit from the aeration tank.

Contact stabilization provides for reaeration of return activated sludge from the final clarifier, which allows a smaller aeration or contact tank.

While conventional system maintains a plug flow hydraulic regime, completely mixed process aims at instantaneous mixing of the influent waste and return sludge with the-entire contents of the aeration tank.

The extended aeration process employs low organic loading, long aeration time, high mixed liquor suspended solids (MLSS) concentration and low F/M. Because of long detention in the aeration tank / oxidation ditch, the MLSS undergo considerable endogenous respiration and get well stabilized and in these cases, the excess sludge does not require separate digestion and it can be directly dried on sand beds or dewatered in equipments. In addition, the excess sludge production is minimum in this case. The conventional system, the complete mix and the extended aeration have found wider acceptance.

5.3.1.2 Fixed Media System

The primary sedimentation is a pre-requirement in these applications. These are the older trickling filters with stone media and now use synthetic media such as inclined corrugated media placed in cube sized packs and the inclinations changed to opposite directions in successive layers.

The applied sewage is distributed from the top of the media pack by a stationary or hydraulically driven reverse jet arms on opposite radii or rotated by a mechanical drive. This arrangement is needed to apply the sewage on the entire plan area uniformly. This sets up a hydraulic draft and allows the lighter gases of metabolism to escape upwards and fresh air to rush in at the bottom through open ports on the side walls at the floor.

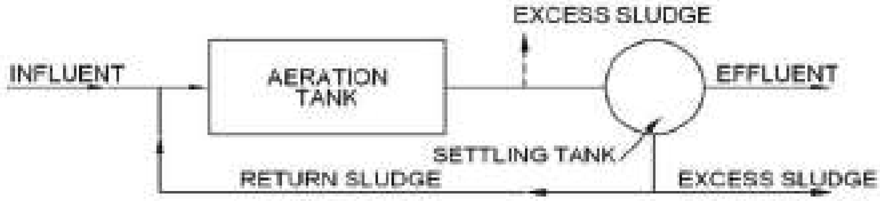


FIG.(a): CONVENTIONAL ACTIVATED SLUDGE

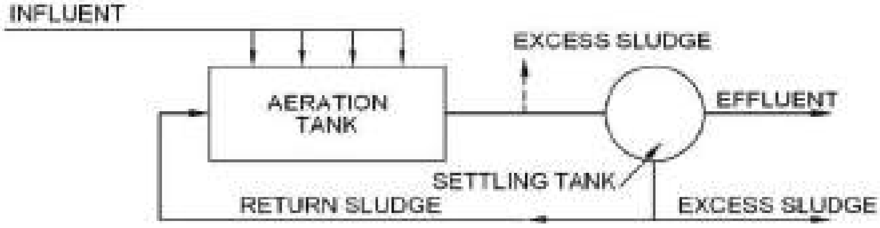


FIG.(b): STEP AERATION

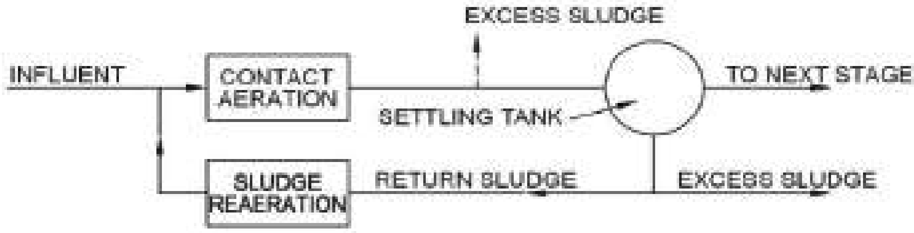


FIG.(c): CONTACT STABILIZATION

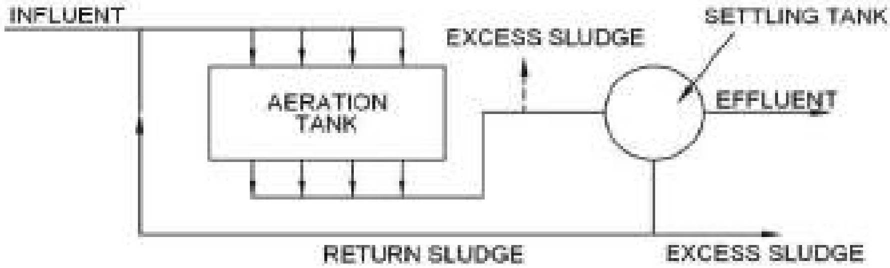


FIG.(d): COMPLETE MIX PLANT

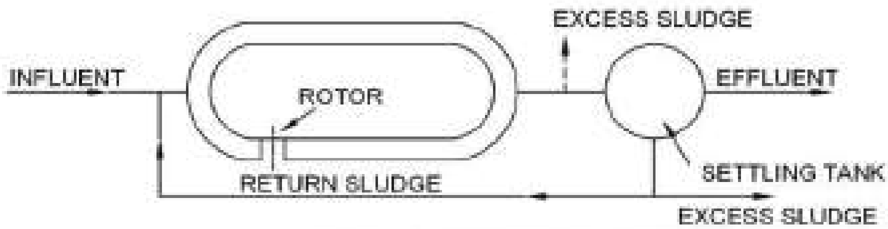


FIG.(e): OXIDATION DITCH

Source: CPHEEO, 1993

Figure 5.6 Schematic diagrams of activated sludge treatment with different modifications

The organisms grow as a film on the fixed media and bring about the metabolism as the sewage passes over them as a film. In due course of time, the thickness of the film increases. This results in the film shearing away from the media which is called sloughing. This is carried away to secondary settling tanks. Recirculation of the settled sewage is sometimes practiced to the inlet of the reactor. This helps to return the enzymes released by the microbes back to the reactor for solubilizing the sewage organic matter. The media should be only made of virgin material like HDPE, PVC. The fixed film media system is in Figure 5.7.

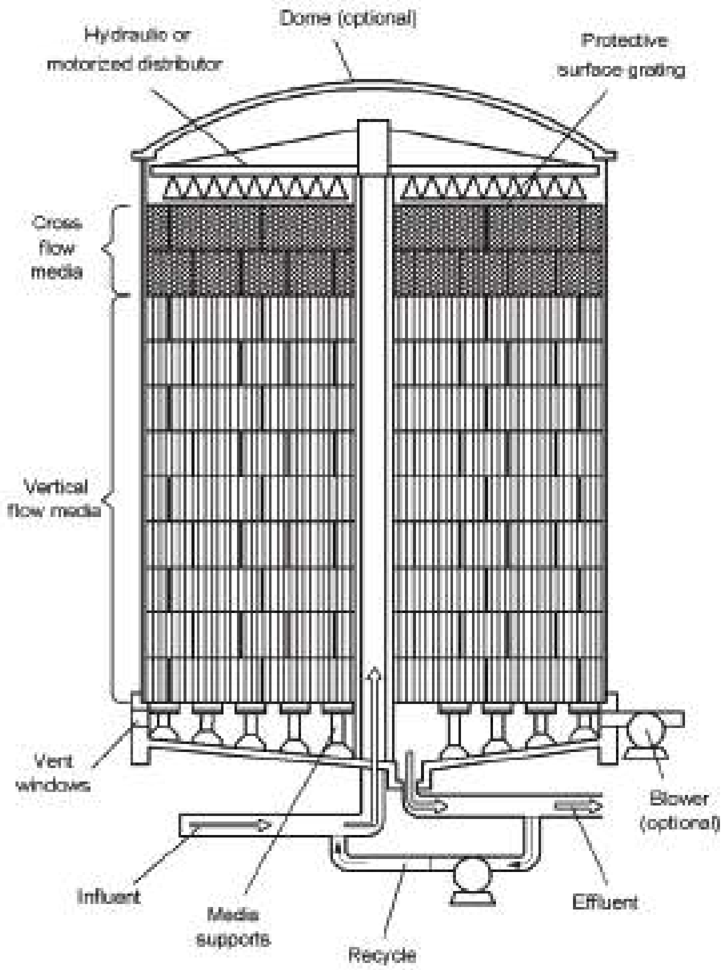


Figure 5.7 Fixed film synthetic media filters

5.3.1.3 Moving Media Systems

These involve the synthetic small sized media, which are fluidized in the reactor by artificial air supply by compressed air released at the floor of the reactor. This brings about the circulatory movement of these media into the tank contents. The trade names are Fluidized Aerobic reactor (FAB), Moving Bed Biological reactor (MBBR) as also Fluidized Anaerobic reactor and their operational principle is illustrated in Figure 5.8.

The microbial film that develops over the surface of the fluidized media permits the metabolism. Secondary settling is needed in the case of FAB and MBBR. In the case of FAB, additional further treatment may also be necessary.

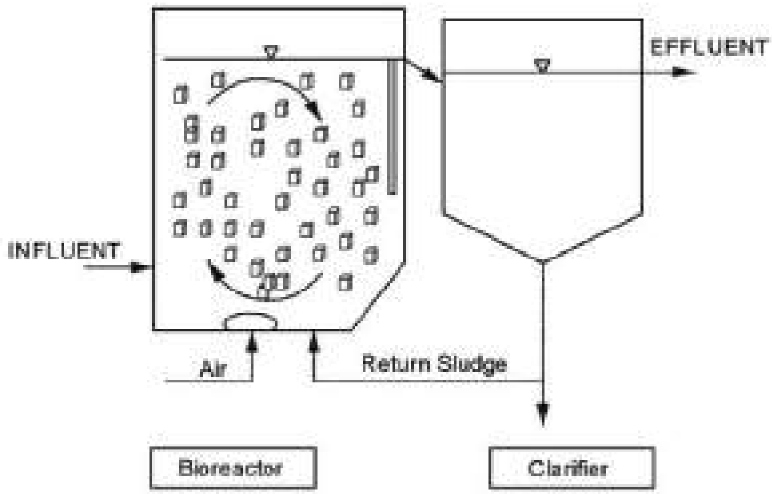


Figure 5.8 Fluidized aerobic bed reactor showing the media in motion

5.3.2 Anaerobic Treatment Systems

The following treatment processes fall under the classification of anaerobic treatment.

a) Up flow Anaerobic Sludge Blanket - UASB
b) Anaerobic filter - AF
c) Anaerobic fluidized bed

These are mainly needed in case where bio-methanation is possible to recover the energy. Instances are the Up Flow Anaerobic Reactor and sludge digesters and the schematic of these are shown in Figure 5.9, Figure 5.10, and Figure 5.11. The principle of anaerobic treatment is already shown in Figure 5.2.

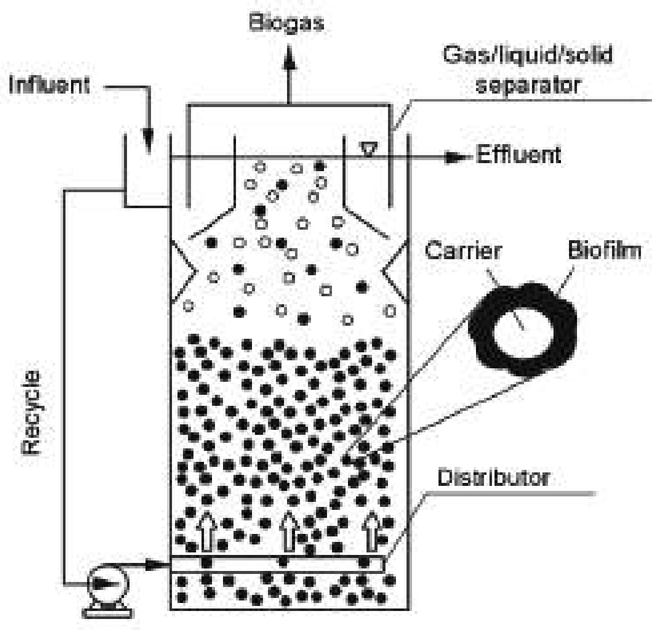


Figure 5.9 Fluidized anaerobic bed reactor showing the media in motion

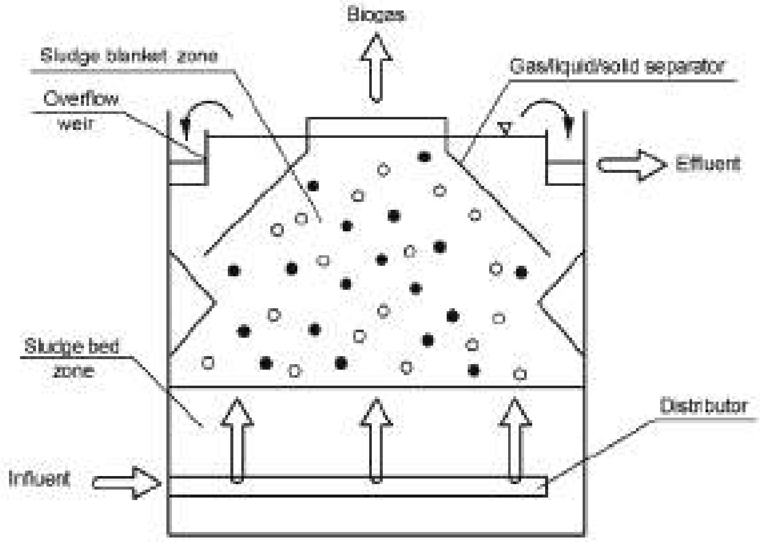


Figure 5.10 Up flow anaerobic sludge blanket reactor

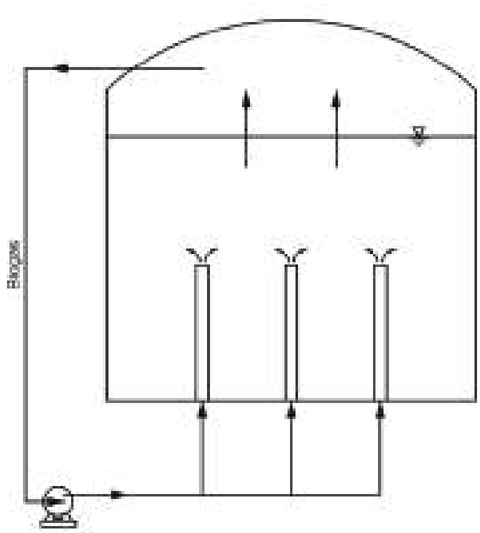


Figure 5.11 Anaerobic sludge digester mixing with generated biogas

5.3.3 Facultative Treatment Processes

The following treatment processes fall under the classification of facultative treatment.

a) Aerated lagoon - AL

b) Waste stabilization pond - WSP

c) Eco Bio Block - EBB

5.3.4 Performance Efficiency of Conventional Treatment Processes

The performance efficiency of the conventional treatment processes are in Table 5.5.

Table 5.5 General Treatment Efficiencies of Conventional Treatment Processes

	Process	Percentage Reduction		
		SS	BOD	Total Coliform
1	Primary treatment (sedimentation)	45-60	30-45	40-60
2	Secondary treatment			
	(i) Activated sludge plants	85-90	85-95	90-96
	(ii) Stabilisation ponds (single cell)	80-90	90-95	90-95
	(iii) Stabilization ponds (two cells)	90-95	95-97	95-98

Source: CPHEEO, 1993

5.3.5 STP Land Area

In recent times, population densities have increased in ULBs. Even public lands are being encroached upon and these become difficult to vacate. Thus getting open vast land areas for setting up STPs is a problem.

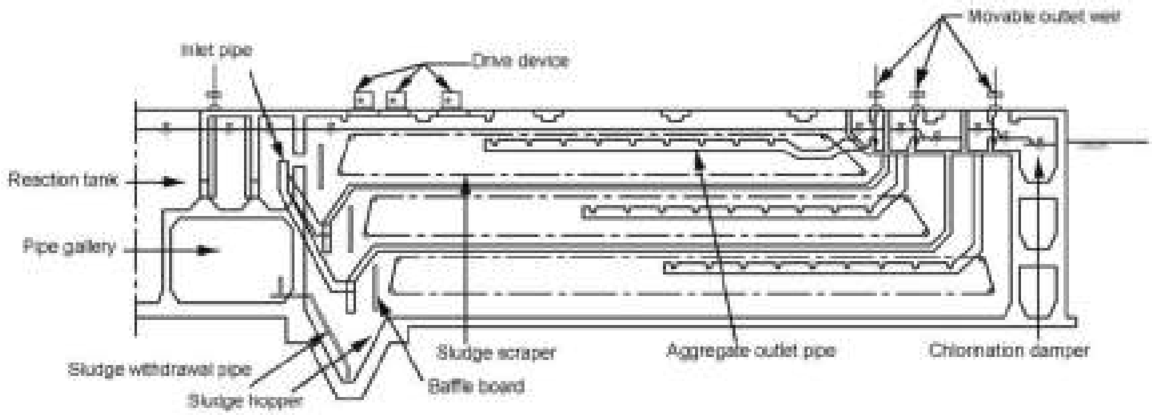
Moreover, because of densification, even if open lands are available, the population is very close to these lands and they object to the STPs nearby. Thus, the STPs have to be planned to occupy lesser areas than the STPs of olden days.

This has brought in the concept of multi-tier STPs where the primary clarifier can be at the topmost floor with aeration tank below it and secondary clarifier at the bottom floor. It is also possible to construct multi-tier STPs like the SBR based STP in Bangkok as in Figure 5.81 (later on in this chapter) whereby land area can be reduced. Similarly, MBR also results in lower area because of higher MLSS concentration and reduced volume of aeration tanks permitting vertical arrangement of tanks one over the other.

An important engineering requirement in such cases is the need to ensure headroom of 4.5 m in between the top of sidewall of the bottom tank and the roof of the upper tank. This is because of the fact that these locations come under industry classification and electrical utilities are involved.

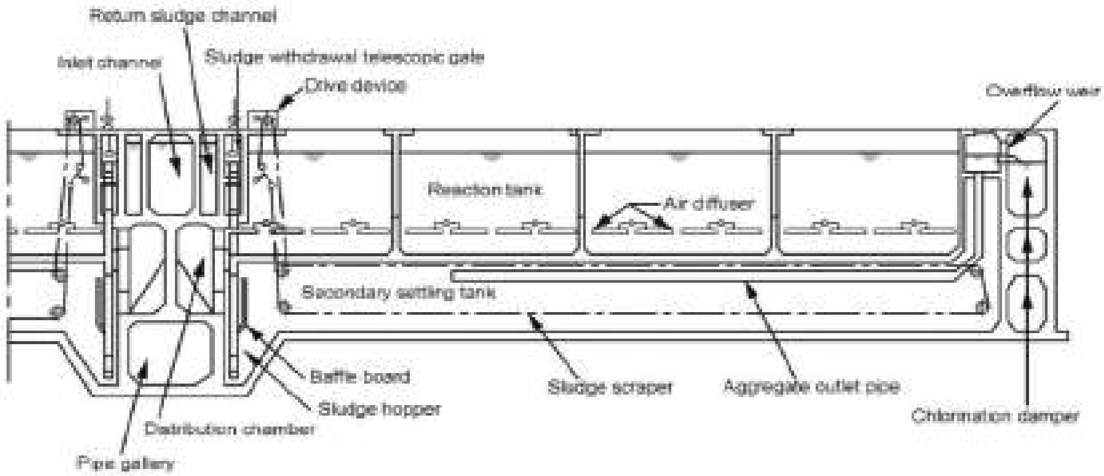
The recurring cost of additional pumping of all the raw sewage over the entire increased height has to be considered in such cases.

There are also similar STPs in Japan (Figure 5.12 and Figure 5.13) which have been built conserving space. These types of facilities have a complicated structure compared to the conventional facilities. Therefore, while making decision on adopting these facilities, the difficulty in O&M of these facilities has also to be well considered.



Source:JSWA, 2009

Figure 5.12 Example of secondary clarifier built in 3 layers (Hirano STP, Osaka, Japan)



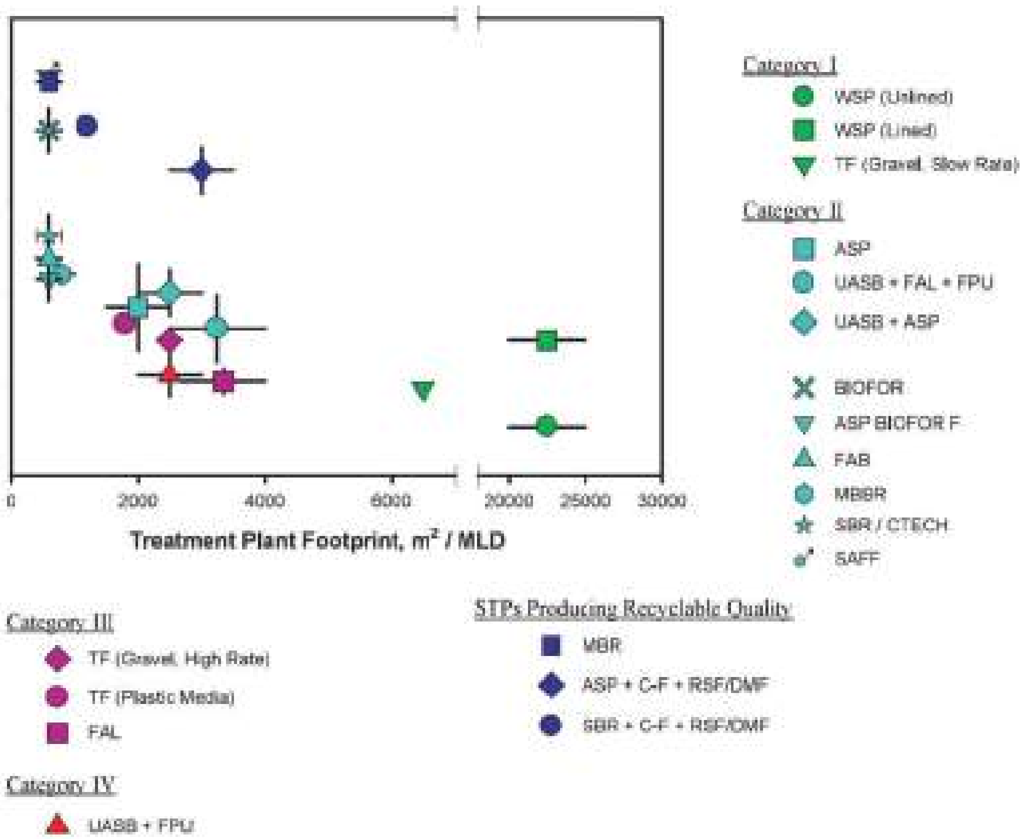
Source: JSWA, 2009

Figure 5.13 Example of reaction tank and secondary clarifier in layers (Imafuku, Osaka, Japan)

5.3.5.1 STP Land Area for different technologies

The STP land area required for various treatment technologies are shown in Figure 5.14. It can be observed that excluding WSP the land area is in the range of 0.2 to 1.0 hectare per MLD for STP as per the technology adopted keeping in view the size of the town / area.

The minimum foot print will also be an important factor in evaluating the treatment technology. This is because in the case of Koyambedu 125 MLD STP at Chennai for the CMWSSB, the cost of the land based on official rates exceeded the cost of the STP itself. This was because the location of the STP site is in the prime hub of metropolitan transport, metro rail, wholesale vegetable market, long distance bus station, etc. This however may not be the case everywhere. Hence, the decision on relative costing of savings in land area is best left to the concerned ULB's depending on the nature of the site and its potential uses.



Source: NRCD, MoEF, 2009

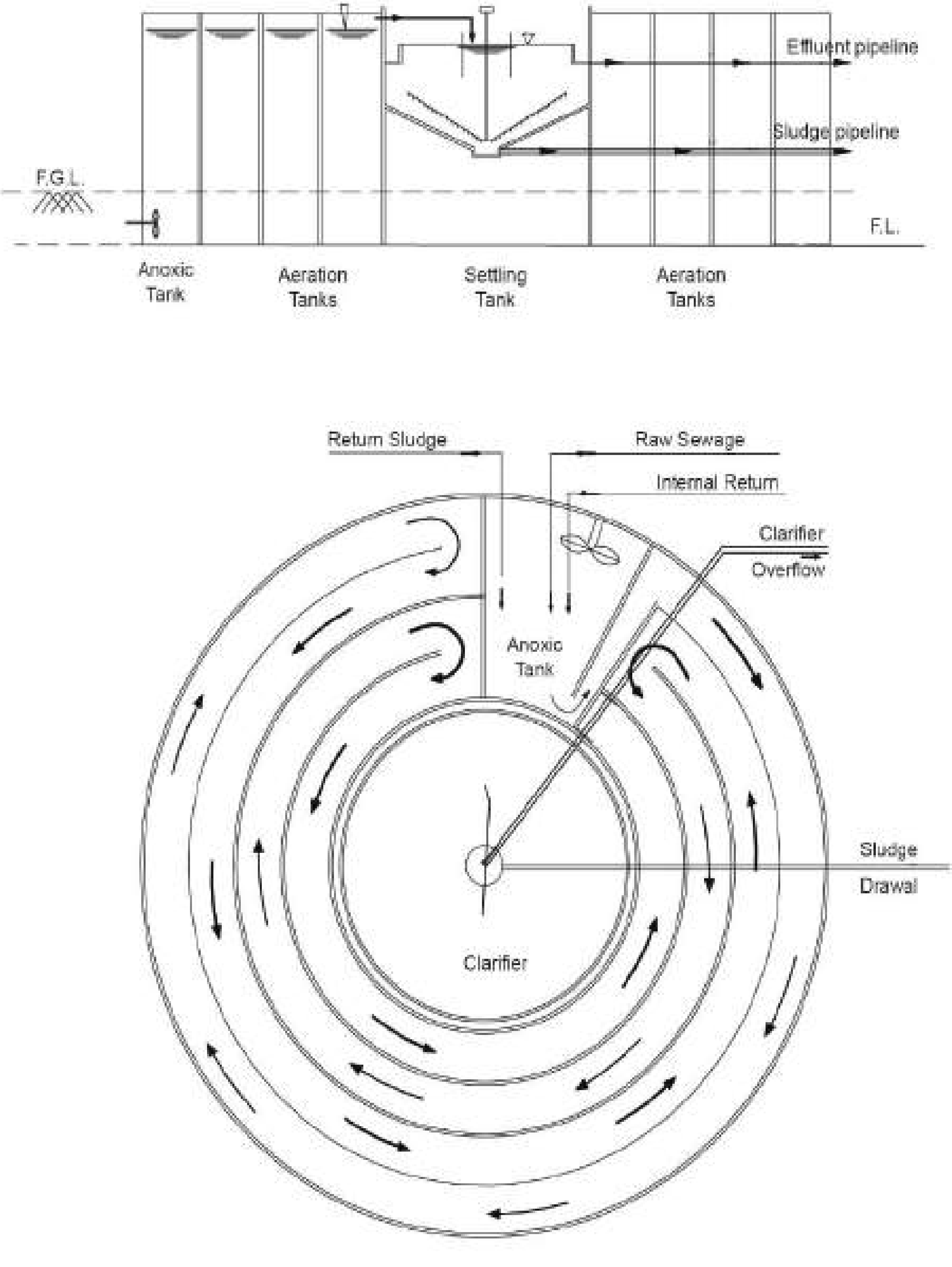
Figure 5.14 STP Land Area required for various treatment technologies

Considering the foregoing, the energy cost, operating cost and capital cost will be the determining factors in DPR's while looking into the technologies. The land cost is to be kept out at the DPR stage.

5.3.6 Arrangement of Treatment Units in STP

The preferred arrangement will have the following guidelines

- The civil construction of units can be integrated by common walls especially sewage holding structures
- This can be either rectangular or square tanks touching each other or circular tanks inscribed concentrically.
- This will economize on costs of piping because sewage can be conveyed by channels through all such units.
- Stop gates operated by hand wheels on a rack and pinion method can replace costly buried valves and avoid the difficulty of O&M of such buried valves.
- An illustration of one such construction is the biological STP at the Indira Gandhi International Airport terminal T3 as illustrated in Figure 5.15.



Source: Indira Gandhi International Aiport, Delhi

Figure 5.15 An illustration of a nitrification-denitrification bioreactor in India

- The anoxic tank, aeration tank and clarifier are all arranged in a single civil structure for the 17.5 MLD STP designed with two parallel modules. Photographic views in construction and in O&M are shown in Figure 5.16


Source:Indira Gandhi International Airport, Delhi

Figure 5.16 Left is the bioreactor during construction; Right is the bioreactor after commissioning showing the rich MLSS culture in the annular plug flow reactor

5.3.7 Control of Hydrogen Sulphide Odour

Whichever be the sewage treatment process that is used, care should be taken to avoid unnecessary stagnation of raw sewage or sludge. At the same time, there are the following locations from which the odour of Hydrogen Sulphide can arise.

a) Sewers that are choked and not flowing,

b) Sewage pumping station sumps where sewage is not pumped out then and there,

c) Primary clarifiers, sludge thickeners, digesters and sludge drying beds in STPs.

The standardizing of technologies for odour control in such locations is yet to be validated. This is due to the factual position that until now, the SPS and STP are located fairly away from the habitations. However, of late with increasing urbanization and densification of land use, the locations of these are almost coming up in the midst of urban living. Hence, it becomes necessary to institute odour control measures in sewerage and sewage treatment systems.

In sewers, keeping the sewage elevation below the crown of the gravity sewers will enable outside air to be drawn and it will rise through the sewers at the rising gradient and thus avoid build up of the foul odours.

In SPS, covering the raw sewage sump is not advised because the submersible pump sets have to raised and lowered for maintenance. The suction of the sewage by the pumps enables the inward flow of the air and this helps in avoiding build up of foul odours.

In the case of STPs, the locations are screen chambers, detritors, equalization tanks, primary clarifiers, sludge thickeners, sludge digesters and sludge dewatering units.

The prevailing technology options for odour control in STPs are compiled in Appendix A.5-1. In essence, the options are in providing a cover over such units to contain the odourous gas and air mixture and then draw it into a gas purification unit and then condition the purified air to be pumped back into the enclosure.

The illustrations of the hydrogen sulphide stripping units described in Figure 6.14 in Chapter 6 can be followed. Their design criteria will be as per the respective equipment manufacturers.

For this purpose, the layout of the various unit operations is best arranged in such a way to group the odour causing units together for a single master enveloping dome instead of multiple domes over each of the units. At the same time, the contained space must permit access to the mechanical equipment for repairs, renewals etc. In general, a circular configuration will be relatively advantageous in economically providing a funicular polygon type of dome using a translucent, synthetic non corrosive light weight material for the dome. The dome itself can be raised free of the structure during the repairs and renewals to provide unhindered access and visibility in work. During the containment position, the free air space above the unit can be minimal so that the volume of air-gas mixture to be purified will be minimal and will permit higher concentration of the gas which will help in better efficiency in purification. One such layout at the Thiruvottoyur 31 MLD capacity STP of CMWSSB now under construction in the peripheral urban location of Chennai is shown in Figure 5.17.

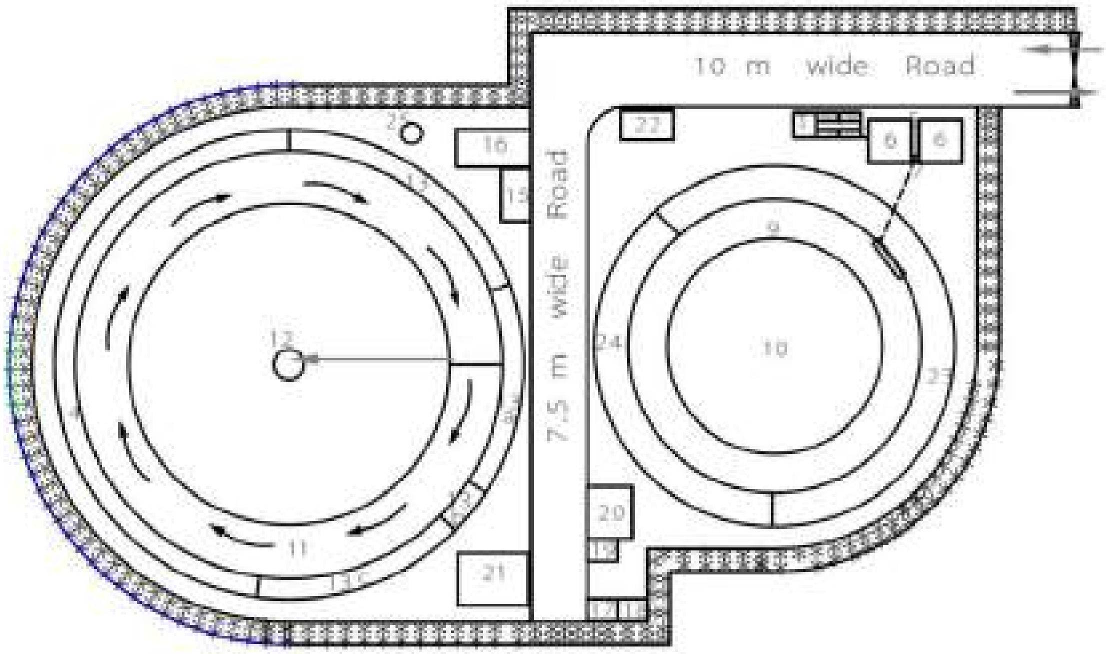


Figure 5.17 Layout of the civil units at the 31 MLD Thiruvottiyur STP of CMWSSB

The circular layout at right incorporates the raw sewage equalization tank (9), primary clarifier (10), sludge thickener (23) and sludge digester (24). The circular unit at left incorporates the aeration tank (11), secondary clarifier (12), tertiary filter (13) sumps for sludge, filtrate etc (13a to 13c) and chlorine contact tank (14). It may be seen that the odour causing units of raw sewage equalization tank, primary clarifier, sludge thickener and sludge digester are laid out into a single integrated structure for easy odour control and additional dome- type enclosure facility in later days.

5.3.7.1 Buffer zone around the STP

Adequate measures may be taken for de-odourization in the STP. The units in the STP which need deodorization system are screen chambers, grit chamber, primary clarifier, sludge thickening and dewatering units. However, the odor emissions are negligible for sludge treatment facilities of extended aeration systems due to in-situ aerobic digestion of sludge. Wherever the STPs are provided with de-odourization system, specific buffer zones are not required.

In case of STP's where de-odorization system cannot be provided, an aerial / peripheral distance of 100 m from the odour-producing units to the habitation is recommended. However, this distance can be reduced by conducting public consultation.

5.3.8 Required Engineering Data

The mandatory data shall include:

a) Contour map and elevations connected to the nearest Survey of India permanent bench mark.

b) Soil tests till it attains hard stratum and test bores at a grid of 500 m squares

c) Rainfall of nearest observatory for at least 50 years

d) Highest intensity of rainfall

e) Earthquake records of the region

f) Maximum flood level at the site and in the identified receiving water

g) Wind rose.

5.3.9 Design Period

This shall be as in Table 2.1of Chapter 2.

5.3.10 Sewers and Conduits

Generally, urban sewerage schemes involve large volumes of sewage. The sewers conveying it to the STPs can be pipes or in-situ constructed conduits passing through dense areas. These conduits shall be preferably non-corrosive materials such as CI or DI pipes.

They can also be RCC pipes with high alumina cement lining and adequate wall thickness to withstand the traffic loads in public roads. Pre-stressed concrete pipes are not recommended.

This is because if the steel is corroded by Hydrogen sulphide from the sewage, it will lose its thickness. This will affect the ability of the steel rods to withstand the pre-stressed tension and they may snap. Then the entire pipe will crack. These cannot be repaired easily at site.

The old brick-arch sewers are good against corrosion, but the challenges are procuring quality bricks, availability of dedicated skilled masons and difficulties of future repairs if needed.

5.3.11 Installation of Mechanical Equipment

The general arrangement (GA) drawing of each mechanical equipment erected in its civil works tank is called the GA drawing. These drawings shall show clearly the way the bolts, nuts etc are securely fixed in the civil works. It is better if the drawing clearly states the sequence of removing the equipment later on, when needed. It shall be the basis for the erection of the equipment. All steel parts shall be sand blasted and then painted with the approved primer and first coat before it can be assembled at site. All bought out equipment parts shall be once again checked at site for scratches and peelings of paints and rectified before permitting the erection. Levelling of overflow weir edges shall be done by guiding the finished levels with the help of two levelling instruments placed perpendicular to each other and focusing on the same reference at a given time. The run of mechanical equipment first involves the run of electrical prime movers and shall not be run for unduly long periods. The sludge scrapper in clarifiers shall be completed in their erection along with screeding of the inclined floor.

5.3.12 Unit Bypasses

Bypass arrangements shall be by gravity either by a direct outfall conduit or through a dedicated terminal tank before the outfall conduit. In the case of a tank, its volume shall be equal to the buffer volume needed as calculated by mass diagram (Appendix A.5.2). However, the economics of such a dedicated terminal tank versus the size of the outfall conduit to handle the peak flow shall also be considered before arriving at a decision. It should be possible to chlorinate this volume to the same level as for treated sewage before discharge. If pipes or conduits of RCC are used, they shall be with protection by High Alumina cement or sulphate resistant cement. They can also be of other pipe materials or brick masonry conduit. All such pipes or conduits shall be designed for a velocity of not less than 0.6 m/s and not more than 2.5 m/sec.

The bypass facility is to be provided after each location as the screen chamber, detritor and the primary clarifier. There shall not be any bypass before the secondary clarifiers of activated sludge plants because the MLSS will be washed out. The chlorinator should be designed to handle a dosage of 10 mg/L. The volume of the chlorine contact tank shall be 30 minutes based on the average flow. The discharge location shall be got approved from the PCB. Its physical arrangements shall be carried out under intimation to the pollution control authorities and the authority in charge of the waterways.

5.3.13 Unit Dewatering

Dewatering of treatment units is not a standard requirement in STPs because the sewage flow is for 24 hours 7 days a week. However, if civil work in a tank is needed for maintenance, the dewatering becomes necessary. All units shall be constructed with a puddle flange pipe of not less than 150 mm ID in the side-wall just above the formation level and provided with an isolation valve and closed by a removable blank flange on the airside. A portable diesel based pump set can be connected at this flange to pump out the contents into the plant bypass chambers. This pumping is necessary because these chambers in the by pass pipe-line will have their sidewalls at the same elevation as the sidewalls of treatment units, and hence, gravity dewatering is not possible.

5.3.14 Floatation

Floatation as a method of removing the suspended matters is not usually followed in biological STPs. Their only application is in removing oil & grease from raw sewage if industrial uncontrolled sewage enters into the sewage in the collection system. The oil & grease enters the sewers as free oil and grease. However due to the turbulence of the flow in sewers, pump sets, pumping mains, etc. It results in water and oil forming an “emulsion”. These will not float like oil & grease and they remain distributed in the entire sewage. Thus they cannot be removed directly by floatation. The emulsions in sewage are usually oil in water emulsions. These emulsions must be first broken up by chemical reactions to release the free oil and grease. Then only they can be removed by floatation. This breaking of the emulsions is done by adding chemicals like alum or polyelectrolytes. This is referred to as “de-emulsification”. Thereafter the sewage can be allowed to stand still to allow the de-emulsified oil and grease to float. This is removed by skimming and is called scum. If fine air bubbles are intimately dispersed into the de-emulsified sewage by passing the mixture through pressurised tubes, the bubbles get more intimately bonded to the oil and grease particles. Thereafter the mixture is released immediately into a shallow tank. Here the air bubbles and the de-emulsified oil and grease rise rapidly as scum, and are removed by skimmers. The subnatant of the tank is the oil & grease removed sewage. These are all patented units and overall design criteria cannot be prescribed. Usually this is provided after the screens and grit removal.

5.3.15 Construction Materials

5.3.15.1 Concrete

This section gives only the guidelines and not exhaustive specifications. In general, RCC is the preferred material for civil structures and shall follow IS: 456 and IS: 3370. The cement shall be of IS: 269 with the latest revision thereof for 33 grade ordinary Portland cement, IS: 12330 for sulphate resistance cement, IS: 8112 with the latest revision thereof for 43 grade ordinary Portland cement and IS: 12269 with the latest revision thereof for 53 grade ordinary Portland cement as the case may be. Sand shall follow IS: 2116. Coarse and fine aggregates (stone and sand) shall follow IS: 383 and IS: 456. The maximum quantities of deleterious materials in the aggregates, as determined in accordance with IS: 2386 (Part II) shall not exceed the limits given in Table I of IS: 383 All Reinforcement steel should be either of the following:

(a) Fusion Bonded Epoxy coated having not less than 175 microns thickness and up to 300 microns to reinforcement of all diameters as per IS: 13620 for RTS rods. (b)The binding wire should be of PVC coated and the exposed portion if any after bar bending work should be covered with the Epoxy paint supplied by the coating agency. (c) CTD of high strength deformed CRS steel reinforcement bars conforming to relevant BIS codes (Gr Fe 415, BIS code 1786-1985) from producers approved by the Ministry of Steel (MoS) of GOI.

Water proofing on the inside surface of all liquid retaining structures (except sludge digester and gas holding tank, which shall be of sulphate resistant cement and solvent free epoxy painted) shall be made by thoroughly cleaning of all the dust, grit, grease, oily matter, and other deleterious material.

This will be followed by three coats of sodium silicate solution in a proportion of 1 part of sodium silicate to 3 part of water applied for one litre of solution, to cover 4 sqm of surface, and each coating allowed to dry for 24 hours.

5.3.15.2 Structural Steel

Wherever a structural steel member like channels, angles, I sections etc used in mechanical equipment, gets in contact with sewage it requires special precautions. The sludge scrapers in the floor of clarifiers are held in position by mild steel angles suspended from the walkway platform of the clarifier. These are partly inside the sewage and partly above it. The wind action of the sewage causes oscillation of the sewage surface. This results in alternate exposure of steel to wetting and drying. This condition accelerates the corrosion. In such cases, the steel member shall be spliced with SS members for 30 cm above and 30 cm below the sewage surface. Epoxy or special polymer painting of the other portions of steel is needed. Before painting, the bare steel shall be sand blasted to SA 2.5 Swedish standard SIS 05 5900 with a surface profile not exceeding 65 microns or the equivalent specifications of ASTM. Where sand blasting is not possible, manual chipping or wire brushing to remove loose rust and scale shall be permitted to ST 2 Swedish standard SIS 05 5900. Solvent free epoxy coating shall be for 360 microns and curing shall be done for 7 days, at room temperature if the temperature is less than 15ºC, the surface shall be warmed up by in candescent lamps, heaters, blowers or infrared lamp.

5.3.15.3 Steel Pipes

Bar / wire wrapped steel cylinder pipes with cement mortar lining and coating (including specials) conforming to IS: 15155 shall be permitted for large diameter pipelines that are not laid under concrete structures but only under roadways and freeways. For pipelines under concrete structures, preferred material shall be DI, CI or high alumina RCC pipes and with O-ring joints for all these.

5.3.16 Coating and Painting

Already covered under Section 5.3.15.1

5.3.17 Operating the Equipment

All the operations of equipment shall be carried out as per the preventive maintenance manual of the respective equipment manufacturer and an independent quarterly external third party inspection and certification.

5.3.18 Erosion Control during Construction

The site shall be first set out with a ditch drain in kutcha earthwork with stone boulders loosely placed in a trapezoidal section to intercept all overland runoffs from the surrounding areas onto the STP site. The termination of the ditch drain shall be into a public water course by gravity and if this is not possible a dyke and temporary diesel driven pump sets shall be readied before the monsoon and manned 24 hours seven days a week for the full monsoon period.

Where this type of prevention appears too complicated for reasons like land availability etc., a diaphragm wall shall be first constructed all around the STP site with its sill at least 50 cm above the MFL of the area at the entry and exit at the STP site. Within the STP site, temporary ditch drains along the proposed road alignments shall be provided and the storm water pumped out to the nearest water course. The use of well point system is also recommended.

5.3.19 Grading and Landscaping

The grading of the finished site shall be such that the riding surface of the roads shall be at least 20 cm above the finished ground level on both sides. There shall be a suitable chamber to drain the storm water to the drains on both sides. In main arterial roads, the free land between the edge of storm water drains and the nearest structure shall be not less than 3 m to permit the laying and maintenance of water pipelines, sewers, manholes, power cables, street-lights, instrumentation cables and interconnecting pipelines between STP units. In advanced countries, the cables and interconnecting pipelines between units are taken through RCC walk through box culverts connecting the units below GL. This permits their future maintenance by walking through the box culverts and without any digging up the formed ground. Such man entry box culverts shall follow all the safety precautions for confined spaces and especially the indoor air quality by forced ventilation and adequate lighting and emergency communication facilities. Landscaping shall be confined only to turfing. Flowering plants if used shall be housed only in dedicated ornamental pots or concrete troughs. Trees with spread out roots should never be permitted within the STP site and for clear 6 m from any civil structure. This is because, these roots are known to go in search of water and even pierce through the sidewalls and floor of concrete structures. Thereafter, they will corrode the reinforcement steel rods and weaken the concrete side walls. A good grading and landscaping is shown in Figure 5.18, Figure 5.19 and Figure 5.20 at the Vrishabhavathi Valley STP at Bangalore. The fountain in Figure 5.19 is with the treated sewage.

Source: BWSSB

Figure 5.18 Grading and landscaping at Vrishabhavathi Valley STP, Bangalore

Source:BWSSB

Figure 5.19 Treated Sewage Fountain at Vrishabhavathi Valley STP, Bangalore

Source: BWSSB

Figure 5.20 Grading and landscaping at Vrishabhavathi Valley STP, Bangalore

5.4 PLANT OUTFALLS

The outfall takes the treated sewage from the STP to the disposal location. It can be either a gravity conduit or pumped conduit. The guidelines of section 5.3.10 apply here also. It shall be designed for the DWF if equalization basin is provided in the STP. If not, it shall be designed for the peak flow.

It shall be constructed with the invert at minimum of 0.5 m above the MFL of the receiving water body or other regulations by the local authority. The discharge shall be first let into a receiving well and then spill over to the river or water body. The designs of the receiving well shall be got approved by the competent local body before construction.

The receiving well shall be provided with RCC removable cover slabs of heavy duty such that where required, the full plan view of the well can be ventilated to permit man entry safely when required. The top of the cover shall be sloping with a finish of coarse aggregate like a stucco finish to ensure that it is not used for squatting or as a leisure spot.

There shall be two heavy duty manhole covers on the slab at opposite ends and a stub ventilating pipe with a downward “Tee” and 90 degree bends at each end duly covered with synthetic mosquito mesh secured by nylon ropes with their ends heat sealed.

5.4.1 Discharge –Physical Impact Control

The main objective is to avoid erosion of the banks of the water course or river or reservoir due to the hydraulic turbulence of the treated sewage falling and running over these earth surfaces. This will be controlled by the spillway design.

5.4.2 Protection and Maintenance of Outfalls

This shall be done only by the competent local authority in charge of the water course or river or reservoir. The annual cost shall be paid to them as part of O&M cost of the STP.

5.4.3 Sampling Provisions

Sampling provisions are needed for grab sampling in case of manual collection and automatic instrumentation based on-line testing for automated sampling. The sample collection facility shall be provided in the firm and level ground portion before the bunds and on the roadside.

This shall be by means of a scour pipe enclosed in a masonry chamber with removable heavy duty CI manhole framelid. Trying to sample at the exact discharge end is not necessary as it involves risk of the person falling into the water course or river or reservoir.

5.5 ESSENTIAL FACILITIES

5.5.1 Emergency Power Facilities

The emergency power facilities will be as in Section 5.12 in the electrical section.

5.5.2 Water Supply

The water supply at the STP shall be provided on the same lines as a public water supply system as in the CPHEEO manual on water supply and treatment.

5.5.3 Sanitary Facilities

Toilets and baths in the STP site are to be judiciously located so that the operator need not have to walk long distances. As the sewage flows at these locations will be very small and intermittent, septic tanks shall be provided at each location. The tank shall be emptied by a sewer lorry as part of the O&M segment of the city sewerage system. It shall discharge it into the raw sewage receiving structure of the STP.

5.5.4 Flow Measurement

This shall be as per Section 4.15.

5.5.5 Laboratory

This is dealt with in Appendix A.5.3, Appendix A.5.4 and Appendix A.5.5.

5.6 SCREENING, GRIT REMOVAL AND FLOW EQUALIZATION

5.6.1 Screening

Screening is essential for removal of floating materials which are mainly sachets, plastic sheet bits, leaves, fibres, rags, etc. If these are not removed, they will get into the pumps and entangle in the impellers. They can also be drawn into suction pipes and choke them and it is difficult to locate their position in the pipeline. They can cause objectionable shoreline conditions where disposal into sea is practiced. Screens are used ahead of pumping stations, meters and as a first step in all STPs. A screen is a device with openings generally of uniform size. The screening element may consist of parallel bars, rods, gratings or wire mesh or perforated plates and the openings may be of any shape, although generally they are contrived from circular or rectangular bars. It is recommended that three sequential stages of screens shall be provided being coarse, followed by medium and followed by fine screens. A typical design example is shown in Appendix A.5.6.

5.6.1.1 Coarse Screens

They serve more as protective devices in contrast to fine screens, which function as treatment devices. Coarse screens are usually bar screens and are sometimes used in conjunction with comminuting devices. A bar screen is composed of vertical or inclined bars spaced at equal intervals across a channel through which sewage flows. It is usual to provide a bar screen with relatively large openings of 25 mm. Bar screens are usually raked clean manually or by mechanical devices. These rakes sweep the entire screen removing the floating substances. Some mechanical cleaners utilize endless chains or cables to move the rake teeth through the screen openings. Screenings are raked to a platform with perforations which permits the drainage of water content back to the unit.

Hand cleaned racks are set usually at an angle of 45 to 60 degrees to the horizontal to increase the effective cleaning surface and facilitate the raking operations. Experience indicates that the area of the vertical projections of the space between the bars measured across the direction of the flow should be about twice the area of the feeding sewer.

Mechanically cleaned racks are generally erected almost vertically. Such bar screens have openings 25% in excess of the cross section of the sewage channel. Their area is usually half of that required for hand raked screens. Additional provision should be available for manual raking to take care of the situations where the mechanical rakes are temporarily out of order. Plants using mechanically cleaned screens have controls for (a) manual start and stop (b) automatic start and stop by clock control (c) high level switch (d) high level alarm (e) starting switch or overload switch actuated by loss of head and (f) overload alarm. The fabrication of screens should be such that bolts, cross bars, etc., will not interfere with raking operations.

5.6.1.2 Medium Screens

Medium bar screens have clear openings of 12 mm. Bars are usually 10 mm thick on the upstream side and taper slightly to the downstream side. These mechanically raked units are used before all pumps or treatment units such as the stabilization ponds. The bars used for the screens are rectangular in cross-section, usually about 10 mm × 50 mm and are placed with the larger dimension parallel to the flow. A weir on the side of the screen may be used as an overflow bypass.

5.6.1.3 Fine Screens

Fine screens are not normally suitable for raw sewage directly because of the clogging possibilities. They are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of 5 mm or may be of the drum or disc type, mechanically cleaned and continuously operated.

Fine screens have generally a net submerged open area of not less than 0.05 m^2 for every 1000 m^3 of average daily flow of sewage from a separate system, the corresponding figure being 0.075 m^2 for combined systems. They are also used for beach protection where sewage without any further treatment is discharged into sea for disposal by dilution in situations where sewerage systems are not yet in place.

5.6.1.4 Comminuting Devices

A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that shreds the retained material and enabling it to pass along with the sewage. The solids from the comminutor however, may lead to the production of more scum in the digester.

They are recommended for smaller sized STPs of up to 1 MLD.

5.6.1.5 Location of Screens

Screening devices are usually located where they are readily accessible because the nature of materials handled requires frequent inspection and maintenance of the installation. Where screens are placed in deep pits or channels, it is necessary to provide sufficiently wide approaches from the top and ample working space for easy access and maintenance.

Provision should be made for the-location of penstocks and bypass arrangements for the screens.

5.6.1.6 Housing of Screens

The need for a structure to house the screening equipment depends on two factors viz., the design of the equipment and the climatic conditions. If climatic conditions are not severe and could be withstood by the equipment, the screen housing can be omitted. Mechanically cleaned screens generally need suitable housing to protect the equipment, prevent accidents to operating personnel and improve the appearance of the STP. Ventilation of the housing is necessary to prevent accumulation of moisture and removal of corrosive atmosphere. An illustration is shown in Figure 5.21.

Figure 5.21 Mechanical screen with screen housing and ventilation at Koyambedu STP in Chennai. Manual screen and bypass channel screen in between the mechanical and manual screens is also seen

5.6.1.7 Hydraulics

A screen by its very nature and function collects material which will interfere with the flow. If the screen is cleaned continuously by mechanical arrangement, this interference will be kept to a minimum. Screens with intermittent cleaning arrangement are likely to produce surges of relatively high flow soon after cleaning. The usually accepted design is to place the base of the screen several centimetres below the invert of the approach channel and steepen the grade of the influent conduit immediately before the screen.

5.6.1.8 Velocity

The velocity of flow ahead of and through a screen varies materially and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 m/s through the open area for the peak flows have been used satisfactorily. When considerable amounts of storm water are to be handled, approach velocities of about 0.8 m/s are desirable to avoid grit deposition at the bottom of the screen, which might otherwise become inoperative when most needed during storm though lower value of 0.6 m/s is used in current practice.

Further, the velocity at low flows in the approach channel should not be less than 0.3 m/s to avoid deposition of solids. A straight channel before the screen is mandatory. Its length shall be a minimum of 5 times the width of the screen chamber. A similar channel after the channel is ideal for good hydraulics. Velocities can be got in the channel before the screen by adjusting the floor slope of the channel. These will ensure good velocity distribution across the screen and maximum effectiveness of the device.

5.6.1.9 Head Loss

Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of the screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated by the following formula:

$$h = 0.0729(V^2 - v^2) \tag{5.1}$$

where,

h : Head loss in m

V : Velocity through the screen in m/s

v : Velocity before the screen in m/s

Usually accepted practice is to provide loss of head of 0.15 m but the maximum loss with clogged hand cleaned screen should not exceed 0.3 m. For the mechanically cleaned screen, the head loss is specified by the manufacturers.

Another formula often used to determine the head loss through a bar rack is Kirschmer’s equation:

$$h = \beta (W/b)^{4/3} h_v \, Sin\theta \tag{5.2}$$

where,

h : Head loss, in m

β : Bar shape factor which is assigned value of 2.42 for sharp edged rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular, and 1.67 for rectangular bar with both u/s and d/s face as semi-circular

W : Maximum width of bar facing the flow, m

b : Minimum clear spacing between bars, m

hv : Velocity head of flow approaching rack, m

θ : Angle of inclination of rack with the horizontal

The head loss through fine screens is given by the formula

$$h = (1/2g)(Q/CA)^2 \quad (5.3)$$

where,

- h : Head loss, m
- Q : Discharge, m^3/s
- C : Coefficient of discharge (typical value 0.6)
- A : Effective submerged open area, m^2

5.6.1.10 Quantity of Screenings

The quantity of screenings varies with the size of screen used and on the nature of sewage. Generally it has been found that for every the screenings from sanitary sewage vary from 0.0015 to 0.015 m^3 / million litres with screen sizes having clear opening of 100 mm and 25 mm respectively.

5.6.1.11 Other Screens

5.6.1.11.1 Rotary Drum Screens

Where the incoming sewage to the STP is at a higher elevation than ground level, the use of horizontal rotary screen is advantageous in that it avoids the need for manual scraping and the complicated forward and backward mechanical rakes. A typical drawing is shown in Figure 5.22.

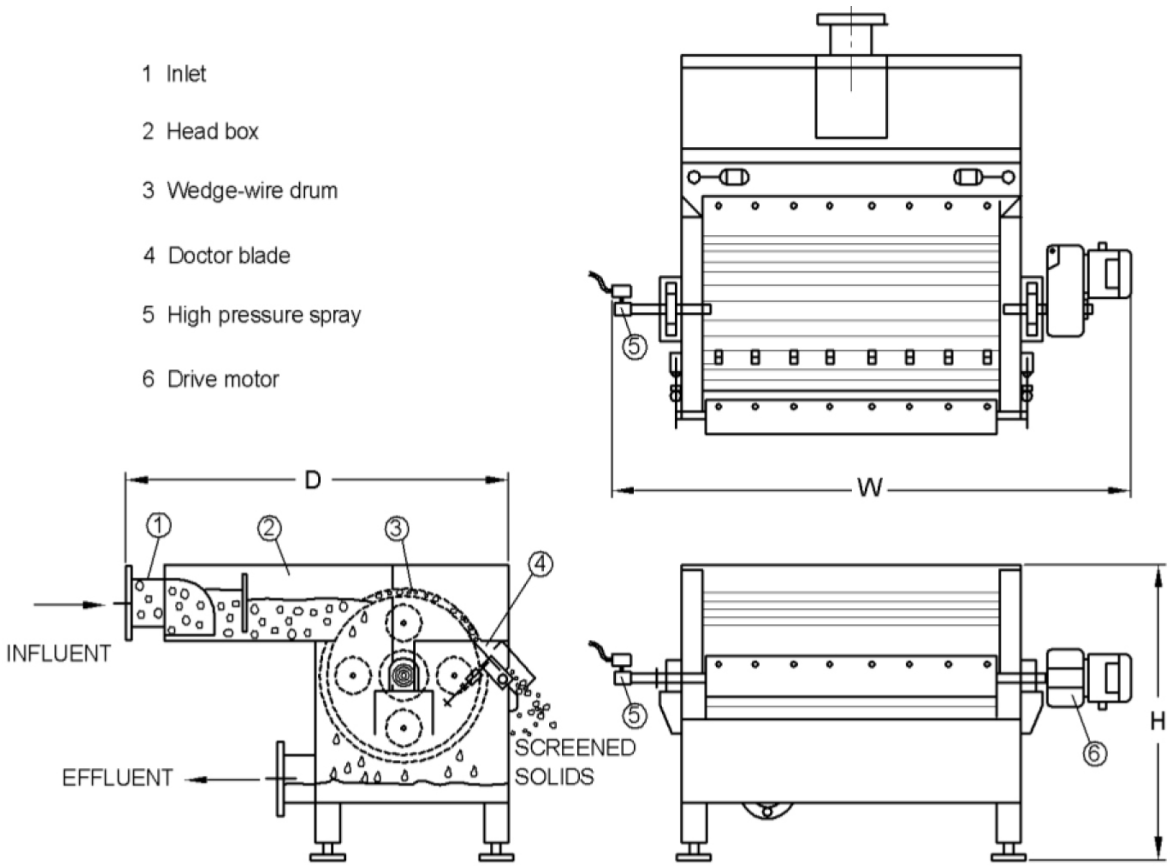


Figure 5.22 Typical rotary drum screen

The inlet sewage skims over the rotating screen and the screenings are intercepted and are rotated forward and to be are scraped onto conveyor belting. The screened sewage will go through the slots and fall downwards and get collected in a bottom trough and in the process it releases any sticking screenings to the circular screen in its downward rotation and the screenings float up for the screening on the upward movement. The screened sewage at the bottom can be collected in a channel and taken out in any suitable direction for downstream units. Though there are advantages of a simple operating system, there is the disadvantage of higher head loss compared to the bar screens due to the drop in sewage elevation for the height of the drum. Design criteria for these are vendor based and hence, these can be alternative offers in tender calls and then decided after evaluating the technical aspects. Their design criteria are generally as per the chosen manufacturer’s design standards.

5.6.1.11.2 Circular Wedge Wire Screens

These are relatively simpler screens which operate on the principle shown in Figure 5.23

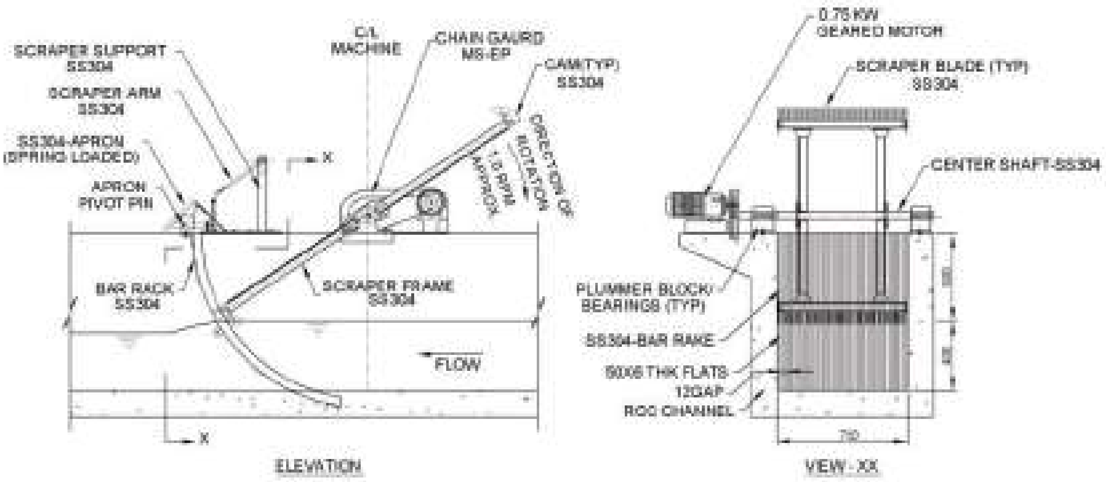


Figure 5.23 Typical circular wedge wire screen

The screen rods are usually wedge shaped which give a narrower slit in the upstream than the downstream slit of the screen openings and this prevents the clumping and jamming of the screenings while flowing across the slits. The screenings are continuously propelled by wire brushes on rotating arms and when they move up above the elevation of the screen rods, the screenings are ejected over a conveyor belt by the release of a spring loaded mechanism in the tubular arms holding the wire brush. The loss of head is just about the same as compared to the bar screens and the mechanism is simple for local servicing. Their design criteria are generally as per the chosen manufacturer’s design standards. An installation is shown in Figure 5.24 overleaf.

5.6.1.12 Screen Compactor

The Screen Compactor is a unit which combines the following. The screen is usually a drilled or wedge wire metal screen section with mesh sizes that can be fabricated to suit and typically can be in the range of 0.25 mm to 10 mm.

Figure 5.24 UNIDO type circular wedge wire screen seen from upstream at left and from downstream at right in Ramanathapuram STP of Tamilnadu

Source TWAD Board, Chennai, Ramanathapuram STP

The conveyor is used to transmit the captured solids out of the sewage and dewater by gravity conveying the separated solids towards the pressing zone where it gets dewatered and compresses the screenings to reduce the volume. Their design criteria are generally as per the chosen manufacturer's design standards. This is shown in Figure 5.25.

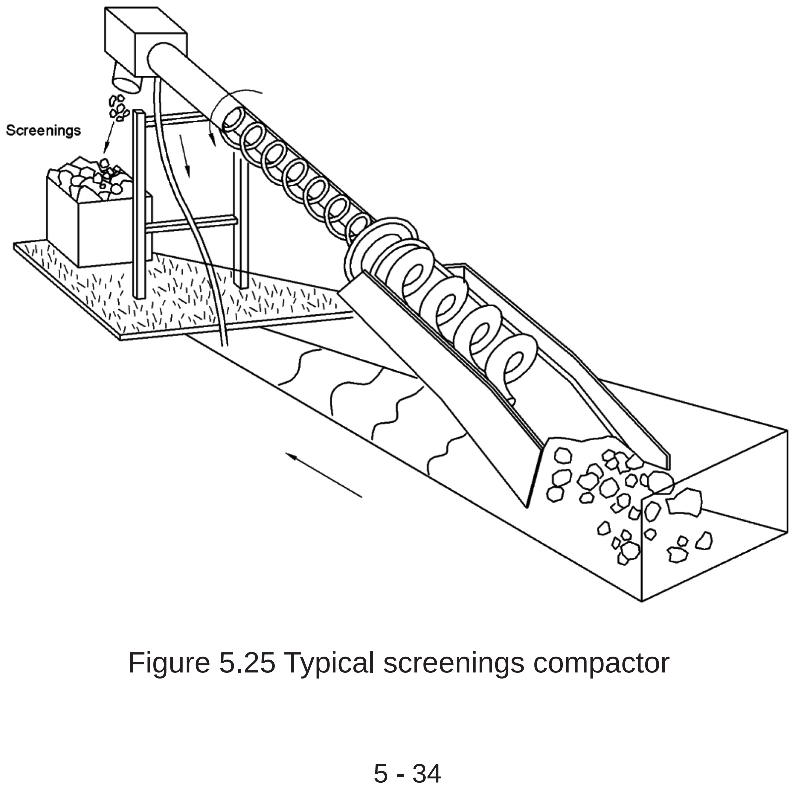


Figure 5.25 Typical screenings compactor

5.6.1.13 Screw Press

These are relatively more modern and are illustrated in Figure 5.26. The progressive compaction in the inclined conduit simultaneously, helps dewatering the screenings and compacting. Their design criteria are generally as per the chosen manufacturer's design standards.

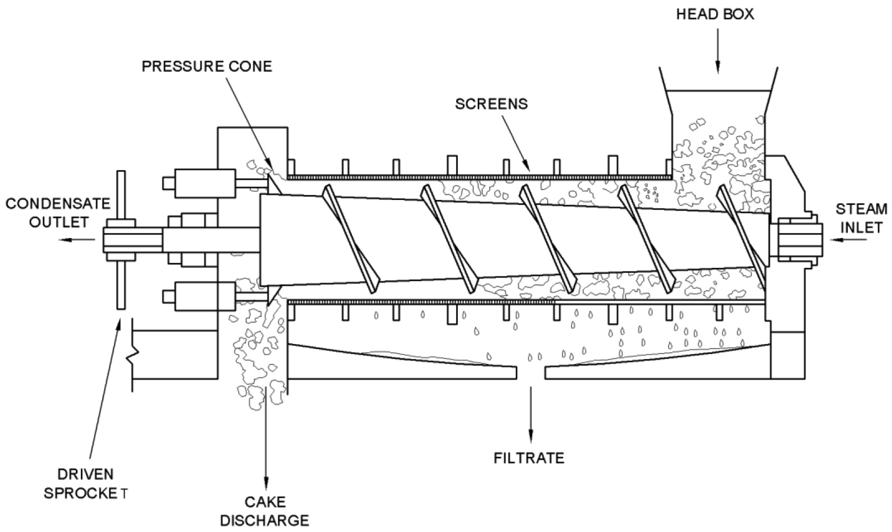


Figure 5.26 Typical screw press

5.6.1.14 Disposal of Screenings

The methods of disposal of screenings could be burial or composting. The screenings should not be left in the open or transported in uncovered conveyors as it would create nuisance due to flies and insects. If conveyors are used, they should be kept as short as possible for sanitary reasons. Burial in trenches usually 7.5 cm to 10 cm deep is practiced particularly in small installations.

At large works, where sufficient land for burial is not available within a reasonable distance from the plant, screenings shall be transported and mixed with town refuse for production of compost or for further processing and disposal as per guidelines / norms of the local PCB.

5.6.2 Grit Removal

Grit removal is necessary to protect the moving mechanical equipment and pump elements from abrasion and accompanying abnormal wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks. It is desirable to provide screens or comminuting device ahead of grit chambers to reduce the effect of rags and other large floating materials on the mechanical equipment, in case of mechanized grit chamber.

However, where sewers are laid at such depths as to make the location of grit chambers ahead of pumping units undesirable or uneconomical, only a bar screen is provided ahead of pumps, with grit chambers and other units following the pumps.

Accordingly, the sewage after removal of grit shall be as far as possible free of grit particles. The grit removal units should always have 100% standby.

5.6.2.1 Composition of Grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both quality and quantity of grit varies depending upon (a) types of street surfaces encountered (b) relative areas served (c) climatic conditions (d) types of inlets and catch basins (e) amount of storm water diverted from combined sewers at overflow points (f) sewer grades (g) construction and condition of sewer system (h) ground and ground water characteristics (i) industrial wastes (j) relative use of dumping chutes or pail depots where night soil and other solid wastes are admitted to sewers and (k) social habits. The specific gravity of the grit is usually 2.4 to 2.65. Grit is non-putrescible and possesses a higher hydraulic subsidence value than organic solids. Hence, it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

5.6.2.2 Types

Grit chambers are of three major types as follows:

i) Velocity controlled V shaped long grit channels

ii) Square shaped chambers with entry and exit on opposite sides and mild hopper

iii) Vortex type cone and the centrifugal action plummets the grit to the bottom

They are mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, head loss requirements, space requirements, topography and economic considerations with respect to both capital and operating costs. In very small plants, mechanization may be uneconomical. For STP flows of more than 10 MLD, mechanized grit removal units are preferred.

5.6.2.3 Vortex Type Units

The sewage is fed tangentially to induce a vortex type of flow, which will funnel the grit towards the centre, and hence be drawn down at the bottom chamber. An auxiliary agitator at this location keeps the grit in suspension and hence it is washed free of organics. The rim flow of the vortex is the degritted sewage to downstream units. The grit at the bottom can be either drained onto a grit filter bed by gravity or pumped to the beds depending on the levels. The filtrate is returned to the raw sewage. Though the centrifugal force and agitation are good controlling mechanisms, the additional head loss incurred in handling the filtrate and pumping if needed are comparatively avoided in the case of velocity controlled channels or detritors. This unit has its advantages in situations where sewage flow rates and durations vary widely. The hydraulic energy required for the vortex may compel the need to impart additional pumped energy to the sewage before degritting. A typical vortex type unit is shown in Figure 5.27 overleaf.

5.6.2.4 Vortex Type Units with Scum Removal

These are similar in their function to the vortex grit separator and have an additional provision to remove the scum. This is useful in situations where this problem is encountered in raw sewage.

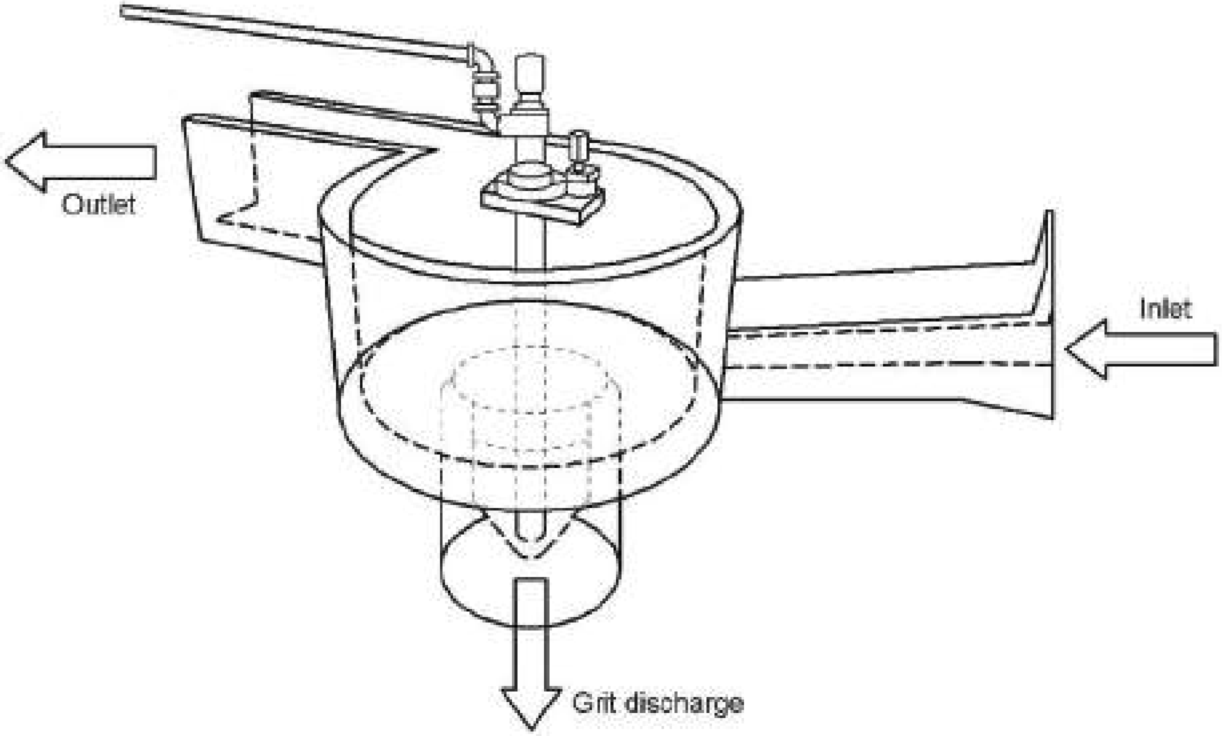


Figure 5.27 Hydraulic regimen of vortex grit separator

As otherwise, these are similar in their function and grit separation methods to vortex grit separator. A typical vortex type unit is shown in Figure 5.28.

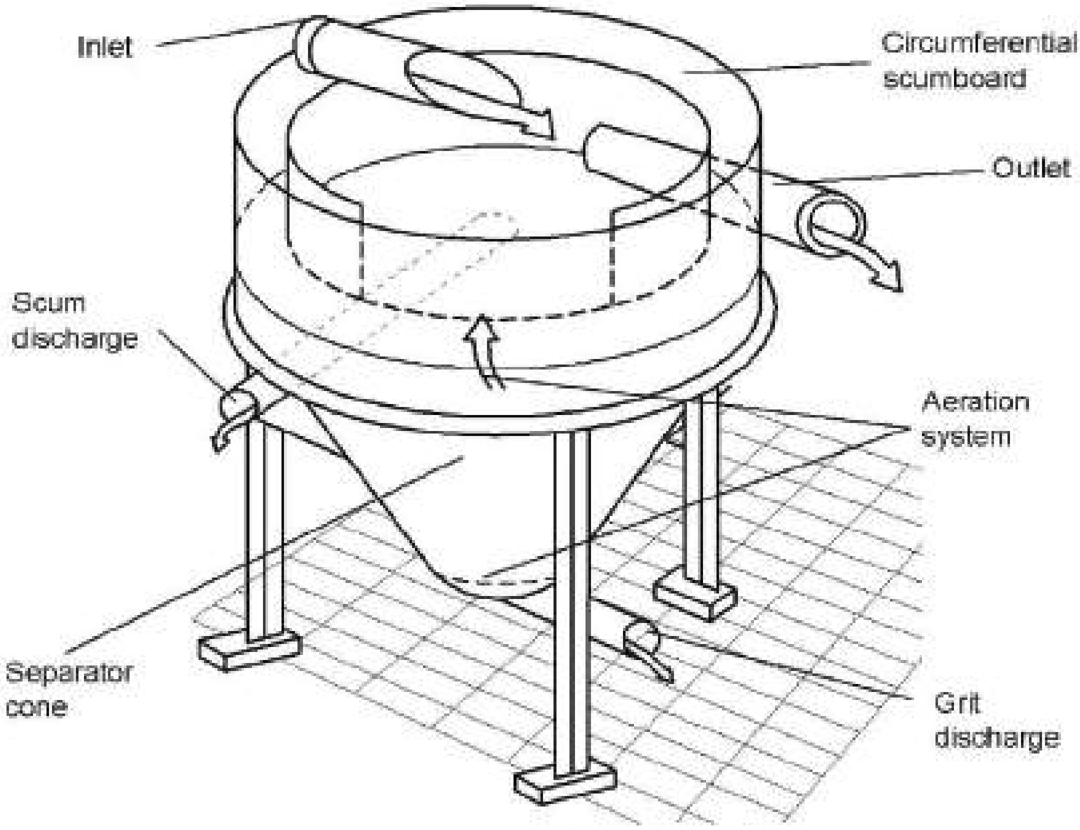
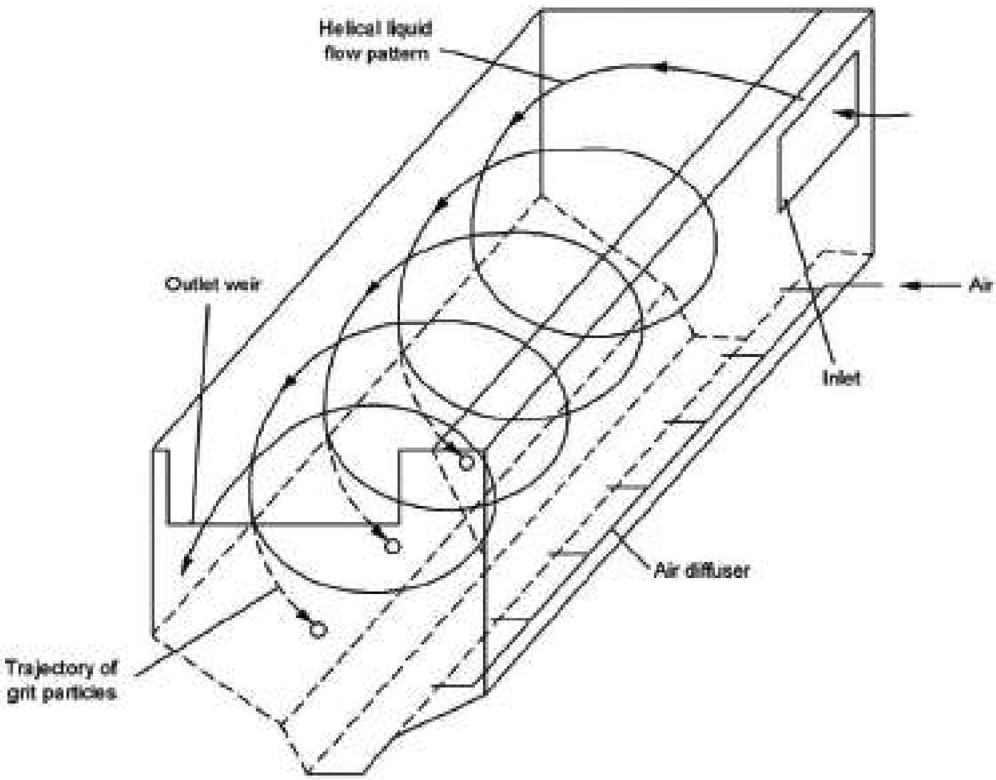


Figure 5.28 Hydraulic regimen of vortex and scum grit separator

5.6.2.5 Aerated Grit Chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral-flow aeration tank provided with air-diffusion tubes placed on one side of the tank, 0.6 to 1 m from the bottom. The grit particles tend to settle down to the bottom of the tank at rates dependent upon the particle size and the bottom velocity of roll of the spiral flow. This is in turn controlled by the rate of air diffusion through the diffuser tubes and the shape of the tank. The heavier grit particles with their higher settling velocities drop down to the floor whereas the lighter organic particles are carried with roll of the spiral motion and eventually out of the tank. The velocity of roll, however, should not exceed the critical velocity of scour of grit particles. Normally, a transverse velocity of flow, not exceeding 0.4 m/s to 0.6 m/s at the top of the tank should satisfy this requirement for differential scour. No separate grit washing mechanism or control device for horizontal velocity is necessary in aerated grit chambers. A typical drawing is shown in Figure 5.29. The aerated de gritting removals are reproduced from WEF MOP 11 hereunder.

Transverse velocity at surface	0.6-0.8 m/s
Depth-to-width ratio	1.5:1 to 2:1
Air supply	4.6-7.7 l/m/s of length
	0.3-0.4 m^3 /m^3
Detention time	3-5 min (peak)
Quantity of grit	7.5-75 ml/m^3
Quantity of scum (skimmings)	7.5-45 ml/m^3



Source: EPA

Figure 5.29 Aerated grit chamber

5.6.2.6 Clearing of the Grit

Manual clearing of the grit is to be avoided except in the case of very small STPs of less than 1 MLD where velocity controlled channels can be cleared by the operator using a shovel and walking on a platform along the length. Mechanical clearing is advocated and these are provided with mechanical equipment for not only collection but also washing of grit and can be operated on either a continuous or an intermittent basis. Scraper blades or ploughs rotated by a motor drive, collect the grit settled on the floor of the grit chamber. The grit so collected is elevated to the ground level by several mechanisms such as bucket elevators, jet pump, screws and air lift. The grit washing mechanisms are also of several designs most of which are agitation devices using either water or air to produce a washing action. In intermittently (normally once or twice a day) operated type, sufficient storage capacity to hold the grit between intervals of grit elevation should be provided.

5.6.2.7 Design Data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flow through velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly, accurate estimation of the flows. The quantity and quality of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.05 to 0.15 m^3/million litres for sewage and 0.06 to 0.12 for combined sewage. The quantity may increase three to four fold during peak flow hours, which may last for 1 to 2 hours.

5.6.2.7.1 Generic Design of Grit Chambers

5.6.2.7.1.1 Settling Velocity

Grit chamber may be designed on a rational basis by considering it as a sedimentation basin. The grit particles are treated as discrete particles settling with their own settling velocities. The settling velocity is governed by the size and specific gravity of the grit particles to be separated and the viscosity of the sewage. The minimum size of the grit to be removed is 0.20 mm although 0.10 to 0.15 mm is preferred for conditions where considerable amount of ash is likely to be carried in the sewage. The specific gravity of the grit may be as low as 2.4, but for design purposes a value of 2.65 is used. The settling velocity of discrete particles can be determined using the appropriate equation depending upon the Reynolds number,

1. Stoke’s Law

$$V_s = \frac{g}{18}\frac{(\rho_s - \rho)}{\rho}\frac{d^2}{\nu} \qquad (5.4)$$

or

$$= \frac{g}{18}(S_s - 1)\frac{d^2}{\nu}$$

where,

V_s : Settling velocity, m/s
g : Acceleration due to gravity, m/s^2
ρ : Mass density of grit particle, kg/m^3
ρ : Mass density of liquid, kg/m^3
d : Size of the particle, m
v : Kinematic viscosity of sewage, m^2/s
S_s : Specific gravity of grit particle, dimensionless

Stoke's law holds good for Reynolds number R below 1.0;

$$R = V_s d / \nu$$

For grit particles of specific gravity of 2.65 and liquid temperature at 10 degree;

$$\nu = 1.01 \times 10^{-6} m^2 / s$$

This corresponds to particles of size less than 0.1 mm. The flow conditions are laminar where viscous forces dominate over inertial forces.

2. Transition Law

The design of grit chamber is based on removal of grit particles with minimum size of 0.2 mm or 0.15 mm and therefore Stoke's Law is not applicable to determine the settling velocity of the grit particles for design purposes.

The settling velocity of a discrete particle is given by the general equation

$$V_s = \sqrt{\frac{4}{3}\frac{g}{C_D}\frac{(\rho_s - \rho)}{\rho} d} \qquad (5.5)$$

Where C_D is the Newton coefficient of Drag which is a function of Reynolds number. The transition flow conditions hold when Reynolds number is between 1 and 1,000. In this range, C_D can be approximated by

$$C_D = \frac{18.5}{R^{0.6}} = \frac{18.5}{\left(\frac{V_s d}{\nu}\right)^{0.6}} \qquad (5.6)$$

Substituting the value of C_D in equation (5.6) and simplifying

$$V_s = \left[0.707(S_s - 1)d^{1.6}\nu^{-0.6}\right]^{0.714} \qquad (5.7)$$

The settling velocity of grit particles in the transition zone is also calculated by the Hazen's modified formula

$$V_s = 60.6(S_s - 1)d\frac{3T + 70}{100} \qquad (5.8)$$

Where d in equation (5.8) is in cm and T is the temperature in degree Centigrade and Vs in cm/s.

The settling velocity of grit particles in the range of 0.1 mm and 1mm can be determined using equation (5.7) and this equation or its approximate empirical form of equation (5.8) should be used in design of grit chambers which are designed to remove particles of size 0.15 mm or 0.2 mm.

3. Newton's Law

When the particle size increases beyond 1 mm and Reynolds number beyond 1,000, the Newton coefficient drag C_D assumes a constant value of 0.4 and the following equation can be used to determine the settling velocity of grit particles.

$$V_s = \left[3.3g(S_s - 1)d \right]^{0.5} \tag{5.9}$$

5.6.2.7.1.2 Surface Overflow Rate

Efficiency of an ideal settling basin is expressed as the ratio of the settling velocity of the particles to be removed (V_s) to the surface overflow rate (V_o).

$$\eta = \frac{V_s}{V_o} \tag{5.10}$$

Where V_o is defined as the ratio of flow of sewage to be treated in an ideal settling tank to the plan area of the tank, i.e., Q/A. It is equal to the settling velocity of those particles which will be 100% removed in an ideal settling tank.

In an ideal settling basin, all particles having settling velocity, Vs ≥ Vo are completely removed. However, particles having settling velocity, Vs < Vo are removed in proportion to the ratio of V_s to V_o.

Table 5.6 gives settling velocity of different size particles of specific gravity 2.65 (inorganic grit particles) and corresponding surface overflow rates for 100% removal of these particles based on Equation (5.8).

Though the different settling patterns occur as described here, for purpose of preparing DPRs, the discreet settling alone shall be considered to simplify the computations and the criteria is shown in Table 5.6.

Table 5.6 Settling velocities and surface overflow rates for ideal grit chamber at 10ºC

Diameter of Particles, mm	Settling velocity m/s,	Surface Overflow rate m^3 / d /m^2
	Ss = 2.65	Ss = 2.65
0.20	0.025	2160
0.15	0.018	1555

Source: CPHEEO, 1993

However, the behaviour of a real grit chamber departs significantly from that of the ideal settling basin due to turbulence and short-circuiting resulting from eddy, wind and density currents. Hence, the surface overflow rates (SOR) should be diminished to account for the basin performance. Following equation could be used to determine the SOR for a real basin for a given efficiency of grit removal and basin performance.

$$\eta = 1 - \left[1 + nV_s / (Q/A)\right]^{-\frac{1}{n}} \quad (5.11)$$

where,

- η : Desired efficiency of removal of grit particle
- V_s : Settling velocity of minimum size of grit particle to be removed
- Q/A : Design surface over flow rate applicable for grit chamber to be designed
- n : An index which is a measure of the basin performance.

The values of n are 1/8, 1/4, 1/2 and 1 for very good, good, poor and very poor performance. It can be seen that the design surface overflow rate will be 66.67%, 58.8%, 50% and 33.3% of the settling velocity of the grit particles to be removed to achieve 75% removal efficiency in grit chamber with very good, good, poor and very poor tank performance respectively. In practice, values of two thirds to one half are used in design depending upon the type of the grit chamber.

5.6.2.7.1.3 Detention Period

Once the area is calculated surface overflow rate and the liquid depth is ascertained from the equipment manufacturer, the resulting volume and hence detention time at average flow shall be checked up as not to exceed 60 seconds.

5.6.2.7.1.4 Bottom Scour and Flow through Velocity

Bottom scour is an important factor affecting grit chamber efficiency and the scouring process itself determines the optimum velocity of flow through the unit. This may be explained by the fact that there is a critical velocity of flow (V_c) beyond which particles of a certain size and density once settled, may be again placed in motion and reintroduced into the stream of flow. The critical velocity for scour may be calculated from modified Shields' formula:

$$V_c = K_c \sqrt{g(S_s - 1)d} \quad (5.12)$$

where K_c = 3 to 4.5. A value of 4.0 is usually adopted for grit particles.

For a grit particle size of 0.2 mm, the formula gives critical velocity values of 17.1 to 25.6 cm/sec. In actual practice; a horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. The horizontal velocity of flow should be maintained constant at other flow rates also to ensure that only organic solids and not the grit are scoured from the bottom. Bottom scour is an important factor particularly affecting the grit chamber efficiency. Design example of velocity controlled grit chamber and detritors are shown in Appendix A.5.7 and A 5.8

5.6.2.7.2 Specific Design of Recent Devices

Their design criteria are generally as per the chosen manufacturer's design standards.

5.6.2.7.3 Velocity Control Devices for Grit Channels

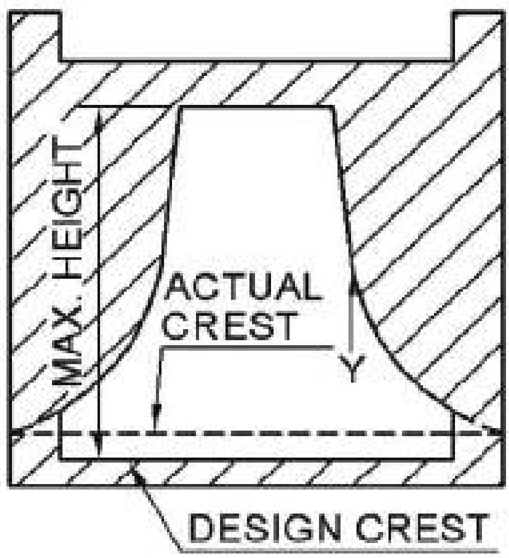
In earlier days, velocity controlled grit chambers were used which needed constant velocities at all depths of flow and for this purpose, devices as proportional weir and Sutro weirs were used downstream. Numerous devices have been designed in an attempt to maintain a constant horizontal velocity of flow through grit chambers in the recommended range of 15 to 30 cm/sec. A variation of 5 to 10% above and below the desired velocity is permitted. Multiple channels with the total capacity to carry the maximum flow are to be adopted.

A satisfactory method of controlling the velocity of flow through the grit channels is by using a control section at the end of the channel, This varies the cross sectional area of flow in the section in direct proportion to the flow.

As for example, for a flow of 5 cum/s the cross-sectional area of flow should be 5 m2 and when flow decreases to 3 cum/s the cross-sectional area of flow should be reduced to 3 m2 to maintain the same velocity in the channel. Such control sections include proportional flow weirs, Sutro weirs and Parshall flumes. In practice, the Parshall flumes are commonly used.

a) Proportional Flow Weir

The proportional flow weir is a combination of a weir and an orifice as shown in Figure 5.30. It maintains a nearly constant velocity in the grit channels by varying the cross-sectional area of flow through the weir so that the depth is proportional to flow.



Source: CPHEEO, 1993

Figure 5.30 Proportional flow weir

The general equation for determining the flow through weir, Q is

$$Q = cb\sqrt{2ag}(H - \frac{a}{3}) \qquad (5.13)$$

where,

- *c* : Coefficient which is assumed 0.61 for symmetrical sharp-edge weirs.
- *a* : Dimension of weir usually assumed between 25 mm and 50 mm
- *b* : Base width of the weir
- *H* : Depth of flow

To determine the shape of the curve forming the outer edges of the cut portion, the following equation of curve forming the edge of the weir may be used.

$$x = \frac{b}{2}\left(1 - \frac{2}{\pi}\tan^{-1}\sqrt{\frac{Y}{a} - 1}\right) \qquad (5.14)$$

where,

- *x* : Weir width at liquid surface
- *Y* : Liquid depth

The weir shall be set from 100 mm to 300 mm above the bottom of grit chamber to provide grit storage or for operation of mechanical grit clearing.

The weir should also be set at such an elevation as to provide a free fall into the outlet channel as it cannot function under submerged conditions. Each grit chamber should be provided with a separate control weir. An installation is shown in Figure 5.31.

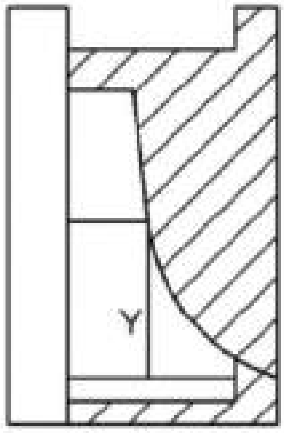
Figure 5.31 Typical Installation of proportional flow weir.

Photo at left is view from the grit channel side. Photo at right is the outlet from the weir showing the graduated readings at various depths of flows

Source-Dr Absar Kazmi, Training Course by Department of Civil Engineering, IIT Roorkee

b) Sutro Weir

The Sutro-weir is a half proportional flow weir cut symmetrically and centrally along the vertical axis as illustrated in Figure 5.32. The orifice has a straight horizontal bottom forming the weir.



Source: CPHEEO, 1993

Figure 5.32 Sutro weir

Grit removal from these channels is normally carried out by suction pumps mounted on a bridge travelling the full length in both forward and return directions.

The suction end of these pumps has a hose suspended into the sewage in the channel. They discharge through a header into a channel built along the side of the grit channel. Most often, the suction fails and the grit is not removed. Hence, these mechanized grit removal systems and the velocity controlled long grit channels are proposed to be phased out.

5.6.2.8 Flow Measurement

Flow measurement is invariably to be provided for at the downstream of the grit removal facilities. The Parshall flume is normally used.

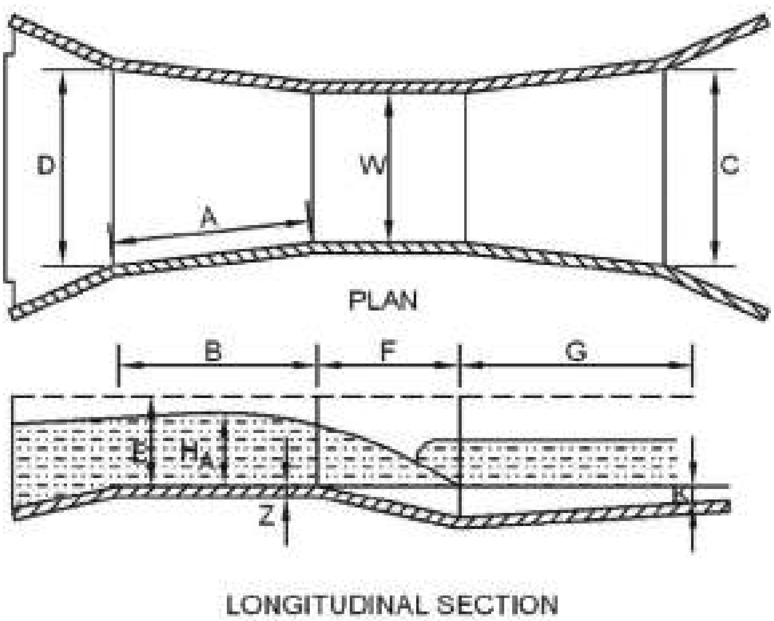
5.6.2.8.1 Parshall Flume

A Parshall flume is an open constricted channel, which can be used both as a measuring device and also as a velocity control device. It is more commonly used for the latter purpose in downstream of grit chambers.

The flume has a distinct advantage over the proportional flow weir, as it involves negligible headloss and can work under submerged conditions up to certain limits as in Figure 5.33 overleaf.

The limits of submergence are 50% in case of 150 mm throat width and 70% for wide throat widths up to 1 m as in Table 5.7 overleaf.

Another advantage is that one control section can be installed after the exit channel of a number of parallel grit chambers. to measure the total flows through all the grit channels.



Source: CPHEEO, 1993

Figure 5.33 Dimensions for Parshall flume

Table 5.7 Dimensions of Parshall flume (mm)

Flow range Q max (mld)+	W	A	B++	C	D+++	F	G	K	Z
Up to 5	75	460	450	175	255	150	300	25	56
5-30	150	610	600	315	391	300	600	75	113
30 - 45	225	865	850	375	566	300	750	75	113
45-170	300	1350	1322	600	831	600	900	75	225
170-250	450	1425	1397	750	1010	600	900	75	225
250 -350	600	1500	1472	900	1188	600	900	75	225
350- 500	900	1650	1619	1200	1547	600	900	75	225
500- 700	1200	1800	1766	1500	1906	600	900	75	225
700-850	1500	2100	2060	2100	2625	600	900	75	225
850-1400	2400	2400	2353	2700	3344	600	900	75	225

+ For average flow and peak factors, see earlier sections

++ Value should be equal to 1.5 × (Qmax) $^{1/3}$ but not less than those shown in the Table

+++ For higher values of B (than shown in the table), the values of D also to be increased to keep D/B ratio same as in table

Source: CPHEEO, 1993

The flume is also self cleansing and there is no problem of clogging. As the Parshall flume is a rectangular control section, the grit chamber above it must be designed to approach a parabolic cross section. However, a rectangular section with a trapezoidal bottom may be used with a Parshall flume in which case the variations in velocity at maximum and minimum flow conditions from the designed velocity of flow should be within permissible limits as given by the following equations.

$$Q = 2264 W (H_A)^{\frac{3}{2}} \tag{5.15}$$

$$D + Z = 1.1 H_A \tag{5.16}$$

and

$$\frac{D_{min}}{D_{max}} = \frac{1.1\left(\frac{Q_{min}}{2264W}\right)^{2/3} - Z}{1.1\left(\frac{Q_{max}}{2264W}\right)^{2/3} - Z} \tag{5.17}$$

$$D = 1.1\left(\frac{Q}{2264W}\right)^{2/3} - Z \tag{5.18}$$

$$b = \frac{Q_{max}}{1000 D_{max} V_{max}} \tag{5.19}$$

or

$$b = \frac{Q_{min}}{1000 D_{min} V_{min}} \tag{5.20}$$

$$V = \frac{Q}{1000 bD} \tag{5.21}$$

where,

Q :	Rate of flow in lps
Qmin :	Minimum rate of flow in lps
Qmax :	Maximum rate of flow in lps
W :	Throat width in m
HA :	Depth of flow in upstream leg of the flume at one third point in m
Z :	Constant in m
D :	Depth of flow in grit chamber in m
b :	Width of grit chamber in m
V :	Velocity of flow in m/s at particular depth of flow

Recommended throat widths for different ranges of flow along with the dimensions of the various dimensions of the flume for the different throat widths are given in Table 5.7 which should be strictly adhered to. A typical example is shown in Appendix A.5.9. A typical Parshall flume installation is shown in Figure 5.34

A Parshall flume in use showing the ultrasonic level measurement equipment mounted over the flume at the throat.

Source-Dr Absar Kazmi, Training Course by Department of Civil Engineering, IIT Roorkee

Figure 5.34 Typical Parshall Flume Installation

5.6.2.8.2 Number of Units

In case of manually cleaned grit chambers at least two units should be provided. All mechanically cleaned units should be provided with a manually cleaned unit as standby.

5.6.2.8.3 Dimensions of Each Unit

The surface areas required for each unit is worked out based on the overflow rate. In case of mechanized grit chambers, the plan dimension and liquid depth shall be readjusted to suit the standard sizes of the mechanical equipment. Additional depth for storage of grit between intervals of cleaning shall be provided depending on the interval of cleaning. A free board of 150 mm to 300 mm shall be provided. Bottom slopes are based on the type of scraper mechanism used.

5.6.2.8.4 Loss of Head

Loss of head in a grit chamber varies from 0.06 m to 0.6 m depending on the device adopted for velocity control in velocity controlled grit chambers. In mechanized units, the free fall over the exit weir shall not be less than 200 mm at peak flow.

5.6.2.9 Disposal of Grit

Clean grit is characterized by the lack of odours. Washed grit may resemble particles of sand and gravel, interspersed with particles of egg-shell, and other similar relatively inert materials from the households. Grit washing mechanism has to be included whenever the detention time is more and flow through velocity is less. Unless washed, it may contain considerable amount of organic matter. This becomes an attraction to rodents and insects and is unsightly and odorous. The grit may be disposed of by dumping or burying or by sanitary landfill. The ultimate method used however depends upon the quantity and characteristics of the grit, availability of land for dumping, filling, or burial. In general, unless grit is washed, provision for burial should be made.

5.6.2.10 Choice of Unit

The square detritor offers perhaps the reasonable preferred combination of (a) mechanical degritting, (b) mechanical grit washing, (c) minimum head loss, (d) grit delivery at a required elevation to stationed trucks beneath it, (e) slow moving central drive and grit classifiers with minimum wear and tear and (f) all these without the need for manual labour to handle the grit. The grit classifier drive system is to be preferred in mild steel appropriately coated as compared to cast iron cam drives because these cast iron parts will need long intervals to be sent back to the foundry for repairs. Here again, the classifiers can be either the screw or the to & fro raker.

The screw type will need a factory made stainless steel trough to match the screw profile. It has the advantage of a flappable semi cylindrical hinged cover and preventing odours and insects around it and protection in rains. The raker type can be accommodated in ordinary masonry channel. Their choice is a matter of preference by the user agency.

5.6.3 Flow Equalization

The design criteria given in the manual for the treatment portion are based on the DWF. There is no need to make changes to these for peak flows. The necessary adjustments are in-built in these design criteria.

However, when the peak factor exceeds 3 by a wide margin, it is advisable to equalize the sewage flow before feeding to the STP units. This can be either by inline equalization tank or side stream flow balancing tank depending on the volume of the raw sewage flow. The illustrations in Section 3.18 using side stream leaping weirs or floor level leaping weirs can be used to allow the average flow to go on to the STP and excess flows to spill into a balancing tank from where it can be supplemented by pumping during lean flow periods. However, the option of providing either the inline equalization tank or side stream flow balancing tank is decided based on the specific situation and cost economics. The volume of such equalization tanks are to be based on the mass diagram. An example for Chennai sewage conditions as measured in summer and monsoon seasons is illustrated as Appendix A.5.2.

5.7 SETTLING

5.7.1 General

The words settling tanks, sedimentation tanks and clarifiers are used in various manuals and text books. All these mean the same. A more popularly used name is clarifiers. These are used to separate the suspended solids, which can settle by gravity when the sewage is held in a tank. If these suspended solids are discharged into water courses, they will result in sludge banks. If these are used for land disposal, it will lead to clogging of soil pores and uncontrolled organic loading.

The primary clarifier is located after screens and grit chambers and reduces the organic load on secondary treatment units. It is used to remove (i) inorganic suspended solids or grit if it is not removed in grit chamber described earlier, (ii) Organic and residual inorganic solids, free oil and grease and other floating material and (iii) chemical flocs produced during chemical coagulation and flocculation.

Secondary clarifier is located after the biological reactor and is used to separate the bio-flocculated solids or bioflocs of biological reactors. In some cases where two stage bio reactors are used, the clarifiers after the first stage of bioreactor is referred to as intermediate clarifiers.

Septic tanks, Imhoff tanks and clarigester are combination units where digestion of organic matter and settling are combined in the same unit and is meant for small installations.

Settling also occurs in waste stabilization ponds and facultative aerated lagoons. However, the settled organic matter is stabilized in the pond itself and no separate unit is provided.

5.7.2 Characteristics of Settleable Solids

The settleable solids to be removed in primary or secondary clarifiers are mainly organic and flocculent in nature. These are either dispersed or flocculated. Their specific gravity varies from 1.01 to 1.02. The bulk of the finely divided organic solids reaching primary clarifiers are low specific gravity solids which are incompletely flocculated but are susceptible to flocculation. Flocculation occurs within primary clarifiers due to eddying motion of the fluid. The aggregation becomes more complete as the sewage is detained for longer periods (hydraulic residence time) in these tanks. However, the rate of flocculation rapidly decreases as the detention period is increased beyond certain values. Hence prolonged detention periods are not productive and in fact may be counter productive by inducing septic conditions and generation of sulphide gas.

5.7.3 Types of Settling

Mainly , four categories of settling occur, depending on the tendency of particles to interact and their concentration. These settling types are (i) Discrete settling (ii) Flocculent settling (iii) Hindered or zone settling (iv) Compression.

5.7.3.1 Discrete Settling

Discrete particles do not change their size, shape or mass during settling. Grit in sewage behaves like discrete particles. The settling velocity of discrete particles is determinable using Stokes or Transition law. Organic solids in raw sewage and bioflocs in biologically treated sewage cannot be considered as discrete particles and hence Stoke’s law is not applicable for these particles.

5.7.3.2 Flocculent Settling

Flocculent particles coalesce during settling, increasing the mass of particles and settle faster. Flocculent settling refers to settling of flocculent particles of low concentration usually less than 1000 mg/l. The degree of flocculation depends on the contact opportunities, which in turn are affected by the surface overflow rate, the depth of the basin, the concentration of the particles, the range of particle sizes and the velocity gradients in the system. No adequate mathematical equation exists to describe flocculent settling and therefore, overflow rates to achieve a given removal efficiency are determined using data obtained from settling column studies. The removal of raw sewage organic suspended solids in primary settling tanks, settling of chemical flocs in settling tanks and of bioflocs in the upper portion of secondary sedimentation tanks are examples of flocculent settling.

5.7.3.3 Hindered or Zone Settling

When concentration of flocculent particles is in intermediate range, they are close enough together so that their velocity fields overlap causing hindered settling. The settling of particles results in significant upward displacement of water. The particles maintain their relative positions with respect to each other and the whole mass of particles settles as a unit or zone. This type of settling is applicable to concentrated suspensions found in secondary settling basins following activated sludge units. In the hindered settling zone, the concentration of particles increases from top to bottom leading to thickening of the settled particles at the bottom. Such secondary clarifiers where zone settling occurs are designed based on solids loading for the given area of the water surface. The required loading rate can be determined by conducting settling column analysis in the laboratory. However, the values of best design are readily given in this manual in section 5.7.

5.7.3.4 Compression

In compression zone, the concentration of particles becomes so high that particles are in physical contact with each other, the lower layers supporting the weight of upper layers. Consequently, any further settling' results due to compression of the whole structure of particles and accompanied by squeezing out of water from the pores between the solid particles. This settling phenomenon occurs at the bottom of deep sludge mass, such as in the bottom of secondary clarifiers following secondary biological treatment and in tanks used for thickening of sludge.

5.7.4 Design Considerations

5.7.4.1 Factors Influencing Design

Several factors such as flow variations, density currents, solids concentration, solids loading, area, detention time and overflow rate influence the design and performance of clarifiers. In the design of some plants, only a few of these factors may have significant effect on performance while in others, all of them may play an important role. Clarifiers are designed for average flow conditions. Hence, during peak flow periods, the detention period gets reduced with increase in the overflow rate and consequent overloading for a short period. If hourly flow variations are wide, a flow equalization tank will be useful before the treatment units so that uniform hydraulic loading is possible.

5.7.4.2 Design Criteria

The design criteria shall consist of surface loading rate, solids loading rate, weir overflow rate and side water depth.

5.7.4.2.1 Surface Loading Rate

This represents the hydraulic loading per unit surface area of tank in unit time expressed as $m^3/d/m^2$ and must be checked, both, for average flow and peak flow.

5.7.4.2.2 Solids Loading Rate

The solids loading rate is an important decision variable for the design of secondary clarifier which settles the bio-flocculated solids. It is expressed as kg $SS/d/m^2$.

5.7.4.2.3 Weir Loading

Weir loading influences the removal of solids particularly in secondary clarifiers. There is no positive evidence that weir loading has any significant effect on removal of solids in primary clarifiers. However, certain loading rates based on practice are recommended both for primary as well as secondary clarifiers. The loading should however ensure uniform withdrawal over the entire periphery of the tank to avoid short-circuiting or dead pockets. Performance of existing clarifiers for SS removal can be improved by merely increasing their weir length.

Primary and secondary clarifiers normally have the V notch at the weir overflow rim. The CMWSSB is operating a 23 MLD STP using conventional ASP with 2 primary clarifiers of each 21.2 m diameter and 2 secondary clarifiers of each 24.4 m diameter. Their RCC sidewall is topped with 14 cm thick brick pillars of 23 cm length interspaced with masonry bevelled weirs of 70 cm length as in Figure 5.35.

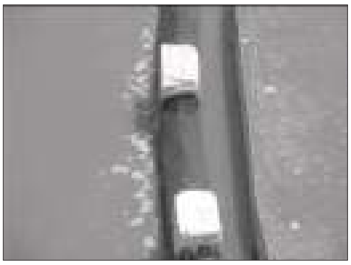
  

Figure 5.35 Masonry Bevelled Weirs of Primary Clarifiers (left & centre) and Secondary Clarifier (right) with additional entrainment aeration of the treated sewage by the freefall over the weir.

In construction, it is easy to guide the mason by a levelling instrument to finish these weirs all at the same elevation as he has to trowel the small length between the pillars one at a time. This arrangement has not resulted in any corrosion and facilitates easy cleaning of the lip and overflow face of the weir daily. In terms of weir length, this effectively means the weir length is 70/93 = 75% of the peripheral length. The weir loading rate at average flow works out to $23{,}000/(2\times3.14\times21.2\times0.75) = 230$ cum/m/day for primary clarifier and $23{,}000/(2\times3.14\times24.4\times0.75) = 200$ cum/m/day for secondary clarifier., though the manual guidelines are limited to 125 and 185 respectively. The suspended solids in overflow in these secondary clarifiers are consistently between 20 and 30 mg/l. Though these higher weir overflow rates are reportedly functioning well, still, complying with the reduced loading rates of 125 and 185 can only prevent the drag of SS over the weirs and better SS removals. However, if v notched weirs are preferred they can be used with appropriate material of weir plates and fasteners.

5.7.4.2.4 Depth and Detention Time

Once the surface area is arrived at from overflow rate and solids loading rate, the next step is the determination of the depth which influences the detention time and vice versa. The depth considered for design is the vertical side water depth (SWD). It influences the hydrostatic compression of the bottom sludge solids. Thus deeper depths will give higher concentration in the sludge solids withdrawn from the bottom sludge pit. Shallow depths will result in loose solids concentration.

This will require huge volumes of wet sludge to be withdrawn for taking out a given weight of sludge solids. In turn, these volumes have a heavy bearing in the required volumes of the sludge handling units and their associated piping and valves etc. Hence drawal of dense sludge is more beneficial.

In the case of secondary clarifiers, another issue is that longer residence times may result in all the residual dissolved oxygen in the treated sewage being fully consumed by the live MLSS in the clarifier itself. Thereafter, these MLSS will not have oxygen until the time they are returned to the aeration tank. It inhibits their metabolism on entering the aeration tank. Thus the return sludge shall not be really live. The depth also influences the hydraulic pattern. Higher depths may cause dead zones and shallow depths may cause short-circuiting between sewage released in the baffle zone and what overflows along the peripheral weirs. The data for clarifiers in STPs built in India and evaluated by NEERI are extracted in Appendix A.5.10.

It is seen that depths of primary clarifiers vary from 2.4 m to 4.2 m with detention times varying from 1.65 hours to 4 hours. In secondary clarifiers, the depths vary from 2.4 m to 4.2 m and detention times vary from 2.2 hours to 4.2 hours. Considering all these factors and the reported performance of these STPs, it requires an iterative approach. In the case of secondary clarifiers for extended aeration plants, deeper depths and longer detention times are not significant from return sludge point of view as the sludge is already mineralized when it leaves the aeration tank. In cases where marginally deeper depths and slightly longer detention times are to be considered for secondary clarifiers, the contact stabilization process is recommended to freshen up the sludge before returning it to the aeration tank.

5.7.4.2.5 Design Guidelines and Procedure

The design guidelines for both the primary and secondary clarifiers are given in Table 5.8.

Table 5.8 Design Parameters for Clarifiers

Type of Settling	Overflow rate, cum/sqm/day		Solid loading, kg/day/sqm		Side Water Depth, m	Weir loading, cum/m/day
	Average	Peak	Average	Peak	Average	Average
Primary Clarifiers						
Primary Settling only	25 - 30	50 - 60	Not applicable		≥ 2.5 - 3.5	125
Followed by secondary treatment	35 - 50	80 - 120	Not applicable		≥ 2.5 - 3.5	125
With excess sludge return	25 - 35	50 - 60	Not applicable		≥ 3.5 - 4.5	125
Secondary Clarifiers						
Secondary settling for activated sludge	15 - 35	40 - 50	70 - 140	210	≥ 3.0 - 3.5	185
Secondary settling for extended aeration	8 - 15	25 - 35	25 - 120	170	≥ 3.0 - 4.0	185

Note: Where the mechanized aerobic treatment is used after UASB reactor, the settling tank design shall be based on conventional activated sludge process as above.

Source: CPHEEO, 1993 and as recommended in the present manual

The smaller values are for plants of less than 5 MLD. It is necessary to provide the clarifiers in at least two parallel units to have availability of one during repairs to the other. For bigger plants, more numbers are needed.

The procedure of sizing the clarifiers shall be as follows. In respect of all STPs where only BOD removal with or without biological nitrification is concerned, the hydraulic load from any return flows as sludge return, thickener supernatant return, sludge filtrate return are not taken into consideration and are deemed to be covered within the design criteria as in Table 5.8

In the case of biological nitrification-denitrification tanks, the flow due to internal return from the secondary clarifier overflow needs to be added to the average and peak flows for verifying the compliance to the design criteria. The recommended design procedures are furnished herein.

Design Procedure for Primary clarifiers:

1. Choose the average overflow rate in Table 5-8 and arrive at the surface area
2. Choose the peak overflow rate in Table 5-8 and arrive at the surface area
3. Choose the higher of the above two values and decide the diameter
4. Verify the weir overflow rate for compliance to Table 5-8
5. If the rate exceeds, verify with a double sided launder inside the clarifier
6. Even with this, if the rate exceeds, increase the diameter suitably.
7. Choose a compatible SWD.

Design Procedure for Secondary clarifiers:

1. Choose the average overflow rate in Table 5-8 and arrive at the surface area
2. Choose the peak overflow rate in Table 5-8 and arrive at the surface area
3. Choose the average solids loading rate in Table 5-8 and decide surface area
4. Choose the peak solids loading in Table 5-8 and arrive at the surface area
5. Choose the higher of the above four values and decide the diameter
6. Verify the weir overflow rate for compliance to Table 5-8
7. If the rate exceeds, verify with a double sided launder inside the clarifier
8. Even with this, if the rate exceeds, increase the diameter suitably.
9. Choose a compatible SWD.

An illustrative sizing of clarifiers in ASP is given in Appendix A.5.11

5.7.4.2.6 Sludge withdrawal from Clarifiers and Thickeners

There is some uncertainty in the issue of whether sludge from clarifiers and thickeners is to be drawn by first drawing it into a sludge sump and then withdrawing by a pump set.

The following clarity is now advocated.

5.7.4.2.6.1 Primary Clarifier for 10 MLD with 400 mg/l of SS & 60 % Removal

The SS removed per day will be 10×400×0.6 = 2400 kg/day

The solids concentration will be 2 %

The volume to be drawn will be 2400×(100/2)/1000 = 120 m^3/day

If drawn continuously for 24 hours, withdrawal has to be 120/24 = 5 m^3/hr

Using minimum specified pipe of dia 200 mm, area is (0.2)×(0.2)×0.785 = 0.03 m^2

Thus the velocity will be 5 / 3600 / 0.03 = 0.04 m/s

Clearly this cannot be permitted.

Suppose the sludge is drawn for 5 minutes every hour, then the flow will be revised as

Withdrawal has to be 120 / (24×5/60) = 60 m^3/hr

Using minimum specified pipe of dia 200 mm, area is (0.2)×(0.2)×0.785 = 0.03 m^2

Thus the velocity will be 60 / 3600 / 0.03 = 0.55 m/s

Obviously this is much better than withdrawal through a sludge sump.

5.7.4.2.6.2 Secondary Clarifier for 10 MLD of Sewage with 0.6 RAS

The volume to be drawn will be 10000×0.6 = 6000 m^3/day

If drawn continuously for 24 hours, withdrawal has to be 6000/24 = 250 m^3/hr

For a velocity of 1.5 m/s, area is 250/3600/1.5 = 0.046 m^2 resulting in a dia of 0.25 m

By using a sludge sump, the height has to be from invert of pipe to top of the clarifier

This height will be anywhere about 4.5 m.

For a minimum diameter of 2 m for man entry when needed, area is 3.14 m^2

Resulting volume will be 4.5×3.14 = 14.13 m^3.

HRT in the sump becomes 14.13/250 = 0.056 hrs or 3.4 minutes

Any sump has to be minimum 10 minutes of HRT

Hence the volume needed is 250×10/60 = 42 m^3

Required diameter becomes 3.5 m

It is a choice by the designer to decide on direct pumping or sump and pumping.

5.7.4.2.6.3 Choice of Sludge Withdrawal

a) Direct suction minimizes the complexities of sumps, valves, etc.

b) If the clarifier water level is just at ground level, it is necessary to construct a dry well and equip it with dry pit submersible pump sets of open impeller or centrifugal screw impeller directly coupled to the flange of the suction pipe after a valve on the upstream. The rpm shall be less than 960.

c) For RAS, if the designer prefers an intermediate sludge sump, necessary diffused air shall be let into such a sump. The pump sets can also be horizontal centrifugal foot mounted pump sets (in a separate dry pit) of motor not more than 960 rpm and equipped with VFD control or with submersible pump sets of speed not over 960 rpm or Archimedean screw pump in the sump itself which has to be rectangular to accommodate the screw.

Sludge can be removed either hydrostatically or mechanically from the sedimentation tanks. The sludge is withdrawn from the tank by hydrostatic pressure or by pumping. Manual cleaning has been largely given up in favour of mechanical cleaning in modern practice. Tanks are also provided with hopper bottoms for hydrostatic sludge removal.

Generally horizontal flow tanks are provided with rectangular hoppers and vertical tanks with circular or square types. Side slopes of the hoppers should be of the order of 1.2:1 to 2:1 preferably with values greater than 1.7:1 and 1.5:1 for pyramidal and conical hoppers respectively. The floor of the hoppers should not be wider than 0.6 m.

Mechanical sludge scraping is best suited for circular or square tanks and occasionally adopted in rectangular tanks. The scrapers or ploughs push the sludge along the tank bottom to sludge collecting channel or pocket from where it is either pumped directly or gravitated to a sludge sump for further disposal. In rectangular tanks, sludge hoppers are generally placed at the inlet end. However, they may be placed at mid-length in long tanks or at the outlet end in case of secondary settling tank of activated sludge plant. The sludge scraping mechanism may be of a moving bridge type of flight scrapers mounted on endless chain conveyors. The linear conveyor speed should not exceed 0.010 to 0.015 m/s.

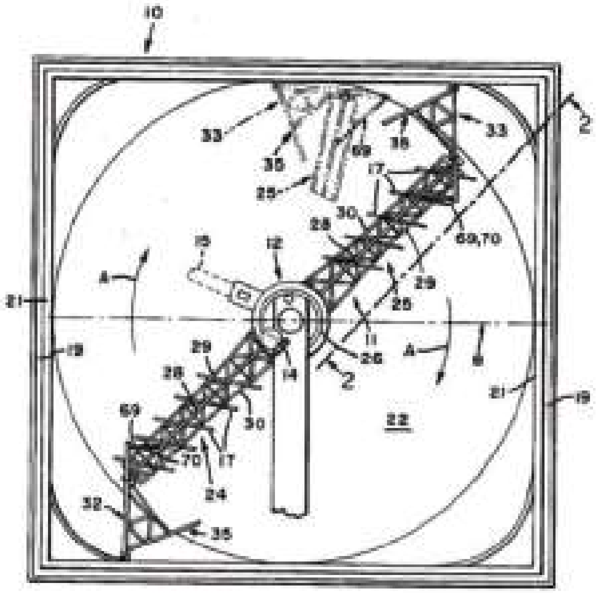
In case of flight scrapers, where the maximum width of tanks is greater than twice the depth, multiple flight scrapers are placed side by side, in which case the width of tank could be increased up to a maximum 30 m. When multiple flight scrapers are used, the receiving sludge hoppers are designed as a trough with transverse collectors to convey the sludge to a single outlet pocket. A bottom slope of 1% is recommended for mechanical scraping of sludge.

The most common type of sludge scraping in circular clarifiers is a revolving sludge scraper mechanism with radial arms having ploughs or blades set at an angle just above floor level and rotating at 1 to 2 revolutions per hour. The ploughs push the sludge to a central hopper as the arms are rotated. Sludge from the central hopper is removed to a sludge sump by the side of the tank from where it is pumped or it is directly sucked and pumped out.

For small diameters of up to 9 m, the revolving bridge is spanned across the diameter of the tank. For bigger diameters, it is supported on the tank wall on one side and on a pillar at the centre of the tank. This pillar is a hollow construction and serves as the inlet to the clarifier. The drive motors for the sludge scraper arms can be either stationary at the centre of the walkway or movable in the case of traction drive and are mounted at one end of the rotating bridge. The interval between sludge removals should be less than 4 hours and never exceed 12 hours. Light flocculent sludge such as the activated sludge or mixture of activated sludge and primary sludge are scraped and shall be removed continuously from the tank to avoid septicity as the activated sludge are live organisms.

The peripheral speed of the scraper should be between 2.5 to 4 cm/sec. All rotary mechanisms are operated at a low speed of 1 to 2 revolutions per hour.

In the case of square tanks, the sludge scraper arms are provided with a pivoted extension with blades which will project out at the corners and retract to suit the diameter once it has crossed them as shown in Figure 5.36.



Source.US Patent Application Number 183106
Figure 5.36 Floor Plan of Square Shaped Clarifiers Sludge Scraper

The pantograph mechanism is numbered as 32 and 35 in Figure 5-36. These remove the sludge from the corners and push it towards the centre. The floor should be suitably finished. These types of clarifier construction is useful in the case of long rectangular aeration tanks where the longer sidewalls can be used to attach primary and secondary clarifiers on each of the shorter sides or on the longer side as abutting clarifiers using common wall construction for saving land area and for superstructure for covering all these for indoor air quality control in due course.

Where sludge is removed intermittently with intervals longer than four hours, provision for sludge storage in the hoppers of the tanks should be made. Sludge conveyor pipes should not be less than 200 mm in diameter. Hopper volumes should be excluded when calculating the effective sedimentation volume of the tank.

As the withdrawal from primary clarifiers is on an intermittent basis, the connecting pipe between the clarifier and the sludge sump outside the clarifier gets into choking problems besides the sludge in the sludge sump also getting anaerobic and malodorous due to storage. There are also installations where the sludge is withdrawn by direct suction.

It is recommended to adopt a sludge withdrawal every hour and of just the adequate volume to not induce the tunnel effect in the sludge zone of the clarifier. The pump impellers are to be preferred as positive displacement stator-rotor or screw centrifugal horizontal foot mounted type working at not over 960 rpm.

5.7.4.2.7 Inlets and Outlets

Performance of clarifiers is very much influenced by inlet and outlet devices. The inlet devices must distribute the flow evenly in the tank.

All inlets must be designed to keep down the entrance velocity to prevent formation of eddy or inertial currents in the tank to avoid short-circuiting. A stilling chamber is necessary ahead of inlets if the sewage is received under pressure from pumping mains.

The design should ensure least interference with the settling zone and promote ideal settling conditions. The outlet devices like the peripheral weirs must withdraw the sewage uniformly over the full length. Hence, the weir elevation must be kept the same all around the periphery.

The choice of inlet and outlet design depends on the geometry of sedimentation tank and the mode of entry and exit from the tank.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite each other separated by the length of tank with the inlet perpendicular to the direction of flow.

In the design of inlets to rectangular tanks the following methods are used to distribute the flow uniformly across the tank.

a) Multiple pipe inlets with baffle boards of depth 0.45 to 0.6 m in front of the inlets, 0.6 to 0.9 m away from it, and with the top of baffle being 25 mm below water surface for the scum to pass over

b) Channel inlet with perforated baffle side wall between the tank and the channels, or

c) Inlet channel with submerged weirs followed by a baffle board inside the tank.

Scum baffles are provided ahead of outlet devices to prevent the escape of scum with the effluent.

In radial flow circular tanks, the usual practice is to provide a central inlet and a peripheral outlet. The central inlet pipe may be either a submerged horizontal pipe from wall to centre or an inverted siphon laid beneath the tank floor or a top entry pipe suspended from the bridge.

An inlet baffle is placed concentric to the pipe mouth, generally with a diameter of 10 to 20 % of the tank diameter and extending 1 to 2 m below water surface. Where the inlet pipe discharges into a central hollow pillar, the top of the pillar is flared to provide adequate number of inlet diffusion ports through which sewage enters the tank with an entry velocity of 0.10 to 0.25 m/s through the ports. The entry ports are submerged 0.3 to 0.6 m below the water surface.

The outlet is generally a peripheral weir discharging freely into a peripheral channel, the crest of the weir is provided with V-notches for uniform draw off at low flows or finished as bevelled masonry wears as in Figure 5.35. In all primary clarifiers, a peripheral scum baffle extending 0.20 to 0.30 m below the water surface is provided ahead of the effluent weir.

If the length of the peripheral weir is not adequate, a weir trough mounted on wall brackets near the periphery with adjustable overflow weir on both sides is provided to increase the length of weir.

5.7.4.2.8 Scum Removal

One distinct feature of primary clarifiers is the skimming device, which could be operated by the same drive mechanism as the sludge scraper arms at the bottom of the tank. It generally consists of a skimmer arm to which a scraper blade is attached and moved, partly submerged and partly projecting above the water surface, from the outlet end towards the inlet end in case of rectangular tanks or in a circular path in the case of circular tanks. The floating scum is thus collected at the forward end of the scraper blade and moved until it is tipped manually or automatically into a scum trough, which discharges the scum to a sump outside the tank from where it is removed for burial, burning or feeding to the digester.

A scum baffle at least 0.15 m above and extending to at least 0.30 m below the water level is provided along the periphery, ahead of outlet device, to prevent the escape of scum with effluent.

5.7.4.2.9 Types and Shapes

Circular tanks are more common than rectangular or square tanks. Up-flow tanks have been used for sewage sedimentation, but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and can be more economically designed in a large plant, where multiple units are to be constructed. They can form a more compact layout with the rectangular secondary treatment units such as aeration tanks in the activated sludge system. The diameters of circular tanks vary widely from 3 m to 60 m although the most common range is 12 m to 30 m. The diameters and depths could be chosen at the discretion of the designer in conformity with the manufactured sizes of scraper mechanisms in the country. The water depth shall be as per Table 5.8. Floors are sloped from periphery to centre at a rate of 7.5 % to 10%. The inlet to the tank is generally at the centre and outlet is a peripheral weir, the flow being radial and horizontal from centre to the periphery of the tank. Multiple units are arranged in pairs with feed from a central control chamber.

For rectangular tanks, maximum length and widths of 90 m and 30 m respectively with length to width ratios of 1.5 to 7.5 and length to depth ratios of 5 to 25 are recommended and depths shall be compatible with the sludge moving equipment manufacturer’s requirements. Bottom slopes of 1% are normally adopted. Peak velocities shall not exceed 1.5 mph.

5.7.5 Performance

Primary clarifiers may be expected to accomplish 30% to 45% removal of BOD, (but shall be taken as maximum of 35% for design) and 60%-70% removal of SS, (but shall be taken as maximum of 60 % for design) depending on concentration and characteristics of solids in suspension. Secondary clarifiers, if considered independently, remove a very high percentage of flocculated solids, even more than 99%, particularly following an activated sludge unit where high mixed liquor suspended solids concentration is maintained in the aeration tank.

However, the efficiency of the biological treatment process is always defined in terms of the combined efficiency of the biological treatment units and its secondary clarifier with reference to the characteristics of the incoming sewage.

5.7.6 Chemical Aided Sedimentation

Chemical aided sedimentation of sewage is not normally recommended in the scope of biological treatment plants, unless it is warranted with reference to needs of compliance with quality of treated sewage, especially to control the residual phosphorous. Sometimes, when biological nitrification is aimed at and the required bicarbonate alkalinity is not inherent in the sewage, Sodium carbonate or bicarbonate will be necessary. In practice, it is analogous to chemical coagulation, flocculation and sedimentation in water treatment. The colloidal and finely dispersed solids which cannot be removed by plain primary sedimentation alone as they possess extremely low settling velocities and are aggregated into settleable particles by addition of chemicals in chemical-aided sedimentation. Commonly used chemicals are salts of lime, aluminium, ferric and ferrous in the form of powder or solutions, polyelectrolytes and polymers.

5.7.6.1 Unit Operations

The process consists of the three unit operations viz., proportioning and mixing of chemicals, flocculation and sedimentation.

5.7.6.1.1 Mixing

The required dose of chemical is weighed and fed to sewage by means of proportioning and feeding devices, ahead of the mixing unit. Mixing is accomplished in a rapid or flash mixing unit provided with paddles, propellers or by diffused air and having detention period of 0.5 to 3 minutes. The paddles of propellers are mounted on a vertical shaft and driven by a constant speed motor through reduction gears. The size and speed of the propeller is so selected as to give a propeller capacity of twice the maximum flow through the tank.

The shaft speed is generally between 100 to120 rpm and power needed is about 0.1 kW / MLD.

5.7.6.1.2 Flocculation

The principle of flocculation in sewage is similar to flocculation in water purification. The flocs that are formed after flash mixing with chemicals are made to coalesce into bigger sizes by either air flocculation or mechanical flocculation. Both diffused air and mechanical vertical draft tube are used for air flocculation. Revolving paddle type is the most common of the mechanical flocculators. The tanks are usually in duplicate with a detention period of 30-90 minutes depending upon results required and the type of sewage treated. However, the combination of chemical dosage and the flocculation period are first determined by laboratory jar test followed by bench scale testing in the field. The paddles are mounted either on a horizontal or vertical shaft. The peripheral speed of the paddles is kept in the range of 0.3 to 0.45 m/s. The flow-through velocity through the flocculator shall be in the range of 15 to 25 cm/sec to prevent sedimentation there itself. The drive motors can be either stationary or movable in the case of traction drive and are placed above the tank. In case of domestic sewage and certain industrial wastes, mechanical flocculation without addition of chemicals will reduce self-flocculation of the finely divided suspended solids and hence increase the efficiency of sedimentation.

5.7.6.1.3 Sedimentation

The flocculated sewage solids are settled out in a subsequent sedimentation tank. Refer Appendix 7.2 to 7.6 of CPHEEO Manual on Water Supply & Treatment 1999.

5.8 SEWAGE TREATMENT

Sewage Treatment detailed here will be on biological treatment technology only. It covers such of those technologies for which validated design guidelines are available in India over the past many decades. There are more recent technologies with each of them having their own design guidelines by the respective equipment vendors and for which obviously there are proprietary issues in procurement out of public funds. No doubt, unless these are tried out at some point in time, there is no way of inheriting these forever, but at the same time, the proprietary issue has to be got over. Hence, these technologies will be addressed later in this chapter under the title “Recent Technologies”. Accordingly, the technologies to be considered in this chapter will be the Activated Sludge Processes, Attached Growth Systems, Treatment Methods Using Immobilization Carrier, Stabilization Ponds and Anaerobic Treatment. It is decided to phase out the stone media trickling filter technology considering the difficulties of upkeep of its rotary distributor, Psychoda flies nuisance and the recent light weight media which give much more surface area for unit volume of the media as compared to the stone media.

5.8.1 Activated Sludge Process

5.8.1.1 Introduction

Aerobic suspended growth systems are of two basic types, those which employ sludge recirculation, viz., conventional activated sludge process and its modifications and those which do not have sludge recycle, viz., aerated lagoons. In both cases sewage containing organic matter is aerated in an aeration basin in which micro-organisms metabolize the soluble and suspended organic matter. Part of the organic matter is synthesized into new cells and part is oxidized to carbon dioxide and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculent sludge in clarifiers. A part of this activated sludge is recycled to the aeration basin and the remaining form waste or excess sludge. In aerated lagoons the microbial mass leaves with the effluent stream or may settle down in areas of the aeration basin where mixing is not sufficient.

The suspended solids concentration in the aeration tank liquor, also called mixed liquor suspended solids (MLSS), is generally taken as an index of the mass of active micro-organisms in the aeration tank. However, the MLSS will contain not only active micro-organisms but also dead cells as well as inert organic matter derived from the raw sewage. The mixed liquor volatile suspended solids (MLVSS) value is also used and is preferable to MLSS as it eliminates the effect of inorganic matter. Aerobic and facultative bacteria are the predominant micro-organisms which carry out the above reactions of organic matter i.e. oxidation and synthesis. Their cellular mass contains about 12% Nitrogen and 2% Phosphorous. These nutrients should be present in sufficient quantity in the waste or they may be added, as required, for the reactions to proceed satisfactorily. A generally recommended ratio of BOD:N:P is 100:5:1. Domestic sewage is generally balanced with respect to these nutrients.

5.8.1.2 Activated Sludge Process Variables

An ASP essentially consists of the following: (i) Aeration tank containing microorganisms in suspension in which the reaction takes place, (ii) Activated sludge recirculation system, (iii) Excess sludge wasting and disposal facilities, (iv) Aeration systems to transfer oxygen and (v) Secondary sedimentation tank to separate and thicken activated sludge. These are schematically illustrated in Figure 5.6 (a) to (e). The main variables of the ASP are the loading rate, the mixing regime and the flow scheme.

5.8.1.3 Loading Rate

The loading rate expresses the rate at which the sewage is applied in the aeration tank. A loading parameter that has been developed empirically over the years is the hydraulic retention time (HRT), θ, d.

$$\theta = \frac{V}{Q} \qquad (5.22)$$

Where,

V : Volume of aeration tank, m^3, and
Q : Sewage inflow, m^3/day

Another empirical loading parameter is volumetric organic loading which is defined as the BOD applied per unit volume of aeration tank, per day.

A rational loading parameter which has found wider acceptance and is preferred, is specific substrate utilization rate, U, per day which is defined as:

$$U = \frac{Q(S_0 - S)}{VX} \qquad (5.23)$$

A similar loading parameter is mean cell residence time or sludge retention time (SRT), θc, day:

$$\theta_c = \frac{VX}{Q_w X_s} \qquad (5.24)$$

where S_0 and S are influent and effluent organic matter concentrations respectively, conventionally measured as BOD_5, (g/m^3) X and Xs are MLSS concentration in aeration tank and waste activated sludge from secondary settling tank under flow, respectively, (g/m^3) and Qw - waste activated sludge rate, (m^3/d). Under steady state operation the mass of waste activated sludge is given by

$$Q_w X_s = YQ(S_0 - S) - k_d XV \qquad (5.25)$$

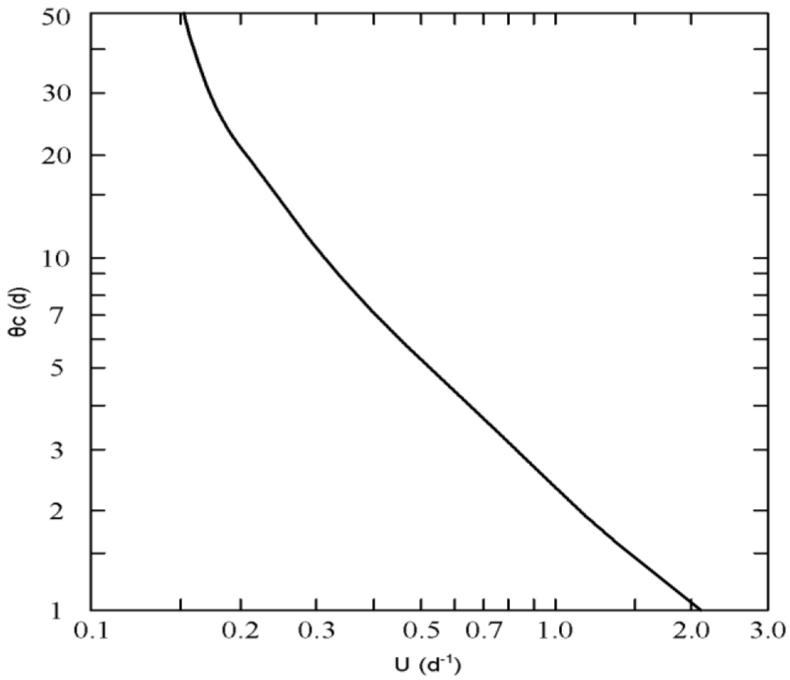
where,

Y : Maximum yield coefficient (microbial mass synthesized/mass of substrate utilized)
k_d : Endogenous respiration rate constant, (d^{-1}).

From the earlier equations it is seen that

$$1/\theta_c = YU - k_d \tag{5.26}$$

Since both Y and kd are constants for a given waste, it is, therefore, necessary to define either θ_c or U. Equation (5.26) is plotted in Figure 5.37 for typical values of Y = 0.5 and k_d = 0.06/d for municipal sewage.



Source: CPHEEO, 1993

Figure 5.37 Relationship between SRT (θ_c) and specific substrate utilization rate (U) for Y = 0.5 and k = 0.06d^{-1}

If the value of S is small compared to S_0, which is often the case for activated sludge systems treating municipal sewage, U may also be expressed as Food applied to Microorganism ratio,

$$F/M = QS_0/XV \tag{5.27}$$

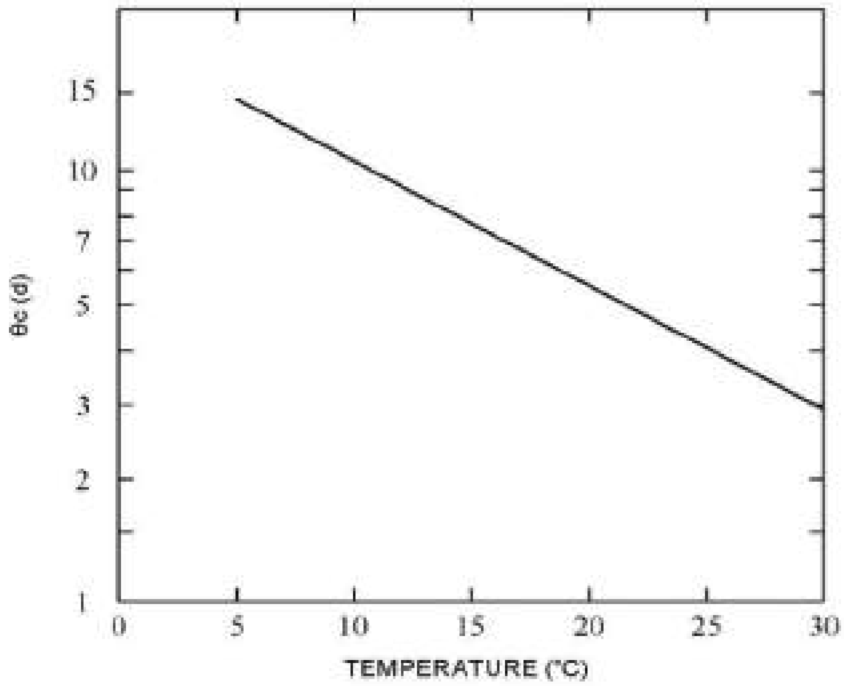
The θ_c value controls the effluent quality, settleability and drainability of biomass.

Other operational parameters which are affected by the choice of θ_c values are oxygen requirement and quantity of waste activated sludge.

Figure 5-38 (overleaf) gives θc value as a function of temperature for 90-95% reduction of BOD of municipal sewage.

5.8.1.4 Design Criteria

Typical values of loading parameters for various activated sludge modifications commonly used in India are furnished in Table 5.9 overleaf.



Source: CPHEEO, 1993

Figure 5.38 SRT as a function of aeration basin temperature for 90-95% BOD removal

Table 5.9 Characteristics and Design Parameters of Activated Sludge Systems for Sewage

Process Type	unit	Flow Regime		
		Conventional	Complete mix	Extended aeration
MLSS	mg/L	1500 to 3000	3000 to 4000	3000 to 5000
MLSS/MLVSS	ratio	0.8	0.8	0.6
F/M	day^{-1}	0.3 to 0.4	0.3 to 0.6	0.1 to 0.18
HRT	Hours	4 to 6	4 to 6	12 to 24
θc	days	5 to 8	5 to 8	10 to 26
Q_R/Q	ratio	0.25 to 0.5	0.25 to 0.8	0.25 to 1.0
BOD removal	%	85 to 92	85 to 92	95 to 98
kg O_2/kg BOD removed	ratio	0.8 to 1.0	0.8 to 1.0	1.0 to 1.2

Source: CPHEEO, 1993

5.8.1.5 Mixing Regime

The mixing regime employed in the aeration tank may be plug flow or completely mixed flow. Plug-flow implies that the sewage moves down progressively along the aeration tank essentially unmixed with the rest of the tank contents. Completely mixed flow involves rapid dispersal of the incoming sewage throughout the tank. In the plug flow system, the F/M and the oxygen demand will be highest at the inlet end of the aeration tank and will then progressively decrease. In the completely mixed system, the F/M and the oxygen demand will be uniform throughout the tank.

5.8.1.6 Flow Scheme

The flow scheme involves the pattern of sewage addition and sludge return to the aeration tank and the pattern of aeration. Sewage addition may be at a single point at the inlet end of the tank or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

5.8.1.7 Conventional System and Modifications

The conventional system represents the early development of the activated sludge process. Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives by modifying the process variables discussed earlier.

In step aeration, settled sewage is introduced at several points along the tank length, which produces uniform oxygen demand throughout. Tapered aeration attempts to supply air to match oxygen demand along the length of the tank. Contact stabilization provides for reaeration of return activated sludge from the final clarifier, which allows a smaller aeration or contact tank. While the conventional system maintains a plug flow hydraulic regime, completely mixed process aims at instantaneous mixing of the influent waste and return sludge with the entire contents of the aeration tank. The extended aeration process employs low organic loading, long aeration time, high MLSS concentration and low F/M. Because of long detention in the aeration tank/oxidation ditch, the MLSS undergo considerable endogenous respiration and get well stabilized. The excess sludge does not require separate digestion and it can be dried directly on sand beds or mechanically dewatered. In addition, the excess sludge production is minimal. The conventional system and the last two modifications named above have found wider acceptance. These are described below in detail.

5.8.1.7.1 Conventional System

The Conventional system is always preceded by primary settling. The plant itself consists of a primary clarifier, an aeration tank, a secondary clarifier, a sludge return line and an excess sludge waste line leading to a digester. The BOD removal in the process is about 85% to 92%. The plant employs a plug flow regime, which is achieved by a long and narrow configuration of the aeration tank with length equal to 5 or more times the width. The sewage and mixed liquor enter at the head of the tank and is withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches. Another limitation of the plug flow regime is that there is a lack of operational stability at times of excessive variation in rate of inflow and in influent strength. For historical reasons, the conventional system is the most widely used type of the activated sludge process. Plants up to 300 MLD capacities have been built in India.

5.8.1.7.2 Completely Mixed

The complete mix activated sludge plant employs a completely mixed flow regime. In a circular or square tank, complete mixing is achieved by mechanical aerators with adequate mixing.

The completely mixed plant has the capacity to hold a high MLSS level in the aeration tank enabling the aeration tank volume to be reduced. The plant has increased operational stability at shock loadings and increased capacity to treat toxic biodegradable wastes like phenols.

5.8.1.7.3 Plug Flow

This occurs in tanks of a longish shape in plan when used with surface aerators and almost all diffused aeration tanks.

5.8.1.7.4 Extended Aeration

The flow scheme of the extended aeration process and its mixing regime are similar to that of the completely mixed process except that primary settling is omitted. The process employs low organic loading, long aeration time, high MLSS concentration and low F/M. The BOD removal efficiency is high. Because of long detention in the aeration tank, the mixed liquor solids undergo considerable endogenous respiration and remain well stabilized. The excess sludge does not require separate digestion and can he dried directly on sand beds. Furthermore, the excess sludge production is minimal. The oxygen requirement for the process is higher and the running costs are correspondingly high. However, operation is rendered simple due to the elimination of primary settling and separate sludge digestion. The method is, therefore, well suited specially for small and medium size communities and zones of a larger city. In small plants intermittent operation of extended aeration systems may be adopted, intermittent aeration cycles are: (i) closing of inlet and aerating the sewage, (ii) stopping aeration and allowing the contents to settle and (iii) letting in fresh sewage which displaces an equal quantity of clarified effluent. Sludge is wasted from the mixed liquor. To handle continuous flows a number of units may be operated in parallel.

The oxidation ditch is one form of an extended aeration system having certain special features like an endless ditch for the aeration tank and a rotor for the aeration mechanism. The ditch consists of a long continuous channel usually oval in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. The sewage is aerated by a surface rotor placed across the channel. The rotor not only aerates the sewage but also imparts a horizontal velocity to the mixed liquor preventing the biological sludge from settling out.

5.8.1.7.5 Design Consideration

The items for consideration in the design of activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge settling and recycle and excess sludge wasting.

5.8.1.7.5.1 Aeration Tank

Equations (5.24) to (5.26) can be combined to yield

$$VX = YQ\theta_c (S_0 - S)/(1 + K_d\theta_c) \tag{5.28}$$

The volume of the aeration tank is calculated for the selected, value of θc by assuming a suitable value of MLSS concentration, X, in Equation (5.28).

Alternatively the tank capacity may be designed from F/M and MLSS concentration according to Equation (5.27). The F/M and MLSS levels generally employed in different types of commonly used activated sludge systems are given in Table 5-9 along with their corresponding BOD removal efficiencies.

It is seen that economy in reactor volume can be achieved by assuming a large value for X. However, it is seldom taken to be more than 5,000 g/m^3. A common range is between 1,000 and 4,000 g/m^3. Considerations that govern the upper limit are

a) Initial and running cost of sludge recirculation system to maintain a high value of MLSS,

b) Limitations of oxygen transfer equipment to supply oxygen at required rate in a small reactor volume,

c) Increased solids loading on secondary clarifier which may necessitate a larger surface area to meet limiting solid flux, design criteria for the tank and minimum HRT for the aeration tank for stable operation under hydraulic surges.

Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. This configuration is achieved by the provision of round-the-end baffles in small plants when only one or two tank units are proposed and by construction as long and narrow rectangular tanks with common intermediate walls in large plants when several units are proposed. In extended aeration plants other than oxidation ditches and in complete mix plants the tank shape may be circular or square when the plant, capacity is small, or rectangular with several side inlets and equal number of side outlets, when the plant capacity is large.

The width and depth of the aeration channel depends on the type of aeration equipment employed. The depth controls the aeration efficiency and usually ranges from 3 m to 4.5 m for surface aerators, the deeper depth being justified by use of hopper bottomed tank square cells and draft tubes. In the case of diffused aeration, the delivery pressure at the compressor plays a crucial part in that, where this exceeds about 6.5 m depth water cooled compressors would be needed and this shall be duly considered. If the capacities are over 70 MLD then duplicate units are preferred.

The width controls the mixing and is usually 5 to 10 m. The width-depth ratio should be adjusted to be 1.2 to 2.2. The length should not be less than 30 m or not ordinarily longer than 100 m in a single section length before doubling back. The horizontal velocity should be around 1.5 m/min. Excessive width may lead to settlement of solids in the tank. Triangular baffles and fillets are used to eliminate dead spots and induce spiral flow in the tanks. The tank free-board is generally kept between 0.3 m and 0.5 m.

Due consideration must be given in the design of aeration tanks for the requirement of emptying them for maintenance and repair of the aeration equipment. Intermediate walls should be designed for empty conditions on each side.

The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tank should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction. The unit dewatering can be as per Section 5.3.13 already detailed in this manual.

The inlet and outlet channels of the aeration tanks should be designed to maintain a minimum velocity of 0.3 m/s to avoid deposition of solids. The channels or conduits and their appurtenances should be sized to carry the maximum hydraulic load to the remaining aeration tank units when any one unit is out of operation.

The inlet should provide for free fall into aeration tank when more than one tank unit or more than one inlet is proposed. The free fall will enable positive control of the flows through the different inlets. Outlets usually consist of free fall weirs. The weir length should be sufficient to maintain a reasonably constant water level in the tank. When multiple inlets are involved, they should be provided with valves, gates or stop planks to enable the regulation of flow through each inlet.

5.8.1.7.5.2 Oxygen Requirements

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and for the endogenous respiration of the micro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required } (g/d) = (Q\,(S_0 - S)/f) - 1.42\,\Delta x \tag{5.29}$$

where,

f : Ratio of BOD to ultimate BOD

1.42 : Oxygen demand of biomass, g/g

Δ_X : Biological sludge produced per day.

$\Delta_X = Q \times Y_{obs} \times (S_o - S)$

$Y_{obs} = Y/(1+K_d \times \theta_c)$

Where Y is 0.5 and K_d is 0.06

The formula does not allow for nitrification but allows only for carbonaceous BOD removal. The extra theoretical oxygen requirement for nitrification is 4.56 Kg O_2/per kg NH_3 -N oxidized to NO_3 – N. The total oxygen requirements per kg BOD removed for different activated sludge processes are given in Table 5.9. The amount of oxygen required for a particular process will increase within the range shown in the Table 5.9 as the F/M value decreases. Appendix A.5.12 presents an illustrative design of conventional ASP aeration.

5.8.1.7.5.3 Aeration Facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the sewage against a specific level of dissolved oxygen in the sewage. The aeration devices, apart from supplying the required oxygen demand shall also provide adequate mixing or agitation in order that the entire mixed liquor suspended solids present in the aeration tank will be available for the biological activity.

The recommended dissolved oxygen concentration in the aeration tank is in the range 0.5 to 1 mg/l for conventional activated sludge plants and in the range 1 to 2 mg/l for extended aeration type activated sludge plants and above 2 mg/l when nitrification is required in the ASP.

Aerators are rated based on the amount of oxygen they can transfer to tap water under standard conditions of 20°C, 760 mm Hg barometric pressure and zero DO. The oxygen transfer capacity under field conditions can be calculated from the standard oxygen transfer capacity by the formula:

$$N = \frac{N_s (C_s - C_L) \times 1.024^{(T-20)} \alpha}{9.17} \tag{5.30}$$

where,

- N : Oxygen transferred under field conditions, kg O_2/kWh
- N_s : Oxygen transfer capacity under standard conditions, kg O_2/kWh
- C_s : Dissolved oxygen saturation for sewage at operating temperature, mg/l
- C_L : Operation DO level in aeration tank usually 1 to 2 mg/1
- T : Temperature, °C
- α : Correction factor for oxygen transfer for sewage,

The value of C_s is calculated by arriving at the dissolved oxygen saturation value for tap water at the operating temperature and altitude as in Table 5.10 and Table 5.11 (overleaf) and then multiply it by a factor (β) which is usually 0.95 for domestic sewage and with TDS in the normal range of 1,200 to 1,500 mg/l.

The value of α requires a detailed understanding. This represents the ratio of the oxygen uptake rate, known as K_{La}, of the given sewage to that of clean tap water at 20 degree Celsius. In simple terms, it is the rate at which oxygen can be dissolved into water and sewage. The K_{La} for water is almost constant. It will however vary in sewage because of the constituents like organic matter, chemicals, biological organisms, detergents, etc, which interfere with the oxygen transfer. Also, the sewage quality itself varies between ULBs depending on water supply rates. Thus the K_{La} value for sewage will always be less than one. The importance of K_{La} as related to the α-value in STP is seen from equation 5.30.

The oxygen requirement at the standard condition denoted as N_s decides the kw of aeration equipment. This value of N_s is inversely proportional to the α-value. Thus, higher value of α-means lesser the kw of aerators and compressors and the entire aeration system. For example, if α-value is 0.8 in one case and 0.4 in another case, the cost of the entire aeration system using the value of 0.4 will be 200% higher. Thus, technically, it becomes highly debatable when specifying this α-value in DPR and in contracts. The second edition of the manual specifies this value as 0.8 to 0.85. This is also corroborated by the publication of Sundaramoorthy & Sundaresan (1972) which evaluated the α- value for the mixed liquor of a conventional ASP at Chennai as 0.847 to 0.854 for a surface aerator system. Many recent tender documents specify it as close to 0.6 in diffused aeration (BWSSB- Contract s1c for 60 MLD average flow BOD removal and Nitrification-denitrification). Even conceding that fine bubble aeration systems are more efficient in dissolving oxygen, Metcalf & Eddy (2003) cites the study by Hwand and Stenstrom (1985) reporting the α value as 0.4 to 0.9 for fine bubble diffuser system. Thus, there is considerable uncertainty in specifying α-value.

Table 5.10 DO Saturation vs. Temperature in Celsius in Tap Water at Mean Sea Level

The relationship between temperature and oxygen solubility	
Temperature(degree C)	Oxygen solubility (mg/l)
0	14.6
5	12.8
10	11.3
15	10.2
20	9.2
25	8.6
30	7.5
35	6.9
40	6.4
100 (boiling)	0.0

Table 5.11 DO Correction Factor for Altitudes

Altitude(meters)	Factor	Altitude(meters)	Factor
0	1	1067	0.88
152	0.98	1219	0.86
305	0.96	1372	0.84
457	0.95	1524	0.82
610	0.93	1676	0.81
762	0.91	1829	0.80
914	0.89		

At the same time, it should be recognized that the compressor capacity needed for ensuring adequate mixing energy is also important. In actual design, the power requirements are calculated separately for aeration & mixing and the higher of the two is chosen. Mostly, the power required for mixing is always higher.

Considering all these, it is considered prudent to opt for a α value of 0.6 for calculating the oxygen and hence the air requirements. If possible this value can also be got tested in the case of upgrading existing STPs

The oxygen transfer capacities of surface, fine and coarse diffused air systems under standard conditions lie between 1.2 to 2.4, 1.2 to 2 and 0.6 to 1.2 kg O_2/kWh respectively. However, it is necessary to secure the test certificates for the same from the diffused air system vendor before deciding on the tendered offers.

5.8.1.7.5.4 Diffused Aeration

Diffused aeration involves the introduction of compressed air into the sewage through submerged diffusers of fine bubble or coarse bubble type. In the former, compressed air is released at or near the bottom of the aeration tank through porous tubes or plates made of aluminium oxide or silicon oxide grains cemented together in a ceramic matrix. Troubles due to clogging from the inside can be reduced by providing air filters and those due to clogging from outside can be avoided by providing adequate air pressure below the diffusers at all times. In spite of such precautions, fine bubble diffusers will require periodical cleaning. Air supplied to porous diffusers should not contain more than 0.02 mg of dust per cum of air. Coarse bubble aerators have lower aeration efficiency than fine bubble aerators, but are cheaper in capital cost and are less liable to clogging and do not require filtration of air. In longish channel type aeration tanks, air diffusers are generally placed along one side of the aeration tank, helping to set up a spiral flow in the tank which improves mixing and prevents the solids from settling. They are located 0.3 m to 0.6 m above the tank floor to aid in tank cleaning and reduce clogging during shutdown. The air volume calculated based on the α-value shall be further adjusted as follows.

a) Percentage of oxygen in the air at the STP location

b) Weight of the air at the STP location

c) Efficiency of the diffusers in transferring the air at the given liquid depth of the aeration tank at the rate of 4 % to 5 % per metre depth.

d) Fouling factor of diffusers at the rate of 4 % to 5 % per year over its life span

e) An overall factor of safety of 10 % to take care of contingencies.

The compressor shall be provided for the above duty and it shall have a VFD so that the actual air requirement and hence the actually required kW alone can be operated to maintain the required residual oxygen in aeration.

The agitator-sparger is a mechanical aerator system involving the release of compressed air at the bottom of the aeration tank in large bubbles and the breaking up of the bubbles into fine bubbles by submerged turbine rotors located above the air outlets are also used.

5.8.1.7.5.5 Surface Aerators

Surface aerators are available in both fixed and floating types. Some of their advantages are higher oxygen transfer capacity, absence of air piping and air filter and simplicity of operation and maintenance. Surface aerators generally consist of large diameter impeller plates revolving on vertical shaft at the surface of the liquid with or without draft tubes. A hydraulic jump is created by the impellers at the surface causing air entrapment in the sewage. The impellers also induce mixing. The speed of rotation of the impellers is usually 70-100 rpm for geared motor systems. The aeration rotors for small oxidation ditches are generally of cage type but may also be of the angle iron type. Particular attention must be paid to the design of shaft length, bearings and alignment. Vertical shaft aerators are easier to maintain and are used with deeper ditches.

5.8.1.7.5.6 Mixing Requirements

The aeration equipment has also to provide adequate mixing in the aeration tank to keep the solids in suspension. The air requirements shall be calculated both for summer and winter as well as mixing power and the higher duty installed. Mixing considerations require that the minimum power input in activated sludge aeration tanks where MLSS is of the order of 4000-5000 mg/l, should not be less than 15-26 W/m^3 of tank volume. The power input of aerators derived from oxygenation considerations should be checked to satisfy the mixing requirements and increased where required. In the case of diffused aeration, the air volume for mixing shall be not less than 1.8-2.7 m^3/hr/m^2 of floor area (US EPA, 625/8-85/0100, p 38). The delivery head shall be as per the chosen liquid depth and friction losses. The surface area of the diffusers shall not be less than 6% of the floor area of the aeration tank. In the case of tubular diffusers, the centre to centre spacing shall be preferably restricted to not over 30 cm and where unavoidable, the interspaces shall be provided with pre-cast RCC ridges so that the MLSS if it settles down will slide to the diffusers and will be automatically pushed up into the aeration tank by the buoyancy. The loss of aeration tank volume by these ridge blocks and blocks for supporting the diffuser headers shall be compensated in deciding the liquid height of the aeration tank.

5.8.1.7.5.7 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each aeration tank. For plants designed for sewage flow of 10 MLD or more, integrating flow recorders should be used.

5.8.1.7.5.8 Secondary Settling

Secondary settling assumes considerable importance in the activated sludge process as the efficient separation of the biological sludge is necessary not only for ensuring final effluent quality but also for return of adequate sludge to maintain the MLSS level in the aeration tank. The secondary settling tank of the activated sludge process is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. The high concentration of suspended solids in the effluent requires that the solids loading rate should also be considered. The recommended overflow rates and solids loading rates for secondary clarifiers of activated sludge have been given in Table 5.8.

5.8.1.7.5.9 Sludge Recycle

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settle ability and thickening in the secondary sedimentation tank.

$$\frac{Q_R}{Q} = \frac{X}{X_s - X} \tag{5.31}$$

where,

Q_R : Sludge recirculation rate, m^3/d.

The sludge settleability is determined by sludge volume index (SVI) defined as volume occupied in ml by one gram of solids in the mixed liquor after settling for 30 min and is determined experimentally. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_s = 10^6/SVI$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids and this can be achieved for values suggested in Figure 5.38. The X_s value may not be taken more than 10,000 g/cum unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value. Using the above value for X_s and 5000 mg/l for X in Equation (5.31), the sludge recirculation ratio comes out to be 1.0. The return sludge is always to be pumped and the recirculation ratio should be limited to the values suggested in Table 5.9.

As stated above, the recirculation ratio computation depends on the concentration of the sludge in the underflow of the clarifier and this in turn can be attributed to the SVI as mentioned. The SVI is a plant control parameter and cannot be assumed as a design parameter. Thus, the concentration of the sludge in the underflow of the clarifier is again not possible to pre-fix in design. Normally well operated clarifiers can be expected to concentrate the MLSS of mixed liquor by about 3 times. Thus, the thumb rule recirculation ratio can also be expressed as 1/(3-1) = 0.5. However, the thumb rule indicates a value of 0.25 to 0.8 in Table 5-9. Moreover, there has to be flexibility in the field to vary the recirculation ratio nearer to the higher limit to reach adequate flows and hence maintain velocities in piping through the plant when the influent sewage volume is very much less. Thus, it is recommended that irrespective of the designer’s choice, the recirculation pump set shall be designed to deliver the higher volume but in actual practice the pumpage can be controlled to the bare minimum through a VFD control.

5.8.1.7.5.10 Excess Sludge Wasting

The sludge generated in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. The excess sludge generated under steady state operation may be estimated from Equation (5.24) and (5.25).

In the case of domestic sewage, the excess sludge to be wasted will be about 0.35-0.5 kg/kg BOD_5 removed for the conventional system and about 0.25-0.35 kg/kg BOD_5 removed in the case of extended aeration plants having no primary settling.

The volume of sludge to be wasted will depend on the suspended solids concentration in the waste stream.

Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter procedure is to be preferred as the concentration of suspended solids will then be somewhat steady in the waste stream providing better control on biomass wasted.

The waste sludge is either discharged into the primary settling tank or thickened in a sludge thickening unit and digested directly, In extended aeration plants, the excess sludge is taken to sludge drying beds or mechanical dewatering directly and the sludge filtrate discharged into the effluent stream.

Excess sludge = (A/(0.6 to 0.8)) + B

A is calculated by the following equation and 0.6 to be used for extended aeration and 0.8 is used for conventional activated sludge.

$A = Q \times Y_{obs} (S_0 - S)$

$Y_{obs} = Y/(1+K_d \times \theta_c)$

Where Y is 0.5

K_d is 0.06

B = Q×inert TSS removal

Inert TSS = Influent TSS – Influent VSS

TSS removal in primary settling tank is 60%.

Inert SS removal in primary settling tank is 60%.

VSS removal in primary settling tank is 60%.
θ_c is from Figure 5-38 for the lowest operating temperature.

Excess sludge in kg/day = Y_{obs} × BOD inlet × Flow MLD

Calculate excess sludge kg/day from the thumb rule in Section 5.8.1.7.5.10

Adopt the higher value

Excess sludge volume m^3/day = (Excess wasted kg/day) × 1000/MLSS in clarifier underflow

MLSS in clarifier underflow is to be assumed based on the SWD and is usually 3 times the MLSS.

5.8.1.7.6 Nitrification

Activated sludge plants are ordinarily designed for the removal of only carbonaceous BOD. However, there may be incidental nitrification in the process. Nitrification will consume part of the oxygen supplied to the system and reduce the DO level in the aeration tank. Nitrification will also lead to subsequent denitrification in the secondary clarifier causing a rising sludge problem also called blanket rising. Nitrification is aided by low F/M and long aeration time. It may be pronounced in extended aeration plants especially in hot weather. At the other extreme in the contact stabilization process and in the modified aeration plant, there may be little or no nitrification.

Nitrification though generally not desired may be required in specific cases, e.g. when ammonia has to be eliminated from the effluent in the interest of pisciculture or when nitrification cum denitrification is proposed for elimination of nitrogenous matter from the effluent for control of eutrophication. In such cases, plug flow systems have been developed for efficient removal of both carbon and nitrogen, Alternatively a two stage system may be designed with carbonaceous BOD removal in the first stage and nitrification in the second stage by ensuring adequate organic matter is still left behind at the end of the first stage to serve as the energy source for the nitrifying organisms in the second stage.

Nitrification requires bicarbonate alkalinity in the ratio of seven times that of the ammonia to be nitrified and if the available alkalinity is inadequate, the addition of Sodium carbonate or Bicarbonate is needed before the aeration tank.

5.8.1.7.7 Denitrification

In general, this is achieved as an integrated nitrification-denitrification process as a variation of the typical activated sludge process. The principle is shown in Figure 5.39 and the flow scheme is shown in Figure 5.40.

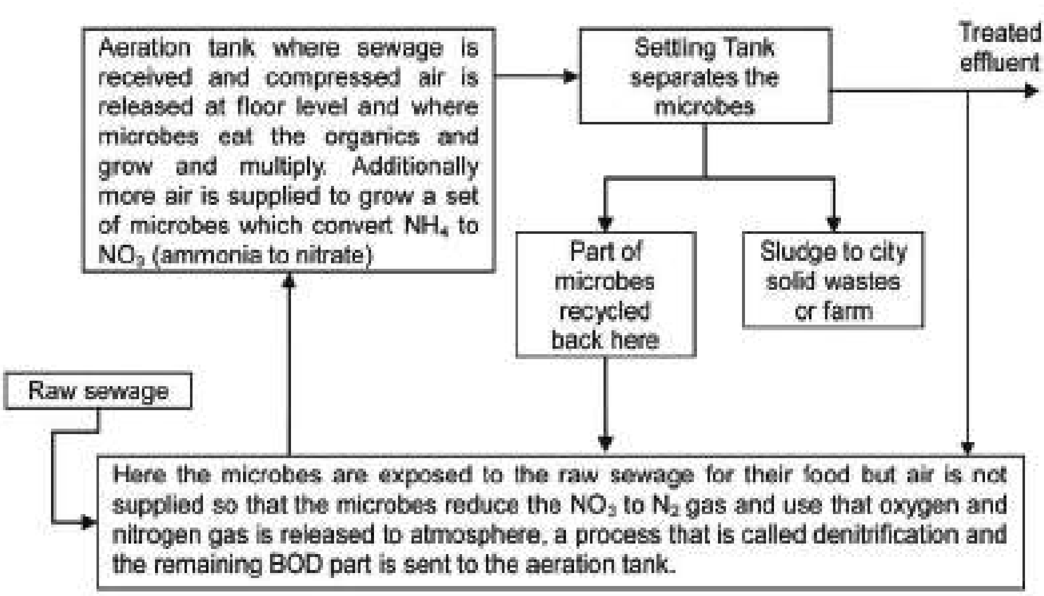


Figure 5.39 Schematic of biological nitrification-denitrification in activated sludge process

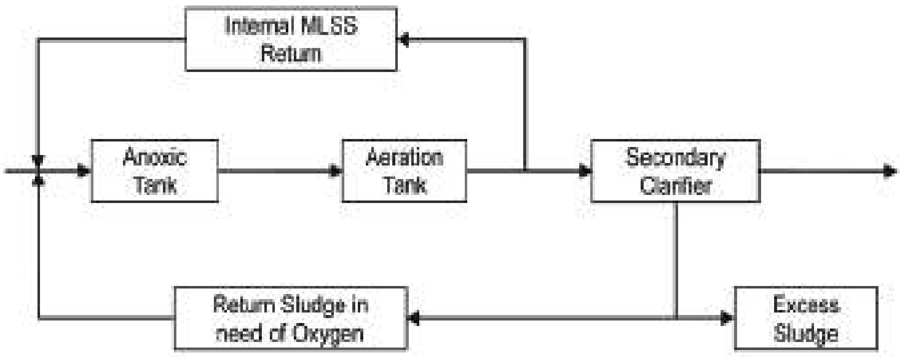


Figure 5.40 Flow routing in Activated Sludge Biological Nitrification-Denitrification process

5.8.1.7.8 Phosphorous Removal

The consciousness to restrict the phosphorous in the treated sewage before discharge into the environment to curtail eutrophication is being recognized. The phosphorous can be removed by a process called as the luxury uptake. There are at least six different variations of these processes which have all been developed in advanced countries and every situation will need a separate evaluation and validation.

An alternative process is to introduce a chemical precipitation either in the secondary clarifier or as a separate tertiary stage where phosphorous is precipitated by coagulating with Ferric or Aluminium salts. There is also another technology of high Lime followed by acidification or carbonation whereby in addition to phosphorous removal, colour, heavy metals, fluorides, silica and magnesium can also be simultaneously removed. It is necessary to conduct lab studies to establish the efficiency and the type of chemicals.

5.8.1.7.9 Aerated Lagoons

Aerated lagoons are generally provided in the form of simple earthen basins with inlet at one end and outlet at the other to enable the sewage to flow through while aeration is usually provided by mechanical means to stabilize the organic matter. The major difference between activated sludge systems and aerated lagoons is that in the latter, settling tanks and sludge recirculation are absent. Aerated lagoons are of two principal types depending on how the microbial mass of solids in the system is handled. Facultative Aerated Lagoons are those in which some solids may leave with the effluent stream and some settle down in the lagoon since aeration power input is just enough for oxygenation and not for keeping all solids in suspension. As the lower part of such lagoons may be anoxic or anaerobic while the upper layers are aerobic, the term facultative is used. Appendix A.5.13 presents an illustrative design of facultative aerated lagoon.

Aerobic Lagoons, on the other hand, are fully aerobic from top to bottom as the aeration power input is sufficiently high to keep all the solids in suspension besides meeting the oxygenation needs of the system. No settlement occurs in such lagoons and under equilibrium conditions the new (microbial) solids produced in the system equal the solids leaving the system. Thus, the solids concentration in the effluent is relatively high and some further treatment is generally provided after such lagoons. If the effluent is settled and the sludge recycled, the aerobic lagoon, in fact, becomes an activated sludge or extended aeration type lagoon. A few typical characteristics of the above types of lagoons are given in Table 5.12.

Table 5.12 Some Characteristics of Aerated Lagoons

No.	Characteristics	Facultative Aerated Lagoons	Fully Aerobic	Extended Aeration System (for comparison)
1.	Detention time, days	3 - 5	2 - 3	0.5 - 1.0
2.	Depth, m	2.5 – 5.0	2.5 – 4.0	2.5 - 4.0
3.	Land required, m^2/person	0.15 - 0.30	0.10 - 0.20	
4.	BOO removal efficiency %	80 - 90	50 - 60	95 - 98
5.	Overall BOD removal rate, K (A)	0.6 – 0.8	1-1.5	20 - 30
6.	Suspended solids in lagoon, mg/l	40 - 150	150 - 350	3,000 - 5,000
7.	VSS/SS	0.6	0.8	0. 6
8.	Desirable power level (B)	0.75	2.75 - 6.0	15 - 18
9	Power requirement, kWh/person/year	12 - 15	12 - 14	16 - 20

Source: CPHEEO,1993

(A) Per day at 20 degree C for soluble BOD only; (B)-in watts per cum of lagoon volume

Facultative type aerated lagoons have been more commonly used the world over because of their simplicity in operation and minimum need of machinery. They are often referred to simply as 'aerated lagoons'. Their original use came as a means of upgrading overloaded oxidation ponds in some countries without adding to the land requirement. In fact, much less land is required compared to oxidation ponds.

In earlier times the design of aerated lagoons was often done using simple thumb-rules of detention time and power per capita. However, over the years it has come to be recognized that lagoons being large bodies of water are subject to seasonal temperature effects and flow mixing conditions. Flow conditions in aerated lagoons are neither ideal complete-mixing nor ideal plug-flow in nature. They are dependent on lagoon geometry and are better described by dispersed flow models of the type given by Wehner and Wilhem for first-order kinetics and hence the design procedure given below takes treatability of the waste, temperature and mixing conditions into account. Fully aerobic lagoons always have a complete-mixing regime and a slightly different mode of design is followed. However, as aerobic lagoons have not yet been built in India (except one case) further discussion is limited to facultative aerated lagoons only.

5.8.1.7.10 Design Variables

For facultative aerated lagoons, the dispersed flow model just referred to gives the relation between influent and effluent substrate concentrations, S_0 and S, respectively and other variables such as the nature of the waste, the detention time and the mixing conditions, as shown in the following equation.

$$\frac{S}{S_0} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad (5.32)$$

in which the term $a = \sqrt{1 + 4K\theta d}$

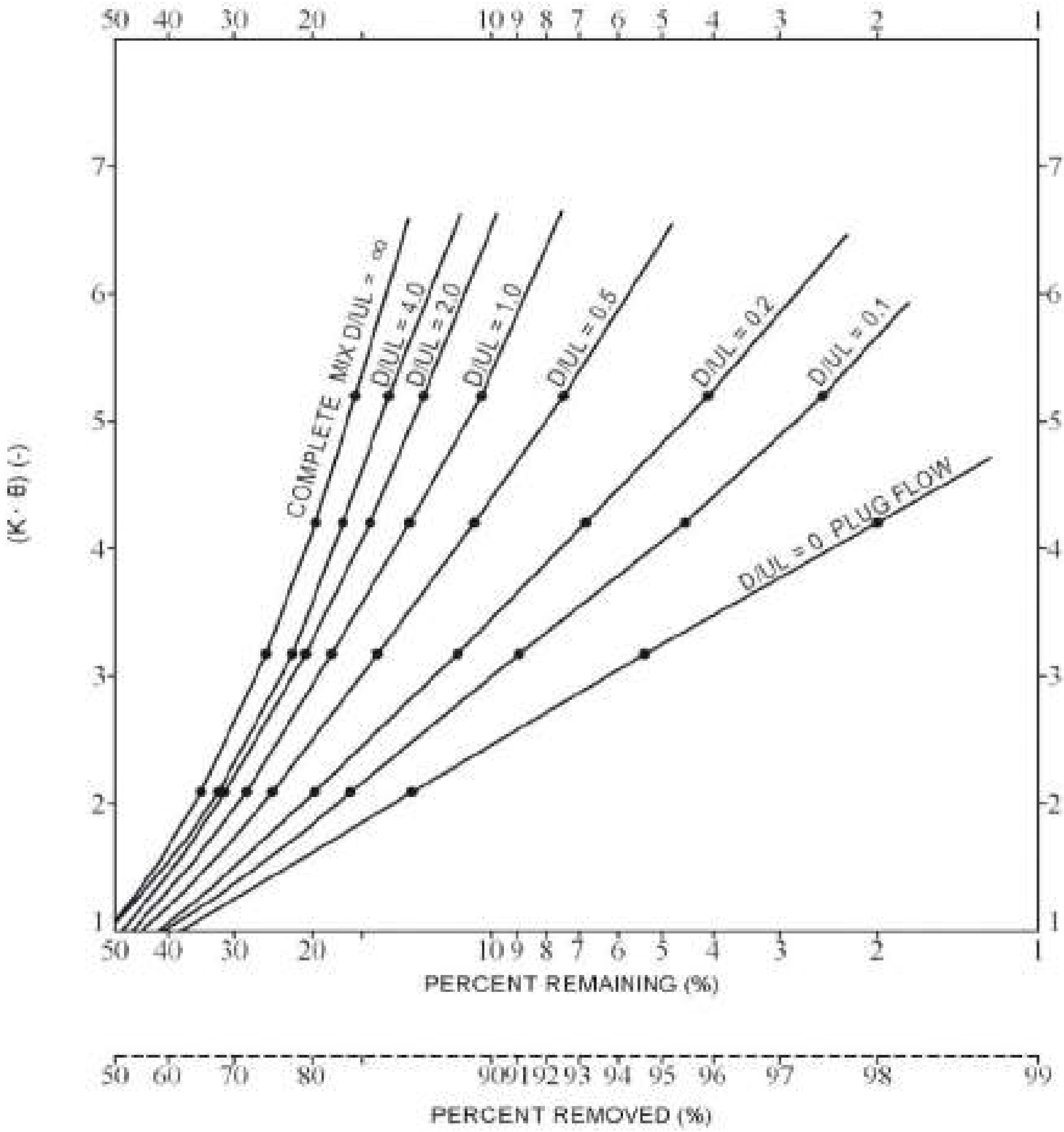
d : dispersion number (dimensionless)

$= D/UL = D\theta/L^2$

where,

- D : Axial dispersion coefficient (length^2/time)
- L : Length of axial travel path
- θ : Theoretical detention time. (Volume/Flow rate)
- U : Velocity of flow through lagoon (length/time)
- K : Substrate removal rate in lagoon (time^{-1})
- S_0 & S : Initial and final substrate concentrations (mass/volume)

A graphical solution of the above equation is shown in Figure 5.41 (overleaf) from which it is seen that prior knowledge of the substrate removal rate K as well as of the mixing condition likely to prevail in a lagoon is necessary to determine the efficiency of BOD removal at selected detention time. This is discussed further.



Source: CPHEEO, 1993

Figure 5.41 Substrate removal efficiency using the dispersed flow model (Wehner - Wilhem equation)

5.8.1.7.11 Mixing Conditions

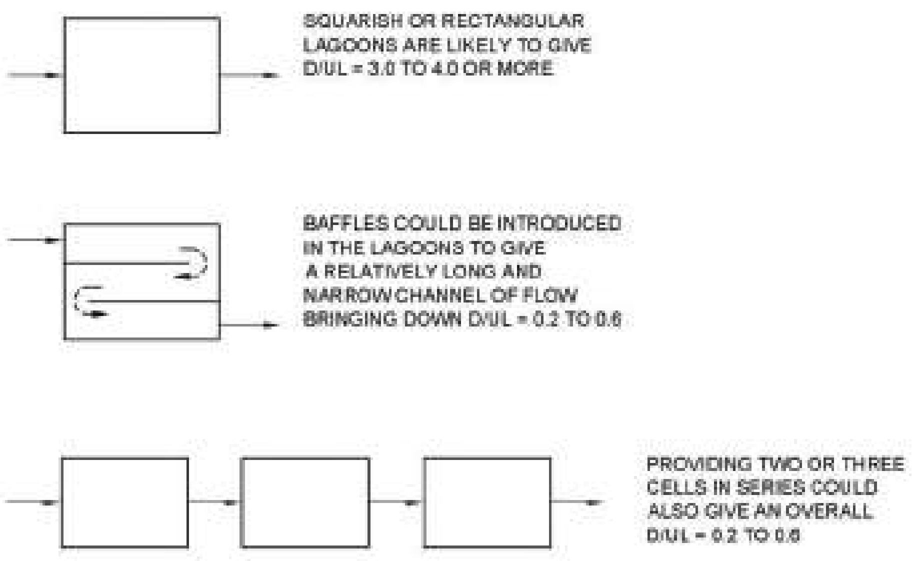
The mixing conditions in a lagoon are reflected by the term ‘d’ which is known as the “Dispersion Number” and equals (D/UL) or (D/L^2). It is affected by various factors. Observed results have shown the (D/UL) values to be in the approximate range given in Table 5.13 (overleaf) for different length-width ratios of lagoons. By suitable choice of a lagoon’s geometry one can promote either more plug flow or more complete mixing type of conditions. In case of cells in series, each cell may be well mixed with value of D/UL approaching 3.0 or 4.0 but overall the arrangements would give a relatively plug-flow type arrangement.

Table 5.13 Likely Values of Dispersion Numbers D/UL at Different Length –Width Ratios

Aerated Lagoon	Approximate range of D/UL values	Typical mixing condition
Length to width ratio 1:1 to 4:1	3.0 to 4.0 and over	Well mixed
Length to width ratio 8:1 or more	0.2 - 0.6	Approaching plug flow
Two or Three cells in series	0.2 – 0.6 (overall)	- do-

Source: CPHEEO, 1993

The values of D/UL can be determined by conducting dye (tracer) tests on existing units using well-known methods, but where D/UL values are required for design purposes prior to construction; they can be estimated either from lab-scale models or by using empirical equations. Low values of D/UL signify plug flow conditions and generally give higher efficiencies of substrate removal whereas the converse is the case with higher values of D/UL. However, process efficiency is not the only consideration; process stability under fluctuating inflow quality and quantity conditions, has also to be kept in view. For municipal or domestic sewage, relatively plug flow type conditions (i.e. low values of D/UL) are preferred. In case of industrial wastes, relatively well mixed condition may be preferred (i.e. higher values D/UL) depending upon the nature of the industrial waste; the greater the fluctuations in quality and quantity of industrial wastes, the greater the advantage in adopting well-mixed conditions. Figure 5.42 gives some examples of different types of arrangements using baffles or cells in series.



Source: CPHEEO, 1993

Figure 5.42 Estimated Effect of Lagoon Geometry D/UL

Lagoons are generally rectangular though it is not particularly essential. Natural land contours may be followed to the extent possible to save on earthwork. Lagoon units may be built with different length-width ratios and arrangement of internal baffles to promote desired mixing conditions. Lagoons may also be provided as two or three stage systems with the subsequent units placed at a lower level than the first if desired.

The construction techniques for aerated lagoons are similar to those used in case of oxidation ponds with earthen embankments. Pitching of the embankment is desirable to protect it against erosion. In cases where soil percolation is expected, suitable lining may have to be provided to maintain the design level in the lagoon and avoid ground water pollution.

5.8.1.7.12 Substrate Removal Rates

As shown in Table 5-12 for facultative aerated lagoons the overall substrate removal rate K for sewage varies from 0.6-0.8 per day (soluble BOD basis) at 20ºC. At other temperatures in lagoons the values are obtained from:

$$(K)_{T^{\circ}C} = (K)_{20^{\circ}C} \times 1.035^{(T-20)} \tag{5.33}$$

The temperature in a lagoon T_L is estimated from the following equation:

$$\frac{\theta}{h} = \frac{T_i - T_L}{f(T_L - T_a)} \tag{5.34}$$

Where

Θ = detention time in days,

h = depth of lagoon in meters,

T_i and T_a are the temperatures (°C) of influent sewage and ambient air respectively and

f = the heat transfer coefficient for aerated lagoons which is 0.49 m/day.

The average winter month temperature is critical for determining the detention time required. As stated earlier, the detention time to be provided in a lagoon can be determined from Equation (5.32) or Figure 5.42 for any desired efficiency for the computed temperature and mixing conditions in the lagoon.

5.8.1.7.13 Power Level

The power input in facultative aerated lagoons has to be adequate only to diffuse dissolved oxygen uniformly in the system and no effort is made to keep the solids in suspension. Hence, a minimum power level of 0.75 watts per cum lagoon volume should be adequate, but this should be checked with the aeration equipment supplier for its oxygenation characteristics and compatibility with proposed depth and shape of lagoon. For treating sewage the power requirement varies from 12 to 15 kWh/person/year or 2 to 2.5 HP per 1,000 population equivalent.

The oxygenation capacity of aerators is reported to range from 1.87 to 2.0 kg Oxygen/kWh at standard conditions for power delivered at shaft. Spacing of aerators should be adequate for uniform aeration all over the lagoon area without much overlap of the circle of influence of adjoining aerators as specified by the manufacturers. A minimum of two aerators would be desirable to provide the required make up for the total power requirement.

Aerators ranging from 3 HP to 75 HP are now readily available in the country. They can be either floating or fixed type. Floating aerators are mounted on pontoons which should be corrosion-free and which have the advantage of being able to adjust themselves to actual levels obtaining in the lagoons due to seepage and / or fluctuating inflows. Fixed aerators are mounted on columns and levelled with regard to the outlet weir level to ensure required submergence.

5.8.1.7.14 Effluent Characteristics

The effluent is generally made to flow over an outlet weir. As the concentration of solids passing out in the effluent may be nearly the same as that in the lagoon, the BOD corresponding to the volatile fraction of these solids (assumed as 0.77 mg per mg VSS in effluent) should be added to the value of the soluble BOD, obtained by use of Equation (5.32) or Figure 5-41. Thus, the final effluent BOD is given by:

$$\text{Final BOD, mg/l} = \text{Soluble BOD, mg/l} + (0.77)\ (\text{VSS in effluent}),\ \text{mg/l} \qquad (5.35)$$

It is because of the suspended solids (expected to range from 40 to 60 mg/l in case of sewage in the final effluent that the total effluent BOD is difficult to reduce below 30-40 mg/l in winter. At other times of the year BOD of less than 30 mg/l may be possible. This range of BOD is more than adequate for irrigation purposes. For river discharge, the applicable standards should be ascertained and design made accordingly, Where necessary, further reduction of BOD can be achieved either by a small increase in detention time or by more efficient interception of solids flowing out (e.g. deeper baffle plate ahead of outlet weir) or by provision of an additional treatment unit. Nitrification is not likely to occur in aerated lagoons, Coliform removal shows considerable seasonal variation (60 - 90% removal).

5.8.1.7.15 Sludge Accumulation

Sludge accumulation occurs at the rate of 0.03 to 0.05 cum per person per year as in the case of oxidation ponds and is manually removed once in 5-10 years and used as good agricultural soil filler. The depth of the lagoon may be increased a little to allow for sludge accumulation, if desired.

5.8.1.7.16 Conclusion

The removal efficiencies in terms of power input are comparable to some of the other aerobic treatment methods seen earlier in this chapter but the greatest advantage with aerated lagoons lies in their simplicity and ruggedness in operation, the only moving piece of equipment being the aerator. Civil construction is mainly earthwork, and land requirement is not excessive.

5.8.2 Attached Growth Systems

5.8.2.1 Historical Development of Attached Growth in Rock Media Filters

The earliest known attached growth systems were cases of raw sewage cascading over rock beds in river courses and microbes growing over the rock surfaces thus bringing about a variety of aerobic metabolism in the upper layers, anaerobic metabolism in the benthic layers and facultative metabolism in the intermediate sections.

The nearest to this can be seen in Rajneesh Ashram in Pune, where the raw sewage of a nearby economically weaker section habitation is diverted into a similar manmade cascading nullah and a light forestry is grown on both sides to encourage evapotranspiration. There are locations where benches have been put up along the course where people can sit and no foul odour perceptible. At the end of the nullah, the treated sewage is clear, odourless and colourless and is pumped over a mini rock built water fall. It is aesthetically a acceptable quality perhaps used downstream for agriculture and horticulture. Scientific data is not available on actual quality improvement, but is an accepted solution by the local population in the absence of an organized STP.

5.8.2.2 Development of Modern Synthetic Media

The trickling filter and intermittent sand filters are the earliest treatment processes. The trickling filter media was rock media of about 100 mm to 150 mm stones loosely placed by hand. Design data were evolved for these trickling filters by compiling the data on their performances especially in the USA and the famous National Research Council (NRC) and Ten State Standards were purely mathematical equations of best fit of the data. Eckenfelder Sr and Rankin were some authors who postulated theoretical approaches and these were used in some situations. However, with passage of time, the stone media has been given up the world over. In India also, the largest known installation at Piranha sewage farm has been since converted to ASP. The reasons were mainly the clogging and choking of the flow channels between the rocks and under drains due to the slow erosion of the stone and due to microbial corrosion and attrition. Consequently it was needed to physically remove and repack the whole filter volume of rocks. The rotary reverse jet distributor also created its own problems due to grit settling in the arm ducts and the turn-table immediately being stuck. This differential weight crushed the ball race and twisted the ball retainer rings in the turn-table. This has prompted innovations whereby light weight synthetic media of much higher surface area per unit volume have come up in the market.

The variations in physical arrangement are:

a) Fixed Film Reactors (FFR) which are attached growth on fixed film on stationary media and the applied sewage trickles down the exposed surfaces of the media.

b) Submerged Fixed Bed Reactors (SFBR) are attached growth on fixed film on submerged stationary media in the reactor and sewage flows through the media upward. The commercially known technologies as Submerged Aeration Fixed Film (SAFF), Rotating Biological Contactor (RBC), Fixed Bed Biofilm Activated Sludge Process (FBAS), etc., come under this and can again be either aerobic or anaerobic.

In both these cases, the microbes grow on the surfaces of the media and increase in thickness by the subsequent microbes adhering on the previous film and once the thickness becomes weighty the microbial film sloughs off the media and fresh microbes start developing. This is a cyclic process and the degree of organic matter removal can be intermittently fluctuating.

The advantage of the FFR is that huge liquid retaining reactors like aeration tanks need not be constructed and the FFR can be designed and constructed as simple silos and save the energy for mixing the contents.

These have to be structurally designed as liquid retaining structures because; it is necessary to flush the media periodically when organisms have grown too much and choke the flow pathways. The flushing is done by filling the entire height of the reactor with sewage after closing the outlet gate and suddenly draining it by opening the gate.

5.8.2.3 Physical Features

5.8.2.3.1 Shape of Reactors

Reactors may have circular, rectangular or square shape. Fixed nozzles or nozzles mounted on moveable arms are used for flow distribution. Rectangular square or circular shapes are used and the circular shape has the advantage of structural economy.

5.8.2.3.2 Provision for Flooding

This is needed in the case of FFR. Provision for intentional flooding and sudden draining of the reactors is useful for controlling filter flies and ponding. To enable flooding, the reactor walls must be designed for the internal water pressure and the main collecting channel must be placed inside the filter and provided with gate valves. An overflow pipe leading from the filter to the main collecting channel downstream from the gate valve is also necessary. Provision for filter flooding should always be made in the case of small reactors. Such a provision in large reactors would not only increase the cost but also cause hydraulic problems with the sudden discharge of large volumes of sewage when the flooded reactor is drained. In such cases alternate methods are needed. These can be pumped spray of treated sewage by hosing it and also air scouring simultaneously from the bottom.

5.8.2.3.3 Side Walls

The side walls of FFR and SFBR shall be RCC or brickwork subject to structural requirements of water pressure on side walls.

5.8.2.3.4 Floor

The floor is designed to support the under-drainage system and the superimposed filter media. The usual practice is to provide an RCC slab over a proper levelling course with slope between 0.5% and 5% towards the main collecting channel. The flatter slopes are used in larger reactors. The floor shall permit installation of fixed air headers for fixing diffuser elements therein and provision for gate controlled draining of the reactor.

5.8.2.3.5 Under Drainage System for FFR

5.8.2.3.5.1 Slope of Under Drains

The under drainage system is intended to collect the trickling sewage and sloughed solids and to convey them to the main collecting channel and to ventilate the media. The under drain covers the entire floor of the reactor to form a false bottom and consists of drains with semi-circular or equivalent inverts.

They can be formed of precast vitrified clay or concrete blocks, complete with perforated cover or they may be formed in-situ with concrete or brick and covered with perforated precast concrete slabs. The slope of the under drain should be the same as that of the floor. The drains shall be so sized that flow occupies less than 50% of the vertical cross-sectional area with velocities not less than 0.6 m/s at average design flow. The cover over the drains shall be perforated to provide a total area of not less than 15% of the surface area of the filter as inlet openings into the drains. The under drains may be open at both ends so that they may be inspected easily and flushed out if they become clogged.

5.8.2.3.5.2 Main Collecting Channel

The main collecting channel is provided to carry away the flow from the under drains and to admit air to the reactor. In a circular reactor, the main channel may be located along the diameter with a slight offset from the centre. Alternatively the channel may be provided along the outer periphery of the reactor. If inside the reactor, the channel shall be provided with perforated covers to enable drainage and ventilation of the reactor media above the channel. The channel should be extended outside the reactor, both at the upper end and lower ends with vented manholes to facilitate ventilation and access for cleaning. The channels shall have semi-circular or other rounded inverts. The velocity in the channels shall not be less than 0.6 m/s for the average hydraulic loading. The flow shall be only half-depth particularly where recirculation is low. At the peak instantaneous hydraulic loading, the water level in the channel should not rise above the inverts of the under drains at their junctions with the channel.

5.8.2.3.5.3 Ventilation

Adequate natural ventilation can be ensured by proper design of the under drains and effluent channels. For reactors larger than 30 m dia., a peripheral head channel on the inside of the reactor with vertical vents is desirable to improve ventilation. One m^2 of open grating in ventilating manholes and vent stacks should be provided for 250 m^2 of reactor area. The vertical vents can also be used for flushing the under drains. In extremely deep or heavily loaded reactors there may be some advantage in forced ventilation if it is properly designed, installed and operated. Such a design should provide for air flow of one $m^3/min/m^2$ of reactor area in either direction. It may be necessary during periods of extremely low air temperature to restrict the flow of air through the reactor to keep it from freezing. However a minimum air flow of 0.1 $m^3/min/m^2$ of reactor area should be provided.

5.8.2.3.5.4 Reactor Media

The requirements for reactor media are high specific surface area, high percentage of void space, resistance to abrasion and good structural strength to withstand deformation during placement, insolubility in sewage and resistance to spalling and flaking. The media shall be of virgin material of PVC or PE or HDPE. Recycled materials shall not be used.

5.8.2.3.5.5 Synthetic Media

Synthetic reactor media have of late been used successfully in super rate reactors for the treatment of strong industrial wastes or sewage mixed with strong industrial wastes.

The hydraulic loading rates are between 40 to 200 $m^3/d/m^2$ and organic loading rates between 0.8 to 6.0 kg BOD/d/m^3. The media consists of interlocking sheets of plastics which are arranged in a honeycomb fashion to produce a porous and non-clog reactor media. The sheets are corrugated so that a strong, lightweight media pack is obtained. Reactors as deep as 12 m are reported to have been used with this type of synthetic media.

5.8.2.3.5.6 Reactor Dosing

In the case of low rate reactors, the minimum flow rate of sewage inflow may not be sufficient to rotate the distributor and discharge sewage from all nozzles. Hence, when adequate head is available dosing tank is provided to collect the settled sewage and dose the reactor through a siphon intermittently. When head is not adequate, a collection well and pump is provided. The dosing siphons are designed to dose the reactors once in about 5 minutes under average flow conditions. In the case of high rate reactors, there is no need for the special dosing device since continuous dosing is possible.

5.8.2.3.5.7 Flow Distribution

Fixed nozzle distributors are not preferred because of the elaborate piping requirement and the necessity of dosing tanks, siphons or motor operated valves to obtain variable dosing rates. Moreover, physical access to each nozzle for cleaning requires the operator to walk over the slippery slime on the media top surfaces which is risky. Among the moving types, the longitudinally travelling distributors with limit switches at each end is a solution but the inlet arrangements from a fixed discharge location to the moving off take of the distributor arms is a challenge for design and upkeep needing bellows etc. The alternative practice is the reverse jet rotary distributors which generate the propulsion by the reverse jets on opposite diametrical arms, but here again, the problems cited already in Section 5.8.2.2 are important. The modern method is to facilitate a peripheral electrically operated drive similar to edge driven bridges of clarifiers and these are commercially available in the country up to 60 m dia. The central feed pipe to the well of the distributor is generally taken up from below the reactor floor or just above the under drains and through the reactor media. The pipe should be designed for a peak velocity of not greater than 2.0 m/s and an average velocity not less than 1 m/s.

The reaction type rotary distributor consists of a feed column at the centre of the reactor, a turn table assembly and two or more hollow radial distributor arms with orifices. The turntable should be provided with anti-tilt devices and arrangements for correcting the alignment to obtain balanced rotation. The turntable assembly is provided with a mercury or mechanical water seal at its base. The current trend is to discourage mercury seals because of the chances of causing mercury pollution. Facilities should be available for draining the central column of the flow distributor for attending to repairs and maintenance.

The distributor arms are generally two in number and multiples of two are being adopted. When multiple arms are provided, low flows are distributed through two arms only and as flow increases, it is distributed by the additional arms. This is achieved by overflows from weirs incorporated in the central column diverting the higher flows into the additional arms. The peak velocities in the distributor arms should not exceed 1.2 m/s.

The distributor arms are generally fabricated of steel and are liable to rapid corrosion. They should be fabricated and bolted together in such lengths as to facilitate dismantling for periodic repainting of their inside surfaces. The orifices in the distributor arms should be of light weight aluminium. Spreader plates, preferably of aluminium, should be provided below the orifices to spread out the discharge. The clearance between the distributor pipe and the top of the reactor media should be greater than 15 cm.

Distributor arms should have gates at the end for flushing them. At least one end plate should have arrangement for a jet impinging on the side wall to flush out fly larvae. The distributor arms may be of constant cross section for small units but in larger units and they are tapered from the centre towards the end to maintain the minimum velocity required in the arms.

The distribution arrangements should ensure uniform distribution of the sewage over the reactor surface for which the size and spacing of the orifices in the distributor arms have to be varied carefully from the centre towards the end. Under average flow conditions, the rate of dosing per unit area at any one point in a reactor should be within 10% of the calculated average dosing rate per unit area for the whole reactor. The distributors should also ensure that the entire surface of the reactor is wetted and no area is left dry.

Reaction type rotary distributors require adequate hydraulic head for operation. The head required is generally 1 to 1.5 m measured from the centre line of distribution arms to the low water level in the distribution well or the siphon dosing tank preceding the reactor. Alternatively, the rotary distributor driven by electric motor may be used and this type is particularly advantageous where adequate head is not available. The rotary speed shall ensure intervals of successive closings are between 15 and 20 seconds.

5.8.2.3.5.8 Pumping Arrangements

In a high rate reactor, pumping is required for recirculation. Pumping may also be required for lifting the reactor effluent to the clarifier or to the next stage reactor. Except in the case of small plants, recirculation pumps should be installed in multiple units so that the rate of recirculation rate can he changed as found necessary. Pumps for lifting the flow-through sewage should have adequate capacity to pump the peak flows through the plant. The pumps should be installed in multiple units to take care of diurnal variations which will approximately be the same as the sewage inflow to the plant. It will further be necessary to provide storage in the suction well equal to about 10 min of discharge capacity of the lowest duty pump. Float control arrangements are desirable in the suction well for controlling the number of pumps in operation. In all the cases, at least one pump should be provided extra as a standby. Furthermore, in the case of recirculation pumps, flow measuring and recording devices are desirable on the discharge line so that a record can be kept of the recirculation flow.

5.8.2.3.5.9 Ponding Problems

Ponding or clogging of the reactor media is one of the important operational problems in these submerged type reactors. Ponding decreases reactor ventilation, reduces the effective volume of the reactor and lowers the reactor efficiency. The ponding or clogging is due to excessive organic loading, inadequate hydraulic loading and inadequate size of media.

The remedies consist of raking or forking the reactor surface, washing the reactor by applying a high pressure stream of treated sewage at the surface, stopping the distributor to allow continuous heavy point by point dosing or chlorinating the influent with a dose not exceeding 5 kg/100 m^2 of reactor area.

Reactor flies pose another serious operational problem. The problem is more intense in the case of low rate reactors and in high rate reactors, fly breeding occurs mainly on the inside walls of the reactor. The problem can be reduced by (a) removing excessive biological growth by the previously discussed methods (b) flooding the reactor for 24 hours at weekly or bi-weekly intervals, (c) jetting down the inside walls of the reactor with a high pressure hose, (d) chlorinating the influent (0.5 to 1.0 mg/l) for several hours at one to two week intervals and (e) applying insecticides. The insecticide should be applied to the reactor side wall surfaces at intervals of 4-6 weeks. Development of resistant strains should be guarded against.

Reactor odour also presents a problem in these reactors. The odours are most serious when treating septic effluents in low rate reactors. They can be controlled by providing recirculation and maintaining a well-ventilated reactor.

In conditions of extreme cold weather, ice cover may form on the surface of the bed. Reduction of the recirculation flow, adjustment of nozzles or construction of wind breakers are methods used to reduce icing problems.

5.8.2.3.5.10 Multiple Units

In a single stage plant, it is advisable to split the required reactor volume into two or more units so that when one reactor is taken out of operation for maintenance or repairs, the entire sewage can be passed through the remaining units, overloading them temporarily. In a two stage plant, if multiple units are proposed in each stage, the entire sewage may be routed through the remaining units of the stage when one reactor in that stage is taken out of operation. However, the recirculation flow is maintained at the original level and operating the stage at a lower recirculation ratio. If, instead, only one reactor is proposed for each stage a bypass should be provided for each stage. It is customary in the design of two stage reactors to use two reactors of equal size.

5.8.2.3.5.11 Plant Hydraulics

The feed pipe to the reactor, the distributor, the under drains and the main collection channel should be designed for the peak instantaneous hydraulic loading on the reactor. In low rate reactors, the peak loading will be the peak discharging capacity of the dosing siphon or the dosing pump. In the case of high rate reactors, the peak loading on the reactors will be the sum of the peak rate of sewage flow and the constant recirculation rate. When multiple units are used for the high rate reactors in any stage, the hydraulics of the plant should be checked for peak loading with one reactor out of operation and the entire flow routed through the remaining units. A reduced recirculation ratio is adopted for this condition to reduce the peak loading and avoid over sizing of the piping.

When multiple units are used care should be taken to ensure that the flow is divided properly between the reactors.

5.8.2.4 Design Guidelines

Arising from the position that each media manufacturer claim their own values of surface area to unit volume of their media, the entire design guidelines being specified as a generic guideline for all these attached growth units becomes a challenge. At the same time, it is necessary not to lose track of the advantages of this technology. The real problem arises in specifying the estimate cost of these while initially preparing the same for according the procedural requirements of DPR and technical sanction for drafting the technical specifications in the tender document.

Until a relative evaluation of the already functioning reactors is completed and a generic set of design guidelines are formulated on a country basis, the design is best left to the tenderer in so far as the unit sizing and associated civil mechanical and piping details are concerned.

Tenders can still be invited and decided based on Section 5.19 titled addressing the recent technologies in choice of STPs.

5.8.3 Treatment Methods using Immobilization Carrier

5.8.3.1 Historical

As an improvement over the attached growth systems, the concept of trapping the microbes into the attached media without allowing them to slough off and keeping the media itself in fluidized state and thus improving on the consistency of the organic removal has been developed. This has been brought under the generic name of immobilized carriers. As otherwise, this technology is also schematically referred to as Moving bed biofilm reactors (MBBR) or Fluidized aerobic bed (FAB). The main reason for this technology to be attractive is its ability to reduce the waste sludge volumes.

5.8.3.2 Specific Microbiology

It has been shown that maintaining high biomass concentration and long solids retention time in a biological reactor can limit the waste sludge production for a given reduction of BOD. This is due to the higher biomass concentration in the reactor due to the immobilized biomass and hence the Food/Microorganism ratio going beyond the extended aeration. It is stated that during aeration, the synthesis and accumulation of readily biodegradable storage compounds are observed and these can be used for denitrification under starvation conditions.

5.8.3.3 Status

Enhancing active biomass concentration, prolonging the life of immobilized carrier and improving the stability of immobilized microorganism play important roles in the process efficiency. The construction, operation, preventing clogging and reducing renewal costs are challenges in the commercial engineering of this technology. However the fact remains that there are commercially operating STPs built with this technology in our country using various patented media of the respective vendors and with their own design criteria. As such, this technology holds the potential of reducing the footprint of the STP especially in land locked high density urban centres and thus merits its relative consideration.

5.8.3.4 Physical Features

5.8.3.4.1 Shape of Reactors

Reactors are usually like aeration tanks and the circular shapes find better acceptance due to reduced civil construction costs and also permitting tall structures as long as air cooled compressors can be installed to pump the air against such high heads of water column.

5.8.3.4.2 Floor

The floor shall permit installation of fixed air headers for fixing diffuser elements thereon inside the reactor and provision for gate controlled draining of the reactor.

5.8.3.4.3 Reactor Media

The requirements for reactor media are high specific surface area, high percent void space, resistance to abrasion or disintegration during placement, insolubility in sewage and resistance to spalling and flaking. The inbuilt configuration must permit hydraulic self-cleaning of the media itself and thereby safeguarding the need to take the reactor out of service to attend to cleaning the clogged media.

5.8.3.4.4 Netting to Hold Back Media

This is an important requirement and is usually provided near the top outlet of the treated sewage in the form of spread out netting across the entire plan area or a netted cowl around the off take of the inlet pipe. Care is needed periodically to renew these.

5.8.3.4.5 Auxiliary Mixers in the Reactor

The specific gravity of the media shall have to permit its floatation but at the same time the required value for keeping it submerged. It will be advantageous to install side wall mounted slow moving propeller blade mixers which can plummet the media downwards and thereby they can rise back and again be plummeted downward and thus ensure optimum contact between all media and the sewage.

This will prevent chances of microbes building up on the floor due to lack of transport velocities.

5.8.3.4.6 Multiple Units

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.3.4.7 Plant Hydraulics

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.3.5 Design Guidelines

The same guidelines as in section 5.8.2 on attached growth systems apply here also.

5.8.4 Stabilization Ponds

5.8.4.1 General

Stabilization ponds are open, flow-through earthen basins designed and constructed to treat sewage and provide comparatively long detention periods extending from a few to several days. During this period the organic matter in sewage is stabilized in the pond through a symbiotic relationship as illustrated in Figure 5.3 earlier. Lightly loaded ponds are also used as a tertiary step in sewage treatment for polishing of secondary effluents and destruction of coliform organisms and are called maturation ponds. In warm climate countries, the pond systems are cheaper to construct and operate compared to conventional methods. They also do not require skilled operational staff and their performance does not fluctuate from day to day. The only disadvantage of pond systems is the relatively large land that they require, but this is sometimes over-emphasized. In addition, land on the outskirts of a growing city can be a worthwhile investment. Pond systems must be considered as an alternative when treatment of sewage or upgrading of existing facilities are planned and the life time costs of various other treatment system should be calculated and compared.

5.8.4.2 Classification

5.8.4.2.1 Aerobic

Aerobic ponds are designed to maintain completely aerobic conditions. The ponds are kept shallow with depth less than 0.5 m and BOD loadings are 40 to120 kg/ha.d. The pond contents may be periodically mixed by float mounted paddle mixers. Ponds like these give rise to intense algal growth and have been used only on experimental basis

5.8.4.2.2 Anaerobic

Completely anaerobic ponds are used as pretreatment sometimes for municipal sewage. They are also used for digestion of STP sludge. Depending on temperature and waste characteristics, BOD load of 400 to 3000 kg/ha.d and 5 to 50 day detention period would result in 50 to 85% BOD reduction. Such ponds are constructed with a depth of 2.5 to 5 m to conserve heat and reduce land area. They have an odour problem due to sulphide gases.

5.8.4.2.3 Facultative

The facultative pond functions aerobically at the surface while anaerobic conditions prevail at the bottom. The aerobic upper layer oxidizes the sulphide gases and avoid the foul odours. The treatment effected is comparable to that of conventional secondary treatment processes. The facultative pond is suited and commonly used and further discussion in this chapter is therefore, confined to facultative ponds.

5.8.4.3 Mechanism of Purification

The physical, chemical and biological reactions in engineered pond systems are controlled by the design criteria. The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown schematically in Figure 5-3.

Sewage organics are stabilized by both aerobic and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter in the incoming sewage is oxidized to carbon dioxide and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate to upper layers, are also oxidized aerobically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved and suspended organics in the bottom layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles. For each kg of BOD-ultimate stabilized in this manner, 0.25 kg or 0.35 m^3 of methane is formed. Another reaction which sometimes occurs in the anaerobic layers is conversion of hydrogen sulphide to sulphur by photo-synthetic bacteria; if present in sufficient numbers they give a distinct pink hue to the pond appearance.

5.8.4.3.1 Aerobic and Anaerobic Reactions

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Oxygen diffusing from top layers is utilized quickly and completely. Further, there is a decrease in the photo-synthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane bacteria. Production of methane is fundamental to BOD reduction by anaerobic metabolism. If the second step does not proceed satisfactorily there is an accumulation of organic acids in the pond bottom which diffuses towards the top layers. Furthermore, under such conditions the pH of the bottom layers may go down. This would result in complete inhibition of methane bacteria and the pond may turn completely anaerobic due to accumulation of end products of partial anaerobic decomposition, Imbalance between the activities of the two sets of microorganisms in a pond may result from two possible reasons. The waste may contain inhibitory substances which would retard the activity of methane producing organisms and not affect the activity of acid producers to the same extent. In treatment of sewage such a condition, however, does not arise. The other reason for the imbalance may be a fall in the temperature of the pond. The activity of methane bacteria decreases much more rapidly with decreasing temperature as compared to the acid formers and gas production stops at temperatures lower than 15ºC. Thus, year round warm temperatures and sunshine provide an ideal environment for operation of the facultative stabilization ponds.

5.8.4.3.2 Diurnal Variations

Both the dissolved oxygen and pH of the pond are subject to diurnal variation due to photosynthetic activity of algae which is related to incident solar radiation. A high dissolved oxygen concentration up to about 4 times the saturation value may be observed in the afternoon hours. Simultaneously, the pH value may reach a maximum of 9.0 or more due to the conversion of carbon dioxide to oxygen. Towards the evening or in the night, when photosynthetic activity decreases or ceases, there is a gradual decrease in both dissolved oxygen and pH.

In properly designed ponds, the dissolved oxygen does not completely disappear from the top layers at any time. The increase of pH is beneficial as it increases the die off rate of faecal bacteria like coliforms.

5.8.4.3.3 Odour Control

In a facultative pond, the nuisance associated with anaerobic reactions is eliminated due to the presence of oxygen in the top layers. The foul smelling end products of anaerobic degradation which permeate to the top layers are oxidized in an aerobic environment. Furthermore, due to a high pH in top layers, compounds such as organic acids and hydrogen sulphide, which would otherwise volatilize from the surface of the pond and cause odour problems are ionized and held back in solution.

5.8.4.3.4 Algae

In stabilization ponds, the significant algae are green which include Chlorella, Scenedesumus, Hydrodictyon Chlamydomonas and Ankistrodesmus and blue-green algae which include Oscillatoria, Spirulina, Merismopedia and Anacystis. The Chlorella, Scendesmus and Hydrodictyon possess relatively high oxygen donation capacity per unit weight. However, it is not practical to promote the growth of any particular type of algae in a pond which will depend on such factors as temperature, characteristics of the waste and intensity of sunlight. Concentration of algae in a stabilization pond is usually in the range of 100 to 200 mg/l which gives the pond effluent a typical green colour. Floating blue-green algae mats may develop in ponds during summer months. They are undesirable since they restrict penetration of sunlight leading to reduction in depth of aerobic layer. They also encourage insect breeding.

5.8.4.4 Design Considerations

The facultative pond system, though simple to operate, is a complex ecosystem. It is only by experience and understanding of the reactions that rational criteria are evolved. Appendix A.5.14 presents an illustrative design.

5.8.4.4.1 Areal Organic Loading

The permissible areal organic loading for the pond expressed as kg BOD5/ha.d will depend on the minimum incidence of sunlight that can be expected at a location and on the percentage of the influent BOD that would have to be satisfied aerobically. Many different methods have been developed for determining the permissible area loading and two methods are discussed here, being

(a) The BIS has related the permissible loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free (Refer IS: 5611) and

(b) another based on field experience. The recommended loading rates are in Table 5.14 overleaf.

The values are applicable to towns at sea levels and locales where the sky is clear for nearly 75% of the days in a year.

Table 5.14 Permissible Organic Loadings at Different Latitudes

Latitude (N) degree	Organic loading Kg BOD/ha.d
36	150
32	175
28	200
24	225
20	250
16	275
12	300
8	325

Source: CPHEEO, 1993

The values of organic loading given in Table 5.14 may be modified for elevations above sea level by dividing by a factor of (1+0.003 EL) where EL is the elevation of the pond site above MSL in hundred meters. An increase in the pond area has to be made when the sky is clear for less than 75% of the days. For every 10% decrease in the sky clearance factor below 75%, the pond area may be increased by 3%.

Another design approach, based on field experience in warm climates relates the permissible area BOD loading to the ambient temperature on the assumption that temperature would depend on solar radiation:

$$L_0 = 20T - 120 \tag{5.36}$$

where

L_o = design organic load in kg BOD_5/ha/d and
T = average temperature during coldest month of the year in degree Celsius.

The designs based on the two methods given above, as well as other methods developed empirically, wherever possible should be checked against field experience in the region. When the ponds are intended to serve small communities or when they are located close to residences, it will be prudent to adopt lesser BOD loading to fully ensure absence of odours.

5.8.4.4.2 Detention Time and Hydraulic Flow Regimes

The flow of sewage through a pond can approximate either plug flow or complete mixing, which are two extreme or ideal conditions. If BOD exertion is described by a first order reaction, the pond efficiency is given by the equation in the next page:

For Plug flow

$$\frac{L_e}{L_i} = e^{-K_l t} \qquad (5.37)$$

For Complete Mixing

$$\frac{L_e}{L_i} = \frac{1}{1 + K_l t} \qquad (5.38)$$

where

L_i and L_e = influent and effluent BOD respectively,
t = detention time,
K_l = BOD reaction rate constant.

The value of K_l varies between 0.05 and 0.2 per day and is independent of temperatures above 15ºC. The lower values were determined for secondary and tertiary ponds.

In practice the hydraulics lies between the two regimes and is described as dispersed flow. The efficiency of treatment for different degrees of intermixing, characterized by dispersion numbers, can be determined as given in Section 5.8.1.7.12 for aerated lagoons. Dispersion numbers are determined by tracer studies. Dispersion numbers for stabilization ponds vary from 0.3 to 1.0. Choice of a larger value for dispersion number or assumption of complete mixing would give a conservative design and is recommended.

5.8.4.4.3 Depth

Shallow depths in facultative ponds will allow the growth of aquatic weeds in the ponds. The optimum range of depth for facultative ponds is 1.0 - 1.5 m. When depth determined from area and detention period works out lesser than 1.0 m, the depth should be increased to 1.0 m, keeping surface area unchanged.

5.8.4.4.4 Sludge Accumulation

The rate of sludge accumulation in facultative ponds depends primarily on the suspended solids concentration in the sewage. It varies from 0.05 to 0.10 m^3/capita/year. A value of 0.07 m^3/capita/year forms a reasonable assumption in design. In multiple cell ponds operated in series, most of the sludge accumulation will be in the primary cells. Continued sludge accumulation in ponds over many years will cause (i) sludge carryover into the effluent, (ii) development of aquatic weeds, and (iii) reduction in pond efficiency due to reduction in the detention period, Facultative ponds therefore require periodical desludging at intervals ranging from 6 to 12 years.

5.8.4.4.5 Bacterial Reduction

Bacterial reduction in ponds is similar to BOD reduction except the BOD reduction rate constant is replaced by bacterial die off constant, Kb and inputs and outputs are in terms of bacterial concentrations Ni and Ne, respectively.

It is customary to use completely mixed conditions when calculating bacterial reduction. This gives a conservative design. Overall bacterial reduction in 'n' ponds of equal detention time 't' in series is given by

$$\frac{N_e}{N_i} = \frac{1}{(1+K_b t)^n} \tag{5.39}$$

A commonly used value of K_b for faecal bacteria at 20°C is 2.0 per day. The value of K_b at other temperatures may be calculated by

$$K_{b(T)} = K_{b(20)}(1.19)^{(T-20)} \tag{5.40}$$

where,

$K_{b(T)}$ and $K_{b(20)}$ are values of the constant at *T* and at 20°C respectively.

5.8.4.4.6 Mosquito Aspects

It is popularly believed that ponds will promote the growth of mosquitoes. This is not true. The stages of growth of mosquitoes are shown in Figure 5.43.

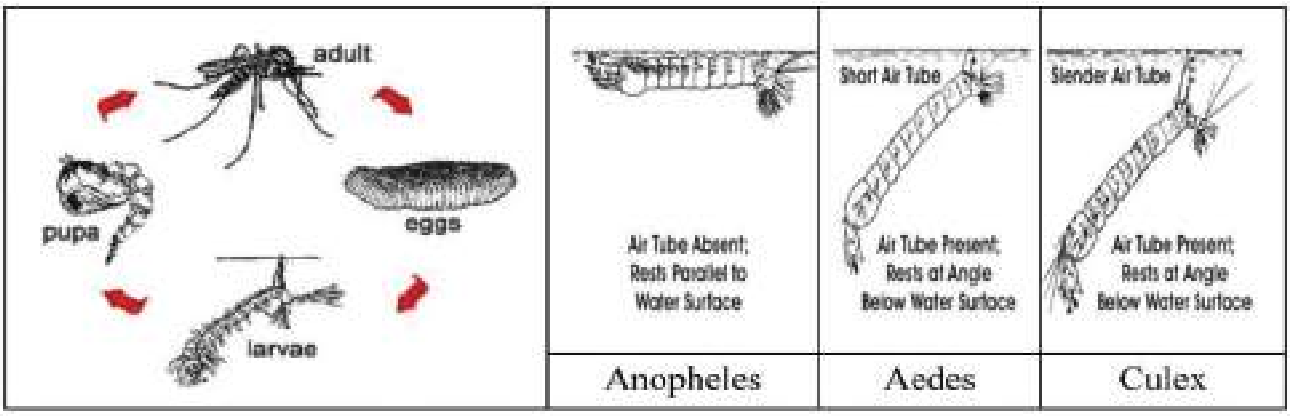


Figure 5.43 Stages of Mosquito growth and Larvae Stage Breathing near Water Surface

It may be seen that the mosquito eggs becomes larvae and they must breathe air to survive. This is why the larvae generally remain just beneath the surface of the water. There are three broad types of mosquito larvae like this. These are Anopheles, Aedes and Culex. The Anopheles lay parallel to the water's surface in order to get a supply of oxygen through a breathing opening. The other two types have siphon tubes and lay beneath the water surface and the siphon pipe punctures the water and protrudes into the air. After some days in the larval stage they grow to pupa and then adults. In a well operated pond free of any weeds, the wind causes the water surface to be continuously oscillating non-stop. Due to this, raw sewage gets into the breathing system of the larvae and chokes them. Thus the ponds do not promote mosquitoes. Hence, the pond shall be free of weeds which will otherwise prevent the oscillation of the water surface.

5.8.4.5 Construction Details

5.8.4.5.1 Site Selection

Facultative pond sites should be located as far away as practicable (at least 200 m) from habitations or from any area likely to be built up within a reasonable future period. If practicable the pond should be located such that the direction of prevailing wind is towards uninhabited areas. The pond location should be downhill of ground water supply source to avoid their chemical or bacterial pollution. Special attention is required in this regard and in porous soils and in fissured rock formations. The pond site should not be liable to flooding and the elevation of the site should permit the pond to discharge the effluent by gravity to the receiving streams. The site should preferably allow an unobstructed sweep of wind across the pond and open to the sun. Trees should not be grown in the bunds and for an annular distance of 10 m from the toe of the bunds. Advantages should be taken of natural depressions while locating the ponds.

5.8.4.5.2 Pre-treatment

Medium screens and grit removal devices shall be provided before facultative ponds.

5.8.4.5.3 Construction in Stages

In cases where the design flow will occur only after a long time, it is important to design facultative ponds in multiple cells and construct the cells in stages. Otherwise, the small flows in the initial years may not be able to maintain satisfactory water levels in the ponds. This will cause objectionable weed growths. The weeds will prevent the water surface from oscillation by wind. Hence mosquitoes can breed and multiply. Construction in stages will also reduce initial costs and help in planning future stages based on the performance data of the first stage.

5.8.4.5.4 Multiple Units

Multiple cells are recommended for all except small installations (0.5 ha or less). Multiple cells in parallel facilitate maintenance as any one unit can be taken out of operation temporarily for desludging or repairs without upsetting the entire treatment process. The parallel system also provides better distribution of settled solids. Multiple cells in series decrease dispersion number and enable better BOD and coliform removal and reduced algal concentration in the effluent. The series system implies a high BOD loading in the primary cells and to avoid anaerobic conditions in these cells, they should have 65% to 70% of the total surface area requirements. A parallel series system possesses the advantages of both parallel and series operations. A convenient arrangement for this system consists of three cells of equal area, of which two are in parallel and serve as primary ponds and the third serves as secondary pond in series. Individual cell should not exceed 20 ha in area.

5.8.4.5.5 Pond Shape

The shape should be such that there are no narrow or elongated portions. Rectangular ponds with length not exceeding three times the width are to be preferred. Maximum basin length of 750 m is generally adopted. The comers should always be rounded to minimize accumulations of floating matter and to avoid dead pockets.

5.8.4.5.6 Embankment

Ponds are usually constructed partly in excavation and partly in embankment. The volume of cutting and the volume of embankment should be balanced to the maximum extent possible in order to economize construction costs. Embankment materials usually consist of material excavated from the pond site. The material should be fairly impervious and free of vegetation and debris. The embankment should be compacted sufficiently. The top, width of the embankment should be at least 1.5 m to facilitate inspection and maintenance. The free board should be at least 0.5 m in ponds less than 0.5 ha in area. In larger installations, the free board should be designed for the probable wave heights and should be at least 1.0 m. Embankment slopes should be designed based on the nature of soil, height of embankment and protection proposed against erosion. Outer slopes are generally 2.0 to 2.5 horizontal to 1 vertical. Inner slopes are made 1.0 to 1.5 when the face is fully pitched and flatter and 2.0 to 3.0, when the face is unprotected. Inner slopes should not exceed 4 as flatter slopes create shallow areas conducive to the growth of aquatic weeds. The outer faces of the embankments should be protected against erosion by turfing. The inner faces should preferably be completely pitched to eliminate problems of erosion and growth of marginal vegetation. Pitching may be by rough stone revetment or with plain concrete slabs or flat stones with adequate gravel backing. When complete pitching is not possible, at least partial pitching from a height 0.3 m above water line to 0.3 m below water line is necessary and the face above the line of pitching should be turfed to the top of embankment. A properly constructed pond is shown in Figure 8.10 in this manual.

5.8.4.5.7 Pond Bottom

The pond bottom should be level, with finished elevations not more than 0.10 m from the average elevation. The bottom should be cleared of all vegetation and debris. The soil formation of the bottom should be relatively impervious to avoid excessive liquid losses due to seepage. Where the soil is loose, it should be well compacted. Gravel and fractured rock areas must be avoided.

5.8.4.5.8 Pond Inlets

The pipeline conveying raw sewage to the pond, whether by gravity or by pumping, should be terminated in a flow measuring chamber located close to the pond. There should be sufficient fall from the measuring chamber to the pond surface so that the measuring weir may not be submerged. If the pond installation is in multiple parallel cells, the measuring chamber should have flow splitting provision and there should be separate pipeline to each cell. The size of the pipeline may be designed to maintain an average velocity of 0.3 m/s. The pipeline should be semi-flexible and should be properly supported inside the pond. In case the pond cell is large, multiple inlets should be provided along the inlet side of the pond at the rate of one for every 0.5 to 1.0 hectare of pond area. This requirement applies also to outlets. In case the pond is small, a single inlet and a single outlet will be sufficient. The inlets in the pond shall be so located as to avoid short-circuiting of flow to the outlets. The inlets should not be upwind of the outlets and should be extended into the pond for one-third to one-fourth the pond length or 15 to 20 m, whichever is less. The discharge may be horizontal and at half depth. A concrete apron of adequate size should be provided under the discharge to prevent erosion of pond bottom, especially when the pond is being filled up.

5.8.4.5.9 Pond Outlets

Multiple outlets are desirable except in small ponds and may be provided at the same rate as for inlets, one for every 0.5 ha pond area. The outlets should be so located with reference to the inlets as to avoid short-circuiting. The outlet, structures may consist either of pipes projecting into the ponds or weir boxes. In the former case vertical tees and in the latter case hanging baffles submerged to a depth of 0.25 m below the wafer surface should be provided to ensure that floating algal scum is not drawn along with the effluent. When the outlet structure is a weir box, it is desirable to provide adjustable weir plates so that the operating depth in the pond can be altered if required. Where the pond effluent is to be used for farming and involves pumping, the outlet pipe should be led to a sump of adequate capacity (30 minutes at the rate of pumping). All piping should be provided with suitable valves to facilitate operation and maintenance.

5.8.4.5.10 Pond Interconnections

Pond interconnections are required when ponds are designed in multiple cells in series. These interconnections should be such that the effluent from one cell withdrawn from the aerobic zone can he introduced at the bottom of the next cell. Simple interconnections may be formed by pipes laid through the separating embankments. At their upstream ends, the interconnecting pipes should be submerged about 0.25 m below the water level. The downstream ends may be provided with a bend, facing downward, to avoid short-circuiting by thermal stratification, care being taken to prevent erosion of the embankment.

5.8.4.5.11 Other Aspects

Provision should be made for flow measurement both at inlet and outlet of the ponds, wherever practicable, facilities should be available to drain out the pond completely by gravity through a sluice arrangement. The pond site should be fenced to prevent entry of cattle and discourage trespassing. Public warning boards should also be put up near the ponds clearly indicating that the pond is a sewage treatment facility.

5.8.4.6 Performance

The algae in the pond effluent will exert BOD in the standard laboratory BOD test involving darkroom incubation and will give high SS values. The BOD and SS values may each be in the range of 50 to 100 mg/l. However, the effluent will not cause nuisance when disposed of on land or discharged into receiving waters because the algal cells do not readily decompose or exert oxygen demand under natural conditions, in fact, the algae increases the oxygen levels in the receiving water by continued photosynthesis.

Because of the above reasons, the standard BOD and SS tests are not considered useful for evaluating the quality of facultative pond effluents.

The quality is usually assessed based on the BOD_5 of the filtered effluent, the assumption being that the suspended solids in the effluent are all algae. The filtration procedure adopted for the test is the same as for the suspended solids test.

Well designed facultative ponds give about 80% to 90% BOD reduction based on the filtered BOD_5 of the effluent. Facultative ponds also effect high bacterial reduction, the efficiency being particularly high in multi cell ponds operated in series. Coliform and faecal streptococci removals are as high as 99.99%. Intestinal pathogens belonging to Salmonella and Shigella groups are reportedly eliminated in stabilization ponds. Cysts of Entamoeba Histolytica and Helminthic larvae are also eliminated.

5.8.4.7 Construction for Filtering Out Algae

The algae flowing out of the pond need not be removed when the treated sewage is used for crop irrigation. The most appropriate technique for this is a rock filter, which consists of a submerged porous rock bed within which algae settle out as the effluent flows through. The algae decompose releasing nutrients which are utilized by bacteria growing on the surface of the rocks. In addition to algal removal, significant ammonia removal may also take place through the activity of nitrifying bacteria growing on the surface of the filter medium. The performance depends on loading rate, temperature and rock size and shape. The permissible loading increases with temperature and in general an application rate of 1.0 m^3 of pond effluent per m^3 rock bed per day should be used. Rock size is important, as surface area for microbial film formation increases with decreasing rock size but, if the rocks are too small, then problems can occur with clogging. Rock size is normally 75 to 100 mm, with a bed depth of 1.5 to 2.0 m. A typical rock filter is shown in Figure 5.44.

Source. Duncan Mara

Figure 5.44 Rock Filter Installed in the Corner of a Pond at Veneta, Oregon, USA

The effluent should be introduced just below the surface layer because odour problems are sometimes encountered with cyanobacterial films developing on wet surface rocks exposed to the light. Construction costs are low and very little maintenance is required, although periodic cleaning to remove accumulated humus is necessary, but this can be carried out during the cooler months when algal concentrations are lowest.

5.8.4.8 Applications

The facultative pond is simple and economical to construct. It does not require skilled operation and is easy to maintain. Properly designed, the pond also gives consistently good performance. The facultative pond has therefore become very popular for sewage treatment. The method is suited wherever land is cheap and readily available and may be used for treating sewage either for discharge into streams or lakes or for use on land. The method is particularly useful for interim sewage treatment when due to lack of funds or due to meagre flow in the initial stages, it is considered inexpedient to construct initially the treatment plant envisaged ultimately. Their performance in terms of pathogen removal and reliability is high. The treated sewage can be used for agriculture in conformity with the quality stated in Table 7.19. In regard to pisciculture pl refer to section 4.15 of Part B manual.

5.8.5 Anaerobic Treatment of Sewage

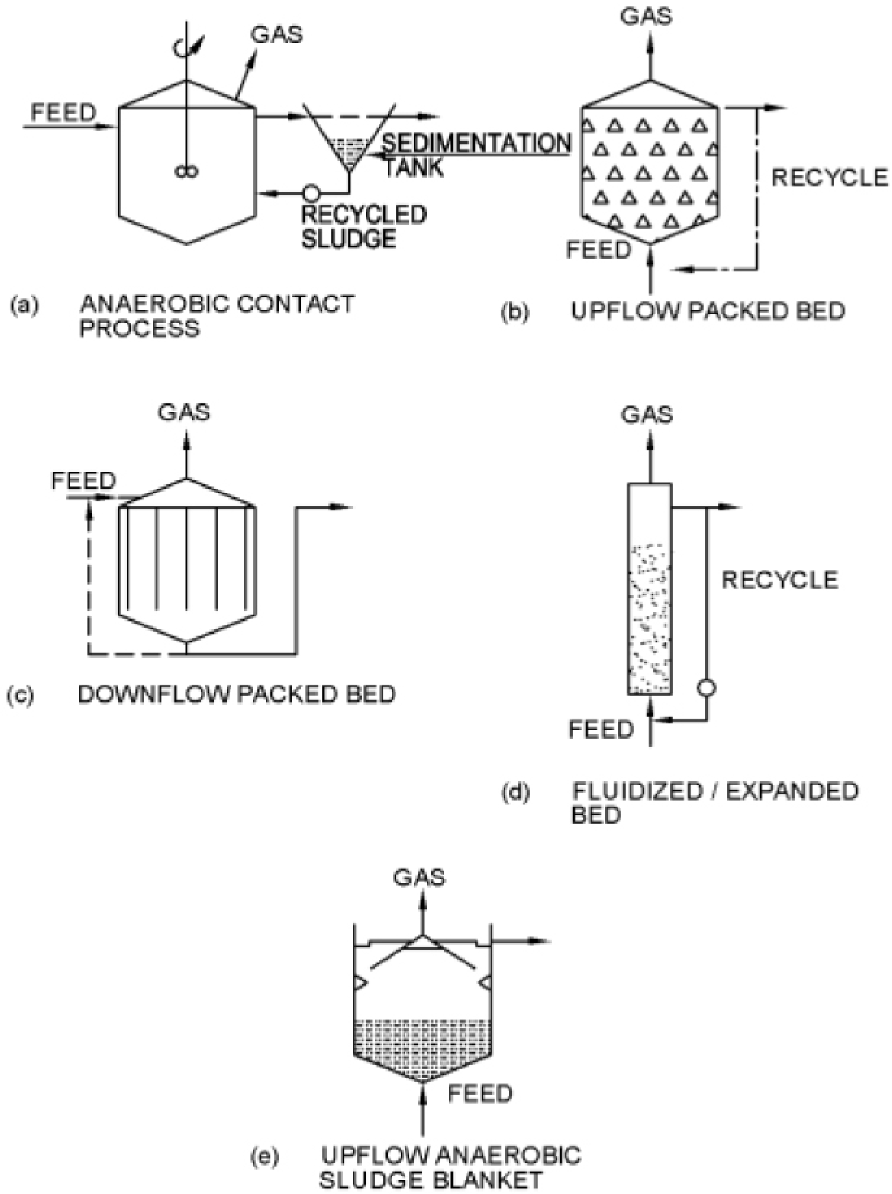
5.8.5.1 Introduction

Anaerobic treatment of sewage has a number of advantages over aerobic treatment processes. These are (a) lesser electrical energy input of the system because oxygen is not required and hence, aeration is not needed and (b) Methane gas, which has a thermal value is produced from which electrical energy can be generated, However, there is a disadvantage in that the corrosive Hydrogen Sulphide gas is produced from the sulphate present in the sewage. Anaerobic digestion as a unit process in municipal sewage treatment has been in use since the beginning of this century. It is employed for stabilization of sludge solids from primary and secondary sedimentation tanks either in closed digesters or open lagoons. Anaerobic lagoons are also used for treatment of industrial wastes. Conventionally, the anaerobic process is considered a slow process requiring digesters of large hydraulic retention time (HRT). In recent years a number of high rate systems have been constructed to treat concentrated liquid industrial wastes and for direct treatment of municipal sewage. Application of anaerobic treatment technology, for treatment of municipal sewage has special significance in India because of high-energy savings and low capital and O&M and renewal and replacement costs. This section briefly reviews various high rate systems and summarizes the available design criteria. It also lists aspects of anaerobic treatment, which must be evaluated in the designs.

5.8.5.2 High Rate Anaerobic Systems

High rates of conversion of organics into methane can be obtained by maintaining a high concentration of microorganisms in a reactor and preventing them from escaping with the effluent. This concept is expressed as Sludge Retention Time (SRT) and is the ratio of mass of biological solids in the system to that escaping from the reactor. Maximal SRT is desirable for process stability, minimal sludge production and minimal reactor volume and thus reduces capital costs. Other requirements of high rate systems are intimate contact between incoming waste and the biological solids and maintenance of sufficiently warm temperatures.

Figure 5.45 (overleaf) shows the basic configurations of high rate anaerobic systems.



Source: CPHEEO, 1993

Figure 5.45 Configurations of High Rate Anaerobic Systems

5.8.5.2.1 Anaerobic Contact Process

The Anaerobic Contact process as in Figure 5.45 (a) is a stirred tank reactor in which the biomass leaving with the reactor effluent is settled in a sedimentation tank and recycled, thus increasing the SRT. The settling of the anaerobic sludge may at times be a limiting factor. Biomass separation may be improved using parallel plate separators. The process lends itself to concentrated wastes containing refractory suspended matter. Continuous and complete mixing in the reactor is not recommended, since this may adversely affect settling characteristics of the sludge, On the other hand, inadequate mixing may result in formation of dead zones inside the reactor. This process has been used for treatment of industrial wastewaters.

5.8.5.2.2 Anaerobic Filter

The Anaerobic Filter as in Figure 5.45 (b) is a tank in which microbial cells are both entrapped as clumps of cells in the interstices between packing material and as biofilm attached to the surface of the packing material. The packing or filter media is usually of naturally crushed rock of 15 mm to 25 mm size or consisting of plastic or ceramic material. The filter media should have high specific surface and porosity to allow for maximum possible film growth and retention of biomass. The reactor is operated as up flow submerged packed bed reactor. A number of such filters have been constructed for treatment of low strength wastes such as municipal sewage.

5.8.5.2.3 Anaerobic Fixed Films Reactor

The Anaerobic Fixed Film Reactor as in Figure 5.45 (c) is a tank in which the microbial mass is immobilized on fixed surfaces in the reactor. It is operated in down flow mode to prevent accumulation of refractory particulates contained in the influent and sloughed biofilm. The sloughed biofilm is also discharged with the effluent. The reactor may be operated in either submerged or non-submerged condition. The reactor packing is usually of modular construction consisting of plastic sheets providing a high void ratio. Such reactors have been constructed to treat high strength wastes.

5.8.5.2.4 Fluidized and Expanded Bed Reactor

The Fluidized Bed reactor as in Figure 5.45 (d) is a tank which incorporates an up flow reactor partly filled with sand or a low density carrier such as coal or plastic beads. A very large surface area is provided by the carrier material for growth of biofilm. The system readily allows passage of particulates, which could plug a packed bed, but requires energy for fluidization. Expanded Bed (EB) reactors do not aim at complete fluidization and use a lower up flow velocity resulting in lesser energy requirement. These reactors can be used for treatment of municipal sewage as well.

5.8.5.2.5 Up flow Anaerobic Sludge Blanket Reactor

The Up flow Anaerobic Sludge Blanket Reactor (UASB), Figure 5.45 (e), maintains a high concentration of biomass through formation of highly settleable microbial aggregates. The sewage flows upwards through a layer of sludge. At the top of the reactor phase, separation between gas-solid-liquid takes place. Any biomass leaving the reaction zone is directly recirculated from the settling zone. The process is suitable for both soluble wastes and those containing particulate matter. The process has been used for treatment of municipal sewage at few locations and hence limited performance data and experience is available presently.

5.8.5.3 Design and Operational Considerations

Appendix A.5.15 presents an illustrative design of Upflow Anaerobic Sludge Blanket Reactor.

Appendix A.5.16 presents an illustrative design of Anaerobic Filter.

5.8.5.3.1 Organic Load and Sludge Retention Time

It is customary to express the organic matter in sewage in terms of Biochemical Oxygen Demand (BOD) or Chemical Oxygen Demand (COD). In anaerobic treatment systems, the COD value is finding greater usage, which lends itself directly to mass balance calculations. Reduction in COD for municipal sewage would normally correspond to equivalent amount of ultimate BOD reduction. Table 5.15 summarizes volumetric organic loads used in some of reactors for municipal sewage.

Table 5.15 Organic Loadings and Performance Efficiencies of Some High Rate Anaerobic Reactors

Reactor Type	Organic Load kg COD/m^3d	Efficiency %
AF	0.3 - 1.2	65 - 75
UASB	1.0 - 2.0	50 - 70

Source: CPHEEO, 1993

SRT, which is a more rational design parameter, is difficult to calculate for anaerobic reactors. For anaerobic contact and UASB, it ranges between 15 to 30 days while for other systems it is estimated to be about 100 days or more, giving them greater operational stability.

5.8.5.3.2 Hydraulic Load

For dilute wastes, the minimum HRT at average flow may be 6 to 12 hours for wastes containing suspended organic matter. In UASB reactors where a settling zone is provided, the average hydraulic over flow rate should not exceed 1 m/hour for flocculent sludge and 3 m/hour for granular sludge. The velocity through port between reaction zone and settling zone for the two types of sludge should not exceed 3 and 12 m/hour respectively. The face velocities depend on the characteristics of the media used.

5.8.5.3.3 Effect of Temperature

Activity of methanogenic bacteria is strongly influenced by temperature. It approximately doubles for every 10°C rise in temperature in the range of 18 to 38°C. However, high micro-organism concentration in high rate anaerobic reactor compensates the decreased activities of the anaerobic organisms at lower temperature.

5.8.5.3.4 Excess Sludge Production and Nutrient Requirement

In UASB treatment systems directly treating municipal sewage, the sludge production is reported to be 0.1 to 0.2 kg dry matter/m^3 sewage treated or 0.4 to 0.7 kg dry matter / kg BOD removed, and these include both inert matters present in raw sewage and end products of biological synthesis. The sludge production due to microbial synthesis from anaerobic systems is of the order of 0.01 to 0.1 kg VSS/kg COD removed. The lower values are for systems maintaining high SRT values. Consequently, the requirement of nitrogen and phosphorus is also low. In addition to nitrogen and phosphorus, methanogenic bacteria also require iron, cobalt, nickel and sulphide. These elements are usually present in sewage, but may have to be added if needed.

5.8.5.3.5 Toxicity

Anaerobic bacteria like most micro-organisms can be acclimated to different levels of various toxicants. However, because of their slow growth rate the acclimatization period may be comparatively longer. The sewage characteristics should be evaluated for their toxic effects before anaerobic treatment is adopted.

5.8.5.3.6 Recirculation

Recirculation may be practiced for dilution of incoming waste organic matter and/or biodegradable toxicants; it also provides flow for fluidization in case of FB/EB reactors. In case of municipal sewage, no recirculation is required except for fluidization in FB/EB reactors.

5.8.5.3.7 Gas Yield and Utilization

Methane production can be directly related to degree of treatment based on of COD value of methane produced and COD reduction. Theoretically, 0.35 m^3 methane is produced per kg COD reduction. Biogas normally contains 65% to 70% methane and 30% to 35% carbon dioxide. Since for low strength wastes there is considerable throughput of liquid in a high rate anaerobic treatment system, the gases also escape from the system with the effluent in soluble form. For municipal sewage, therefore, only 0.15 to 0.2 m^3 methane/kg COD removed may be recovered. Further, because of considerably higher solubility of carbon dioxide in comparison to methane, the off gas is enriched in its methane content to about 90%.

The generation of biogas is considered an asset of anaerobic sewage treatment. It is true for anaerobic digestion of sludge and strong industrial wastes where large amounts of gas may be generated. However, in the case of “weak” municipal sewage the recovery is less. Furthermore, for financial viability there should be an opportunity for utilization of the gas. Direct use of biogas in boiler houses in industries, utilization in institutions or in households is a more attractive option compared to generation of electricity which requires greater initial investment and operational and maintenance cost.

5.8.5.4 Pre-Treatment

Screening and grit removal are commonly used pre-treatment unit operations before direct anaerobic treatment.

5.8.5.5 Effluent Quality and Post Treatment

In the case of treatment of municipal sewage, the effluent BOD can be expected to be about 50 mg/l assuming influent BOD of 200 mg/l. For concentrated wastes, the BOD concentration would be higher. Depending on the situation, one or more of the following post-treatment operations may be considered:

i) Holding pond of one day detention time followed by Fish pond/aqua culture pond

ii) Aerobic treatment (aerated lagoons, oxidation pond, etc.)

iii) Tertiary chemical treatment processes

5.8.5.6 Choice of Process

In general, anaerobic treatment does not produce the treated sewage quality fit for discharge for any of the receiving environments and invariably a downstream aerobic treatment is needed. Further, the generation of foul odours as sulphide and potential methane which can ignite and are relevant. These are the experiences with UASB plants adapted for sewage in India have also not been greatly enthusing. As such, the following recommendations are made out.

a) Where facultative ponds are proposed, anaerobic ponds may be used as pre-treatment to reduce the land area of the downstream facultative pond and if feasible the methane gas can be collected by a synthetic gas dome and used if necessary after sulphide stripping.

b) The use of UASB shall be discontinued gradually, over a period of time.

5.8.6 Supplemental Treatment Processes

5.8.6.1 Historical

Historically, the general objective of sewage treatment has been to achieve a BOD of 20 mg/l and SS of 30 mg/l. With the passage of time, the need to ensure against waterborne pathogen removal and against eutrophication, the recent STP tenders brought in the additional stipulations to control the eutrophication of receiving waters and ensuring better removal of waterborne pathogenic organisms in treated sewage, besides reducing the oil and grease limit to 5 mg/l. These requirements have necessitated augmentations of conventional secondary treatment and technologies beyond the secondary treatment and referred to as the tertiary treatment. The technologies are dealt with below.

5.8.6.2 Tertiary Treatment Technologies

By definition, these are removal of constituents beyond the ability of secondary treatment. These are Chemical Precipitation and Membrane Technologies.

5.8.6.2.1 Chemical Precipitation

This is required to remove the phosphorous for control of eutrophication in receiving waters, salts if the treated sewage is to be used for industrial purposes and heavy metals. The technology part alone is dealt with here. The guidelines for these are dealt with in Chapter 7. The precipitation reactions in water are shown in Table 5.16 (overleaf) for an illustrative composition in water.

If we refer to the Table 5.16 there are five rows for each chemical to be removed. For example, if we consider the removal of magnesium sulphate, the first row names the chemical being considered, the second row presents the chemical reaction formula, the third row shows the stoichiometry, the fourth row shows the actual quantity of the chemical and the quantity of the reactant needed and the fifth row verifies the stoichiometry of the actual reaction.

The last rows sum up the weight of chemicals added and the weight of chemicals precipitated. These are theoretical equations and in actual practice, the minimum solubility of each chemical ion plays a part.

Table 5.16 Illustrative Chemical Water Hardness Precipitation Reactions and Chemical Needs

Precipitation Reactions of $MgSO_4$ – $CaSO_4$						
$MgSO_4$	+	$Ca(OH)_2$	=	$Mg(OH)_2$	+	$CaSO_4$
120		74	=	58	+	136
194			=	194		
2066	+	1274	=	999	+	2341
3340			=	3340		
Precipitation Reactions of $MgCl_2$ – $CaCl_2$						
$MgCl_2$	+	$Ca(OH)_2$	=	$Mg(OH)_2$	+	$CaCl_2$
95		74	=	58	+	111
169			=	169		
4253	+	3313	=	2597	+	4969
6410			=	6410		
Precipitation Reactions of $CaCl_2$						
$CaCl_2$	+	Na_2CO_3	=	$CaCO_3$	+	2NaCl
111	+	106	=	100	+	117
217			=	217		
4969	+	4745	=	4477	+	5237
9714			=	9714		
Precipitation Reactions of $CaSO_4$ – Na_2SO_4						
$CaSO_4$	+	Na_2CO_3	=	$CaCO_3$	+	Na_2SO_4
136		106	=	100	+	142
242			=	242		
2341+4313=6654	+	5186	=	4893	+	6947
11840			=	11840		
Precipitation Reactions of Na_2SO_4 – No further precipitations needed						
Na_2SO_4	+	$BaCl_2$	=	$BaSO_4$	+	2NaCl
142		208	=	233	+	117
350			=	350		
6947	+	10176	=	11399	+	5724
17123			=	17123		
Chemicals added						
$Ca(OH)_2$ = 1274 + 3313 = 4587 (+) Na_2CO_3 = 4745 + 5186 = 9931 (+) $BaCl_2$ = 10176 = 24694 mg/l						
Precipitates						
$CaCO_3$ = 4477 + 4893 = 9370 (+) $Mg(OH)_2$ = 999 + 2597= 3596 (+) $BaSO_4$ = 11399 (+) 92 SiO_2 = 24457 mg/l						

Mass balance becomes 24694 Vs 24457. This yields 99 % and is ok. If decimals are accounted for, there will be 100 % balance

For example, the Calcium and Magnesium can never be precipitated all the way down to zero and there will be a residual of 15 to 20 mg/l remaining as ion in solution. Moreover, the equations are pH dependant especially for Ca and Mg as shown in Figure 5.46 and Figure 5.47.

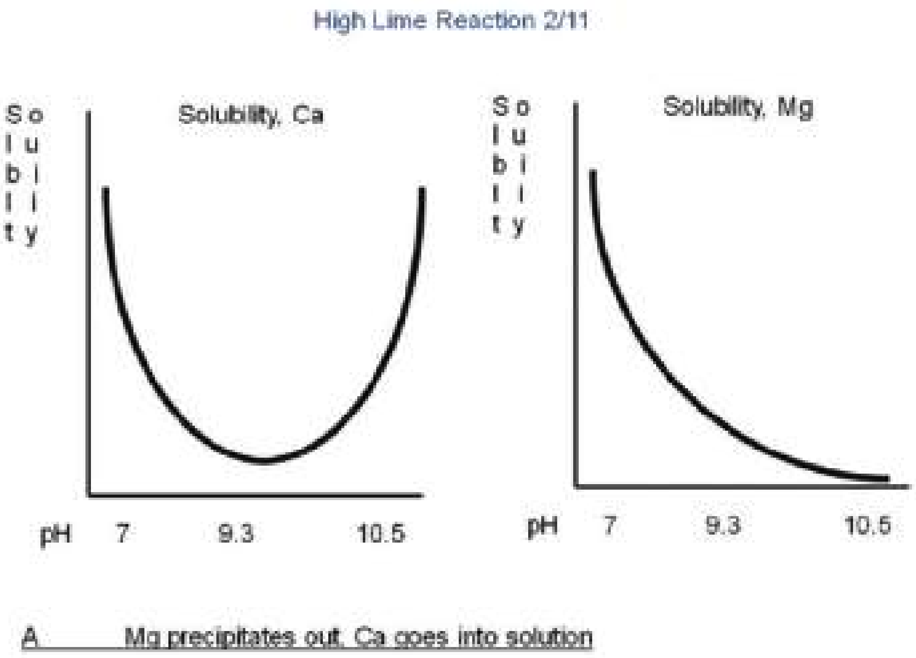


Figure 5.46 Calcium and magnesium solubility with pH variation in water

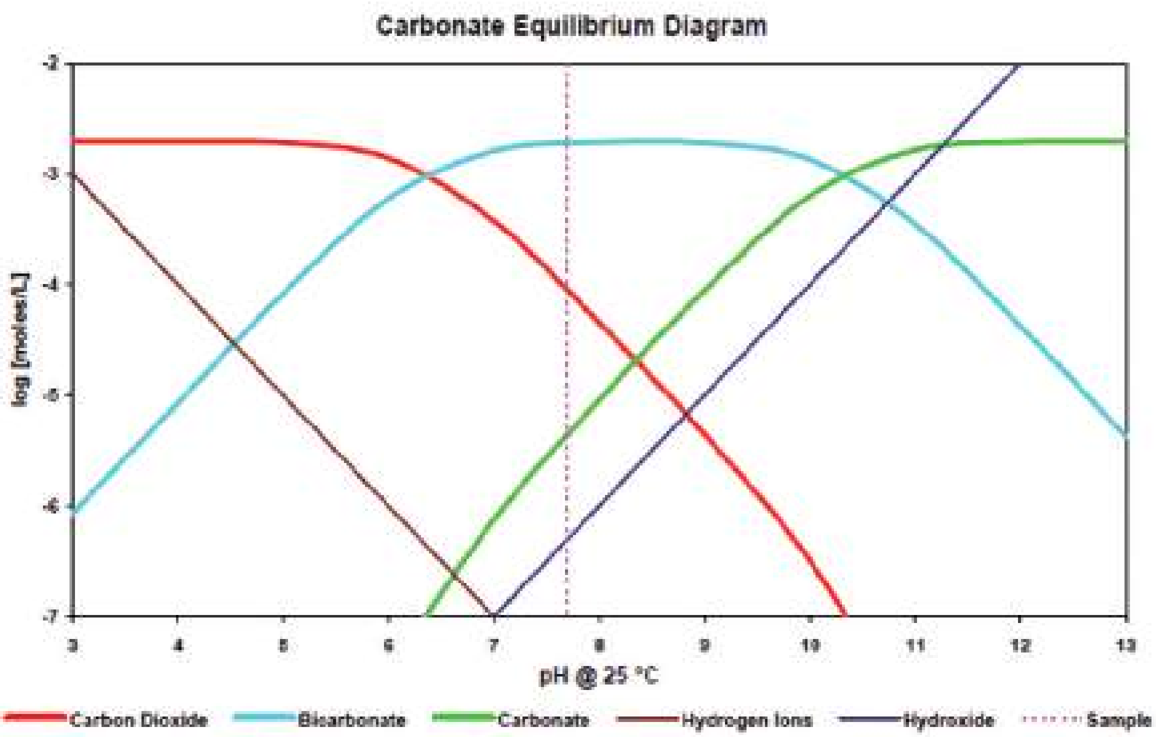


Figure 5.47 Calcium carbonate equilibrium at pH variations

An important factor to be considered in applying these equations to sewage is the fact that even a trace of phosphorous interferes with the precipitation of the compounds of reaction because it coats these compounds as a slimy layer similar to what occurs when Calcium Phosphate when used to prevent scale formation in cooling water circuits. Thus, in applying the equations of Table 5-16 to sewage, it needs to be stressed that the organic matter is to be first removed along with as much phosphorous in secondary treatment by augmenting the same and then only the hardness removal by chemical softening has to be thought off.

This again brings in the question of first removing the phosphorous before attempting to remove the hardness implying a two stage chemical softening. The chemical precipitation of phosphorous is by the use of Ferric or Aluminium salts.

For each Kg of phosphorous 0.9 kg of Aluminium or 1.8 kg of Iron is needed, showing that the sludge production is less by half by using Aluminium. The chemical equations are as under

$$Al_2(SO_4)_3 \cdot 18H_2O + {}_2H_3PO_4 = 2AlPO_4 + 3H_2SO_4 + 18H_2O \qquad (5.41)$$

$$FeCl_3 \cdot 6H_2O + H_2PO_4- + 2HCO_3- = FePO_4 + 3Cl- + 2CO_2 + 8\ H_2O \qquad (5.42)$$

Figure 5-48 shows the phosphorus solubility with pH variation in water.

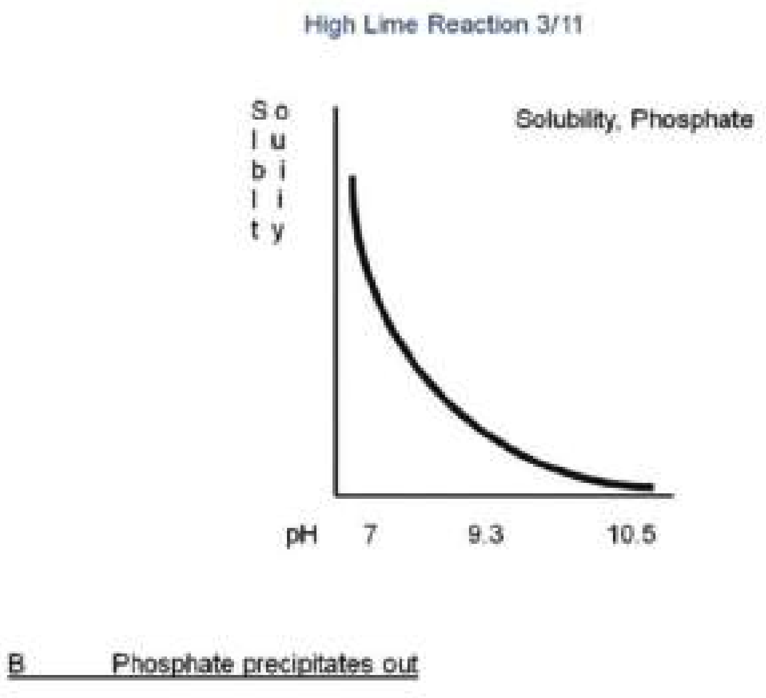


Figure 5.48 Phosphorus solubility with pH variation in water

There are different locations for the addition of the chemical as per various authors. It is added either in the primary clarifier or the aeration tank or in the tertiary stage as in Figure 5.49 (overleaf). In actual practice a separate tertiary stage gives more flexibility and control.

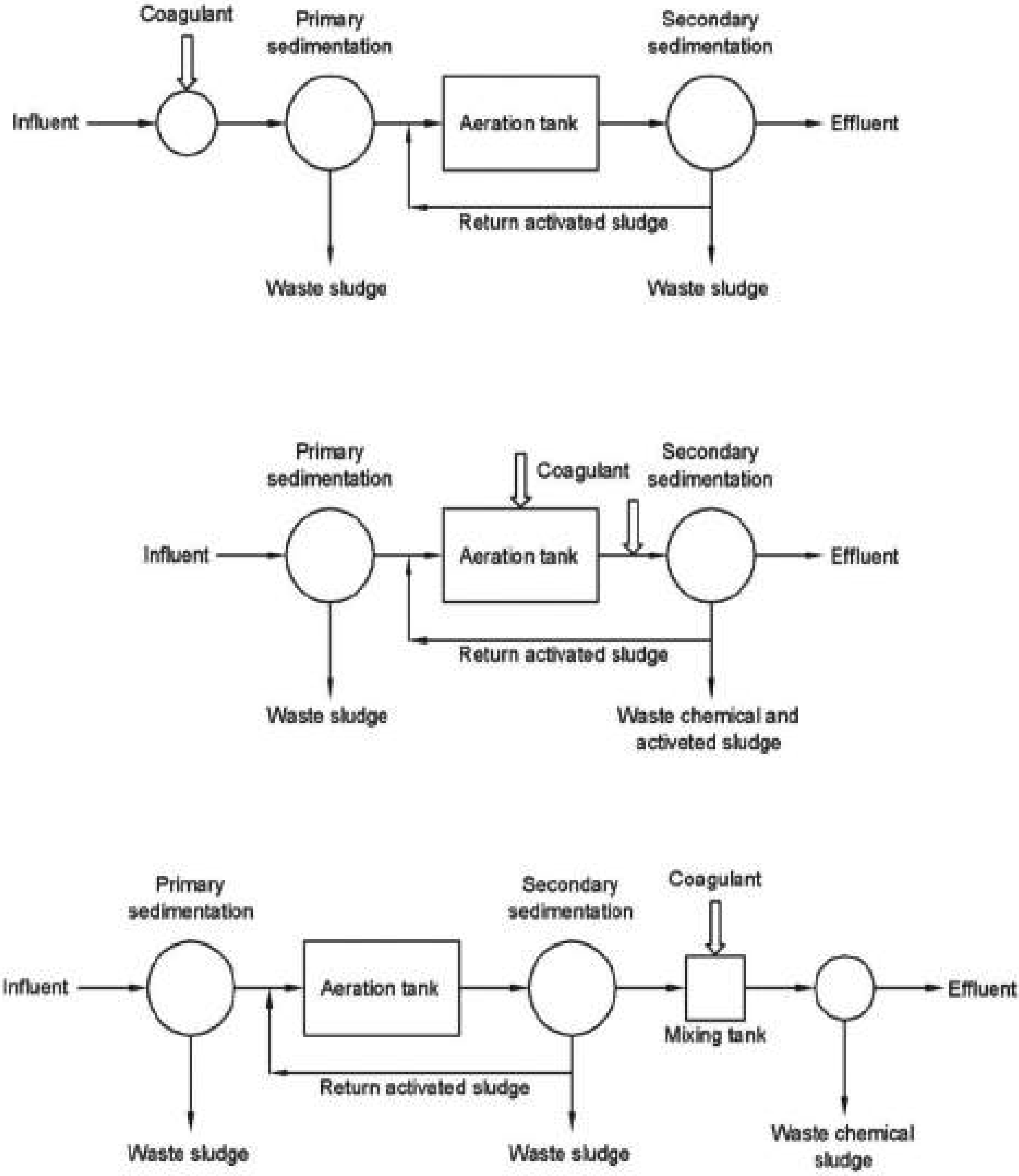


Figure 5.49 Optional points of addition of coagulant chemical in STP
for phosphorus precipitation

Another technology is the high Lime followed by carbonation or acidification. In this case, Lime is added to increase the pH to above 10.5, whereby the Magnesium and Phosphorous are precipitated and simultaneously Silica is co-precipitated with Magnesium at 1:5 as ions. At the same time, by providing a contact period of minimum 45 minutes the waterborne pathogenic organisms are rendered immobile by a process of solidifying their protoplasm thus killing them. In addition, the trace metals present in the aqueous medium are precipitated as their oxides, as the pH increases and once precipitated, they cannot go back into solution. After settling out, pH of the supernatant is carbonated by diffusing carbon dioxide gas. Two-stage carbonation is preferred. In the first stage, the pH is kept around 9.3 to bring about maximum precipitation of the dissolved Calcium as the Calcium carbonate. The pH of the settled overflow is then reduced to around 7 in a separate second stage to dissolve all Calcium as Calcium bicarbonate. The pH can also be reduced by acidifying, but this will convert the Calcium to its sulphate or chloride and increase the TDS.

This high Lime and carbonation technology was evolved many decades back and is very useful in industrial reuse of treated sewage for cooling purposes.

5.8.6.2.2 Membrane Technologies

These are the alternative to removal of hardness, but they result in the removed salts as reject solution, which will need either an ocean disposal or thermal evaporation. Besides they require extensive pre-treatment to eliminate suspended solids altogether. The design guidelines for these are presented in Section 5.18.10.3.

5.9 DISINFECTION FACILITIES

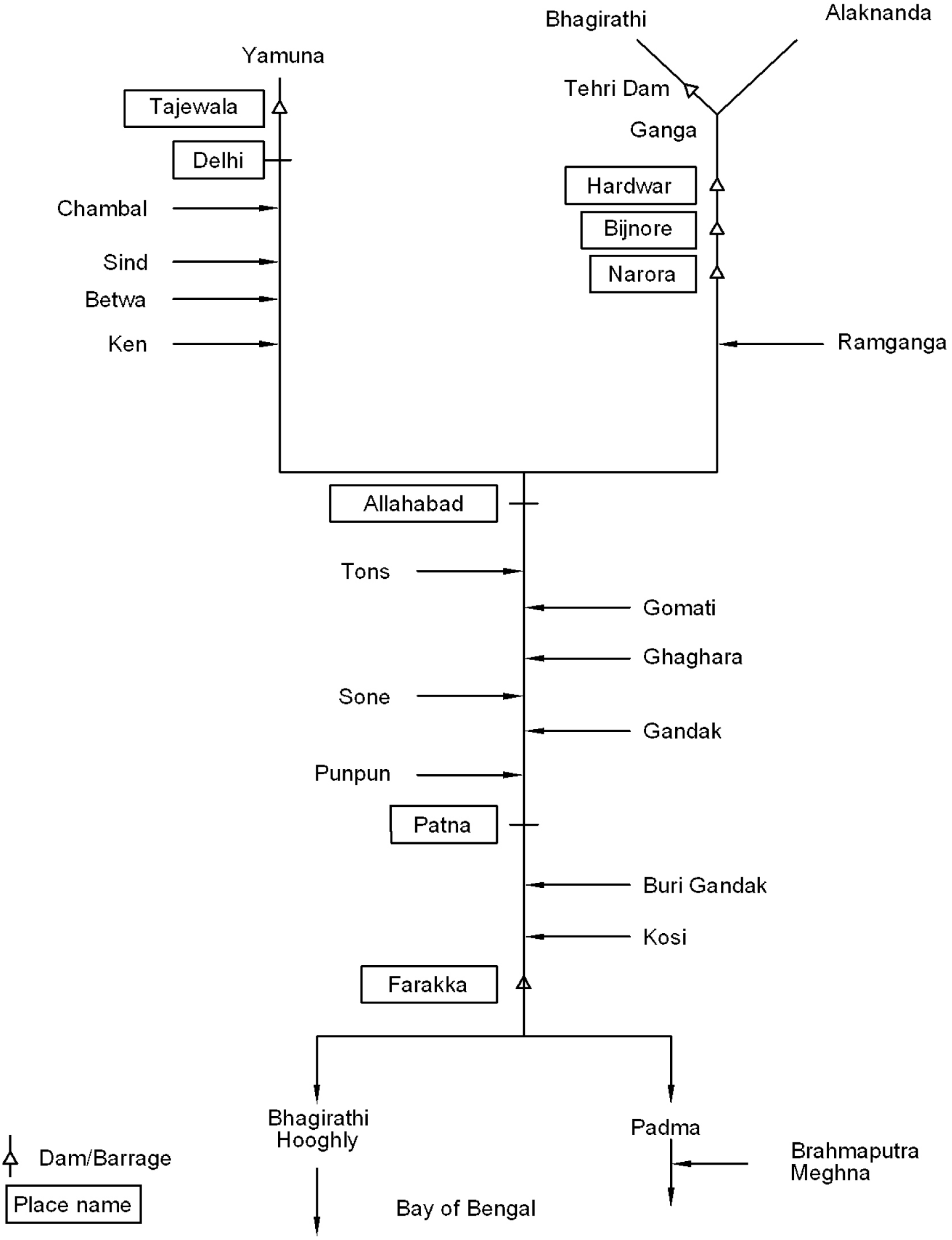
5.9.1 Need for Disinfection

Disinfection of treated sewage may be needed when the receiving water quality may be affected by the Coliforms after the discharge. A case in point is the documentary of the fate of Coliform organisms in India's most revered and most used river Ganga. The habitations supported by this river basin and its tributaries are shown in Figure 5-50 (overleaf) . The results in Table 5-17 shows the BOD and D O at the origin, en route and final confluence over a traverse of 2525 km covering 23% of the country and supporting 43% of its population. Recognizing the seriousness of the pollution, the Ganga Action Plan was completed between the years 1993 and 2000 for sewering and, STPs for 53 habitations covering 1000 MLD. The impact was surely felt, but was not complete enough as can be seen from Table 5.2 read along with Table 5.18.

The point of concern is that despite such massive implementation of sewerage and sewage treatment, almost all stretches elude their up-gradation to the desired grade of at least C, for the rest of the 2100 km after the initial 400 km.

A more recent ongoing work by the IIT's reveals the rather high levels of Total coliforms and Faecal coliforms along this entire course as in Figure 5.51 and Figure 5.52.

The issues of concern are to be read along with Table 5-17, Table 5.2 and Table 5.18 which correlates the total coliform limits for the river classifications A to E.



Source: MOEF, 2009

Figure 5.50 The Traverse of Ganga, the Country's most revered and longest river.

Table 5.17 Variation of data of Ganga River during 1986-2008

No	Location	km	1986		1993		2002		2005		2008		Standard Values	
			(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
1	Rishikesh	0	8.1	1.7	9.0	1.3	8.2	1.2	8.5	1.0	8.1	1.2	5.0	3.0
2	Haridwar	30	8.1	1.8	7.2	1.4	7.8	1.7	8.1	1.4	7.9	1.4	5.0	3.0
3	Garhmukteshwar	175	7.8	2.2	8.5	1.6	7.5	2.1	7.8	2.0	7.8	1.9	5.0	3.0
4	Kannauj U/S	430	7.2	5.5	7.2	2.3	7.7	1.2	8.5	1.7	6.5	2.9	5.0	3.0
5	Kannauj D/S	433	6.5	5.1	8.4	2.5	6.5	4.2	7.6	4.5	6.2	3.1	5.0	3.0
6	Kanpur U/S	530	7.2	7.2	7.5	1.9	6.3	3.8	6.2	4.3	4.9	3.4	5.0	3.0
7	Kanpur D/S	548	6.7	8.6	5.2	24.5	6.7	4.9	4.7	5.4	6.0	4.1	5.0	3.0
8	Allahabad U/S	733	6.4	11.4	6.9	1.8	13.0	8.0	8.5	5.5	8.4	4.8	5.0	3.0
9	Allahabad D/S	743	6.6	15.5	7.2	1.9	8.2	3.8	8.4	3.1	7.7	3.2	5.0	3.0
10	Varanasi U/S	908	5.6	10.1	8.2	0.8	10.8	3.0	8.6	2.0	7.5	2.2	5.0	3.0
11	Varanasi D/S	916	5.9	10.6	7.6	1.0	7.5	2.5	8.3	2.3	7.3	3.0	5.0	3.0
12	Patna U/S	1188	8.4	2.0	8.2	1.2	7.1	1.9	7.4	2.0	6.0	1.7	5.0	3.0
13	Patna D/S	1198	8.1	2.2	8.0	1.5	7.1	2.0	8.0	2.2	5.9	2.4	5.0	3.0
14	Rajmahal	1508	7.8	1.8	8.5	0.7	7.9	1.5	7.4	1.8	6.2	2.0	5.0	3.0
15	Palta	2050	7.3	1.0	7.1	0.9	7.3	2.7	7.0	3.0	6.9	2.2	5.0	3.0
16	Uluberia	2500	5.8	1.1	6.1	0.9	5.4	1.9	5.4	2.6	5.3	3.6	5.0	3.0

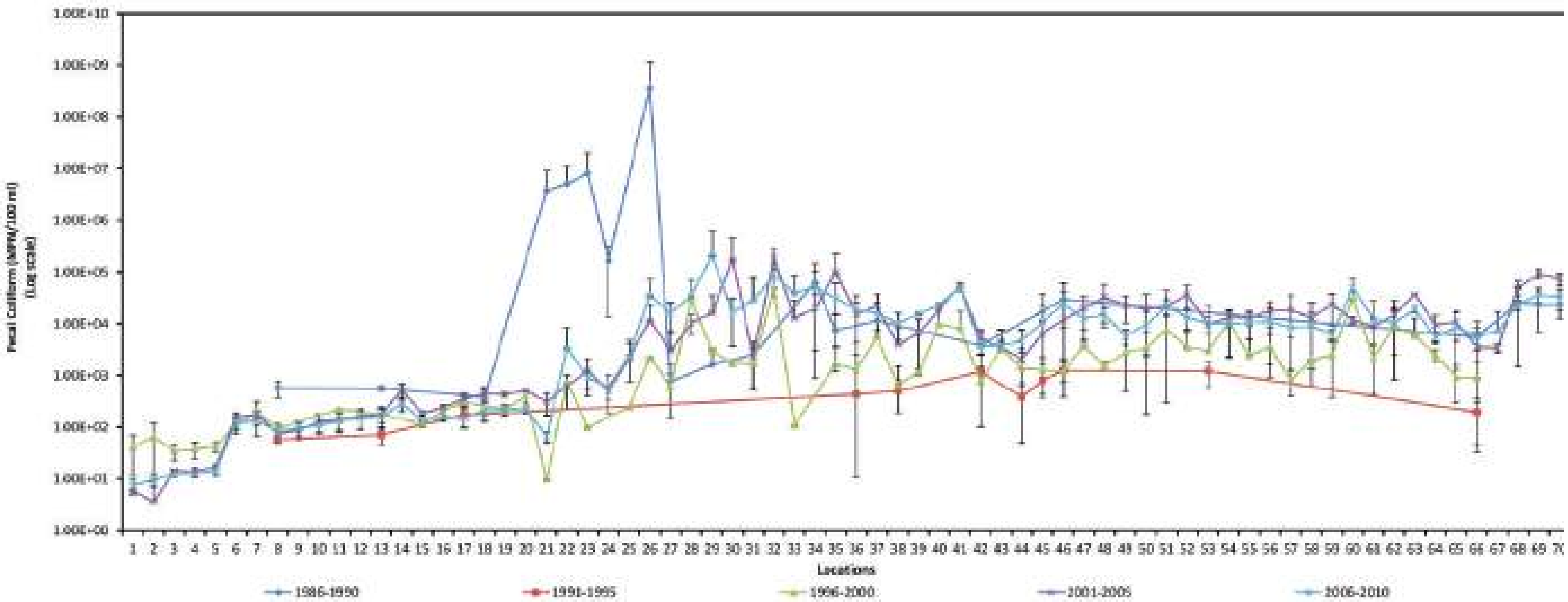
(a) = DO in mg/l, (b) = BOD in mg / l

Source: MOEF, 2009

Table 5.18 Classification of Ganga water at various locations according to designated best use

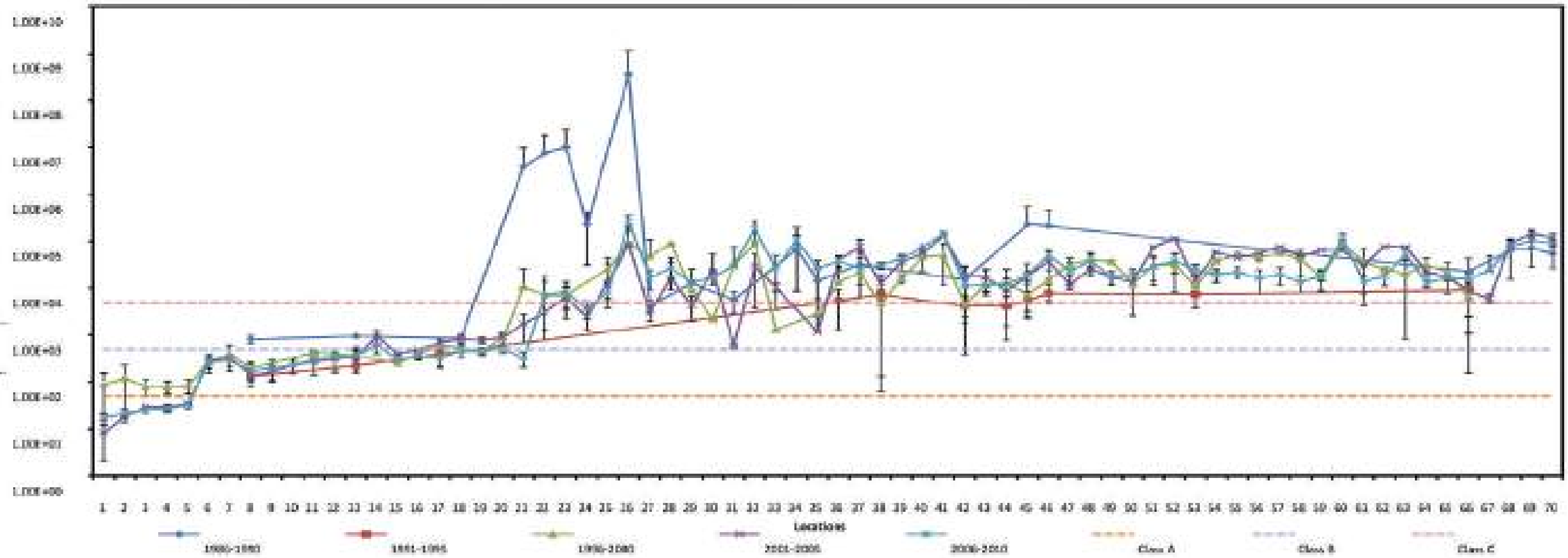
Locations	Desired Class	Observed Class					
		1997	1998	1999	2000	2001	2008*
Rishikesh	A	D	B	C	NA	C	B
Haridwar	B	C	C	C	NA	C	B
Garhmukteshwar	B	B		D	NA	D	NA
Kannauj U/S	B	D	D	D	D	D	D
Kannauj D/S	B	D	D	D	D	D	D
Kanpur D/S	B	D	NA	D	D	D	D
Raibarreilly	B	D	D	D	NA	NA	NA
Allahabad U/S	B	D	E	D	NA	NA	C
Allahabad D/S	B	D	E	D	NA	NA	D
Varanasi U/S	B	D	D	D		D	D
Varanasi D/S	E	E	E	E	NA	NA	D
Gazipur	B	D	D	D	D	NA	NA
Buxar	B	D	D	D	D	D	C
Patna U/S	B	D	D	D	D	D	NA
Patna D/S	B	D	D	D	D	D	NA
Rajmahal	B	D	D	D	D	D	D
Palta	B	D	B	NA	D	D	D
Uluberia	B	D	B	NA	D	D	D
Locations		Critical Parameter					
		1997	1998	1999	2000	2001	2008*
Rishikesh		CF	CF	CF			
Haridwar		CF	CF	CF			
Garhmukteshwar		BOD	BOD			BOD,CF	
Kannauj U/S		BOD,CF	BOD	CF	CF	CF	CF
Kannauj D/S		BOD,CF	BOD	CF	BOD,CF	BOD,CF	BOD,CF
Kanpur D/S		BOD,CF		CF	CF	CF	CF
Raibarreilly		CF	CF	CF			
Allahabad U/S		BOD,CF	CF	CF			CF,BOD
Allahabad D/S		BOD,CF	CF	CF			CF,BOD
Varanasi U/S		BOD,CF	BOD	CF		CF	BOD,CF
Varanasi D/S		DO,BOD,CF	DO,BOD	BOD			BOD,CF
Gazipur		BOD,CF	BOD	BOD	CF		
Buxar		BOD	CF	CF	CF	CF	CF
Patna U/S		CF	CF	CF	CF	CF	
Patna D/S		CF	CF	CF	CF	CF	
Rajmahal		CF	CF	CF	CF	CF	CF
Palta		BOD			BOD,CF	BOD,CF	CF
Uluberia						BOD,CF	BOD,CF

A- Drinking Water Source without conventional treatment but after disinfection
B- Outdoor bathing (Organized)
C- Drinking Water Source with Conventional treatment followed by disinfection
D- Propagation of wild life and fisheries
E- Irrigation, industrial cooling and controlled waste disposal
CF-Coliform Indicator Organisms
Source: MOEF, 2009



1	Uttarkashi u/s (Bhagirathi)	15	Bijnore u/s (Ganga)	29	D/s Deehaghat	43	Hajipur u/s (River Gandak)	57	Sultanganj d/s
2	Uttarkashi d/s (Bhagirathi)	16	Bijnore d/s (Ganga)	30	Vindhyachal, Pakka Ghat	44	Hajipur d/s (River Gandak)	58	Bhagalpur u/s
3	Devprayag u/s (Bhagirathi)	17	Garhmukteshwar u/s	31	Vararasi u/s	45	Patna u/s	59	Bhagalpur d/s
4	Devprayag u/s (Alaknanda)	18	Garhmukteshwar d/s	32	Dashashawmedh Ghat	46	Patna d/s	60	D/s Champanala
5	Devprayag d/s (Ganga)	19	Anoopshahr u/s (Ganga)	33	D/s at Kaithy	47	Fatuha u/s	61	Kahalgaon u/s
6	Ranipur u/s (Ganga)	20	Anoopshahr d/s (Ganga)	34	Near Malviya Bridge	48	Fatuha d/s	62	Kahalgaon d/s
7	Ranipur d/s (Ganga)	21	Fatehgarh u/s	35	Tarighat	49	Barh u/s	63	D/s NTPC Drain
8	Rishikesh u/s	22	Kannauj u/s (a/c with Ramganga & b/c with Kali)	36	Buxar u/s	50	Barh d/s	64	Sahebganj u/s
9	Rishikesh d/s	23	Kannauj d/s (a/c with Kali)	37	Buxar d/s	51	Mokama u/s	65	Sahebganj d/s
10	Haridwar u/s	24	Kanpur u/s (Bithoor)	38	Chapra u/s (Ghaghra)	52	Mokama d/s	66	Rajmahal d/s
11	Har-ki-Paudi	25	Kanpur d/s (Shuklaganj)	39	Chapra d/s (Chapra)	53	D/s Bata - McDowell	67	Berhampore (Middle)
12	Lalta Rao	26	Kanpur d/s (Jane Village)	40	Arrah u/s (River Gangi)	54	Munger u/s	68	Palta (Middle)
13	Dam Kothi	27	Allahbad u/s (Ujahni, Fatehpur)	41	Arrah d/s (River Gangi)	55	Munger d/s	69	Dakshineswar (Middle)
14	Mishrpur	28	Bathing Ghats at Sangam	42	Koliwar (River Sone)	56	Sultanganj u/s	70	Uluberia (Middle)

Figure 5.51 Variation in 5-year average Faecal Coliform at various locations along the Ganga River



1	Uttarkashi u/s (Bhagirathi)	15	Bijnore u/s (Ganga)	29	D/s Deehaghat	43	Hajipur u/s (River Gandak)	57	Sultanganj d/s
2	Uttarkashi d/s (Bhagirathi)	16	Bijnore d/s (Ganga)	30	Vindhyachal, Pakka Ghat	44	Hajipur d/s (River Gandak)	58	Bhagalpur u/s
3	Devprayag u/s (Bhagirathi)	17	Garhmukteshwar u/s	31	Varanasi u/s	45	Patna u/s	59	Bhagalpur d/s
4	Devprayag u/s (Alaknanda)	18	Garhmukteshwar d/s	32	Dashashawmedh Ghat	46	Patna d/s	60	D/s Champanala
5	Devprayag d/s (Ganga)	19	Anoopshahr u/s (Ganga)	33	D/s at Kaithy	47	Fatuha u/s	61	Kahalgaon u/s
6	Ranipur u/s (Ganga)	20	Anoopshahr d/s (Ganga)	34	Near Malviya Bridge	48	Fatuha d/s	62	Kahalgaon d/s
7	Ranipur d/s (Ganga)	21	Fatehgarh u/s	35	Tarighat	49	Barh u/s	63	D/s NTPC Drain
8	Rishikesh u/s	22	Kannauj u/s (a/c with Ramganga & b/c with Kali)	36	Buxar u/s	50	Barh d/s	64	Sahebganj u/s
9	Rishikesh d/s	23	Kannauj d/s (a/c with Kali)	37	Buxar d/s	51	Mokama u/s	65	Sahebganj d/s
10	Haridwar u/s	24	Kanpur u/s (Bithoor)	38	Chapra u/s (Ghaghra)	52	Mokama d/s	66	Rajmahal d/s
11	Har-ki-Paudi	25	Kanpur d/s (Shuklaganj)	39	Chapra d/s (Chapra)	53	D/s Bata - McDowell	67	Berhampore (Middle)
12	Lalta Rao	26	Kanpur d/s (Jane Village)	40	Arrah u/s (River Gangi)	54	Munger u/s	68	Palta (Middle)
13	Dam Kothi	27	Allahbad u/s (Ujahni, Fatehpur)	41	Arrah d/s (River Gangi)	55	Munger d/s	69	Dakshineswar (Middle)
14	Mishrpur	28	Bathing Ghats at Sangam	42	Koliwar (River Sone)	56	Sultanganj u/s	70	Uluberia (Middle)

Figure 5.52 Variation in 5-year average Total Coliform at various locations along the Ganga River

Thus, it is to be conceded that once the initial 400 km is traversed, almost the entire remaining stretch is not fit to the desired classification of at least C water quality. Here is the realistic need for disinfection of the treated sewage discharges to reverse the trend. Coliform organisms are indicator organisms for presence of waterborne disease causing organisms like potentially epidemic causing Cholera, Typhoid, Jaundice, etc. The faecal coliforms are a sub-group within the total coliforms and are a reinforcing indicator-organisms. The present situation does not portend well for the future of this river. The origins of faecal coliforms can be only from human activities of improper sewerage and inadequate sanitation. As such, there arises a need to look into more of coliform aspects and extend beyond the typical discharge standards of 20 mg/l BOD and 30 mg/l of SS when specifying discharge standards for treated sewage discharging into surface waters, which are to be used as potable water sources. Reference to Appendix A.2.1 also needs to be recalled in this context of faecal coliforms. The position in other non-perennial water-courses in the country cannot be anything better.

Recent tenders for STPs are already restricting the Faecal Coliform limit at not over 200/100 ml but the raw sewage value is in the range of 16,00,000/100 ml. As earlier shown in Table 5.5, the reduction even in ASP is only about 96%, which means the treated sewage will have 64,000/100 ml, in order to kill the same and attain the limitation, disinfection of the treated sewage is needed.

5.9.2 Disinfection by Chlorination

This is the most widely used technology in both water supply and sewage treatment. As the treated sewage is fresh from secondary aerobic biological treatment, the chlorination of such effluents does not result in hazards. In the case of effluents from anaerobic processes like UASB, the provision of an aerobic polishing treatment is mandatory before chlorination. The usual dosage used is 10 mg/l and the flow through detention time in the contact tank is 30 minutes based on average flow. Suitable baffles are provided in these tanks to maximize the contact. These tanks shall not be covered. This is because the chlorine gas may permeate into the concrete in the roof and corrode the concrete slab. There is no way this can be detected periodically. It will be known only when the roof collapses. The worst situation will be when operators are standing on the roof. Hence, open tanks and free wind movement must be allowed to blow across the tank. This will also help in detecting excess chlorination. The residual chlorine after the contact has been generally detected at 1 to 1.5 mg/l at the maximum and there are no offensive odours arising there from. In actual practice, the dosage can be varied to be in conformity with faecal coliform limits.

5.9.2.1 De-chlorination

Excess of residual chlorine if any is nullified by dechlorination chemicals like sulphur dioxide (SO_2) gas or salts as sodium thiosulfate ($Na_2S_2O_3$), sodium sulphite (Na_2SO_3), sodium bisulfite ($NaHSO_3$), sodium metabisulfite ($Na_2S_2O_5$), calcium thiosulfate (CaS_2O_3), ascorbic acid (Vitamin C) and sodium ascorbate. sodium bisulfite is used by some utilities due to its lower cost and higher rate of de-chlorination. Sodium sulphite tablets are chosen by utilities due to ease of storage and handling, and its ease of use for de-chlorinating constant, low flow rate releases. Sodium thiosulfate is used for de-chlorination since it is less hazardous in handling and consumes less oxygen than sodium bisulfite and sodium sulphite. Ascorbic acid and sodium ascorbate are used because they do not impact the D O concentrations. Several chemicals are available for de-chlorination.

Additionally, chemicals such as Sodium Metabisulfite, Sodium Sulphite and Sodium Thiosulfate may deplete D O of receiving streams under certain circumstances. Sodium Metabisulfite and Ascorbic acid may decrease the pH of some waters. It is necessary to determine in the laboratory the choice of the chemical for the given sewage quality and keep stock of the chemical for a demand of at least a week. The standard engineering procedures for dispensation shall be organized in consultation with the Material Safety Data Sheet (MSDS) and the recommendations of the authorised supplier.

5.9.3 Ultraviolet Radiation Disinfection

Ultraviolet rays are most commonly produced by a low pressure mercury lamp constructed of quartz or special glass which is transparent and produces a narrow band of radiation energy at 2537 A0 emitted by the mercury vapour etc. Though this is a standard chemistry, in actual practice, its efficiency is largely constrained by the requirements of (a) The water to be free from suspended and colloidal substances causing turbidity, (b) The water does not contain light absorbing substances such as phenols, ABS and other aromatic compounds, (c) The water is flowing in thin film sheets and is well mixed and (d) Adequate intensity and time of exposure of UV rays. The advantage of UV is that exposure is only for short periods, no foreign matter is actually introduced and no toxic and no odour is produced. Over exposure does not result in any harmful effects. The disadvantages are that no residual effect is available and there is lack of field test for assessing the treatment efficiency. Moreover, the equipment needed is expensive.

5.9.4 Ozone Systems

It is a faintly blue gas of pungent odour. Being unstable, it breaks down to normal oxygen and nascent oxygen. This nascent oxygen is a powerful oxidizing agent and germicidal agent. Ozone is produced by the corona discharge of high voltage into dry air and being unstable has to be produced on-site. It poses more superior bactericidal properties than chlorine and is highly effective in removal of tastes, odours, iron and manganese. As Ozone reacts with chemical impurities prior to attacking the microorganisms, it produces essentially no disinfectant unless ozone demand of water has been satisfied, but much more rapid kills are achieved once free ozone residuals are available. Studies have reported 99.99% kills of E Coliform within less than 100 seconds in the presence of only 10 mg/l of free residual ozone. Moreover, the efficiency of its disinfection is unaffected by pH or temperature of the water over a wide range. Among the disadvantages are (a) its high cost of production, (b) its inability to provide residual protection against recontamination and (c) the compulsion for its generation on-site due to instability.

5.9.5 Relative Aspects of Disinfection Processes

In a recent finding the US Water Environment Federation (WEF) observed that “disinfection of wastewater protects the public from potential exposure to pathogenic microorganisms that would otherwise be present in wastewater effluent that is discharged into water bodies that may be used for recreation or drinking water. Wastewater disinfection has traditionally been accomplished using some form of chlorination. In fact, more than 60% of the 20,000 municipal wastewater treatment plants in North America use chlorination as the primary method of disinfecting effluent. Although an effective disinfectant, chlorine (and related compounds) has come under increased scrutiny because of regulatory, safety and security issues.

An on-going unpublished work observes that after studying the performance of disinfectants of chlorination, its variants, solar, UV, Ozone and Peracetic acid, the faecal coliform removal was about the same at up to 4<5 log unit, except in the case of solar where it was up to 2<3 and total coliforms <1000 for all except UV at <100 and Ozone at < 50.

The occurrence and fate of disinfection by-products and related residuals are not readily available in validated reportings.

The relative efficiencies of disinfectants vs. their by-products is long engaging the attention worldwide. Most of the reported works are only in respect of surface waters, ground waters, surface runoff waters, etc. The findings of these studies do not fully apply to disinfection of treated sewage. The US-EPA-Design Manual on Municipal Wastewater Disinfection-EPA/625/1-86/021 observes that even otherwise, the issue of attention has been the disinfection by-products. Though it is contended that chlorination may result in by-products of Trihalomethanes, it needs to be realized that it is the case only when chlorination of humic substances takes place and a treated sewage from an aerobic STP does not have humic substances. Moreover, the inherent alkalinity in sewage curtails on the THM formation potential because the alkalinity in sewage scavenges any hydroxyl free radicals.

In respect of UV, the distribution of biologically stable water is realized by reducing the AOC concentration using GAC filtration only after UV disinfection and as such, UV by itself is not a complete treatment. In respect of Ozonation, the overall effect of ozone on effluent toxicity have found the effects to be variable as ozonation of secondary sewage can both decrease or increase effluent toxicity. Considerable research is still needed on the formation of ozone by-products and the effect of ozone on the treated sewage. It also proposes an approach as in Table 5.19.

Table 5.19 Technical factors and feasibility considerations in disinfectant choice

No.	Considerations	Chlorine	Chlorine Dioxide	Ozone	UV
1	Flexibility	2	2	2	2
2	Reliability	1	2	3	2
3	Complexity	2	3	3	2
4	Effectiveness	2	1	1	2
5	Need for Piloting	1	4	3	3
Rating based on scale of 1 to 5 with 1 indicating best degree of confidence					

Source: USEPA-EPA/625/1-86/021

In considering the foregoing in this sub-section, the recommendation of the CPCB in its publication titled (Performance of Sewage Treatment Plants - Coliform Reduction - CUPS/ 69 /2008-CPCB) that "one of the best methods of achieving 100% faecal microbes removal is coagulation-flocculation followed by chlorination after secondary treatment" appears appropriate as of now, pending more conclusive publications on other disinfection methods.

5.10 THE ISSUES OF NITROGEN AND PHOSPHORUS

It is to be recognized that large volumes of sewage, which far exceed the treatment capacity of sewerage systems and the natural capacity of rivers to purify water, are discharged untreated into rivers untreated. There is thus a serious degradation of the quality of river water, as well as deterioration of the living conditions in urban areas (Source : JICA Press Release-NR/2007 2 April 2, 2007). An example is the Agra city-water supply project, where ammonia pollution of the Yamuna river water is attributable to partly treated sewage discharged into it. The proposed treatment is a biological MBBR and ultra filtration membrane system for biological nitrification-denitrification of the raw water. By instituting this biological nitrification-denitrification in STPs itself, it is relatively easier to deal with far less volumes as compared to higher volumes of polluted water in such WTPs.

The nitrogen in sewage consists of organic nitrogen and ammoniacal nitrogen and their sum total is expressed as Total Kjeldahl nitrogen (TKN). The phosphorous is consisting of dissolved phosphorus and total phosphorus. The discharge standards permit the TKN at 100 mg/l and dissolved phosphorous at 5 mg/l. These chemicals are well known to contribute to eutrophication in receiving waters, especially stagnant ones.

There has been at least one instance in the country in which the uncontrolled sewage nutrients got accumulated in a downstream impoundment and the drought of that year resulted in high algal growth. The conventional water treatment plant with coagulation, sand filtration and disinfection could not remove the algal stench and the colour and the WTP was rejected outright by the population of Hosur town and water famine loomed large. The ground water was also at least 100 m deep. Emergency treatment was carried out for removal of nitrogen and phosphorous and algae by high lime and ammonia stripping followed by carbonation and sand filtration with final chlorination. The WTP was retrieved and the population was spared of a water chaos. (Source: S Saktheeswaran, 2011) The eutrophication issue has not assumed such proportions in the other water courses of the country.

However, if biological nitrification-denitrification can be done in STPs, such difficulties will not recur in the receiving waters.

Biological removal of phosphorous has undergone many advances since the days of Professor James L Barnard, the father of biological phosphorous removal. It is prudent to adopt such biological treatments in the STPs along with the traditional BOD removal.

Some of the biological STPs built in India have already established such a performance like the Koramangala and Chellagatta (K&C) valley STP at Bengaluru, constructed during 2004 under JICA assistance and some of the recent SBR plants.

5.11 DESIRABLE TREATED SEWAGE QUALITY AND PROCESSES

Considering the foregoing aspects and taking a comprehensive view, it appears necessary to embark on a set of tougher limitations of BOD, nitrogen and phosphorous especially in respect of treated sewage discharges into water bodies, which becomes a raw water source for drinking water projects of downstream habitations.

Kazmi et.al. after studies in respect of removal of coliforms in full-scale activated sludge plants operating in northern regions of India, concluded that the inter-relationship of BOD and SS with coliforms manifest.

The improvement of the microbiological quality of sewage could be linked with the removal of SS, and therefore, SS can serve as a regulatory tool in the absence of an explicit coliforms standard. (Kazmi et al., 2008).

Hence, it is recommended that the stringent limitations shall be henceforth adopted as guidelines for treated sewage discharge into surface water which after some travel may join a drinking water source to be used as source of supply for drinking as given in following Table 5.20

Table 5.20 Recommended Guidelines for Treated Sewage if Discharged into Surface Water to be used as source of Drinking Water

Parameter	MOEF Standards (A)	Recommended Values
BOD, mg/L	30	Less than 10
SS, mg/L	100	Less than 10
TN, mg/L	100	Less than 10
Dissolved P, mg/L	5	Less than 2
Faecal Coliforms, MPN/100 mL	Not specified	Less than 230

(A) General Standards, Environmental Protection Rule, 1986 & as authorised by PCB

In order to achieve this, the treatment process would need to be designed for nutrient removal in addition to the conventional BOD and SS removal.

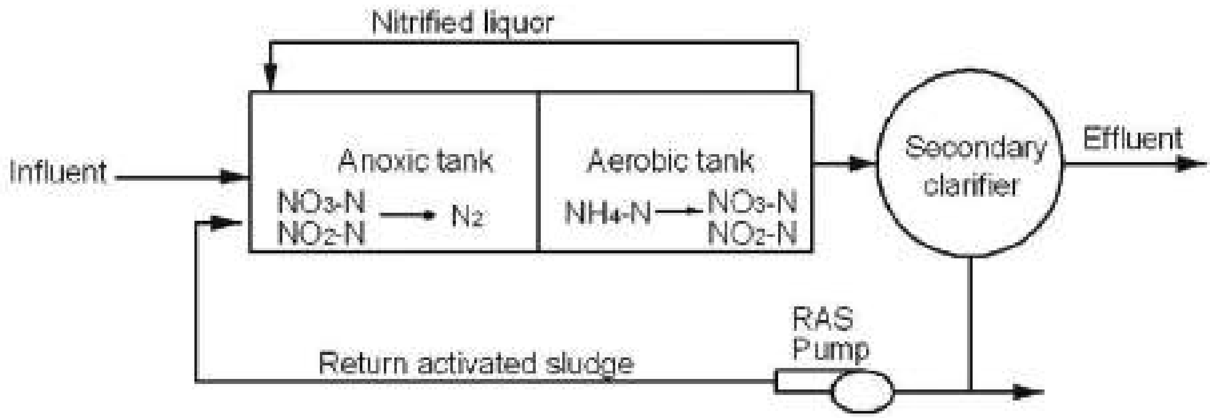
It has also been reported that if the nutrients were removed to the levels mentioned in Table 5.20, then the amount of chlorine required for disinfection would be less at about 5 mg/l.

However, the central and local regulatory agencies / bodies can set the discharge standards as felt appropriate.

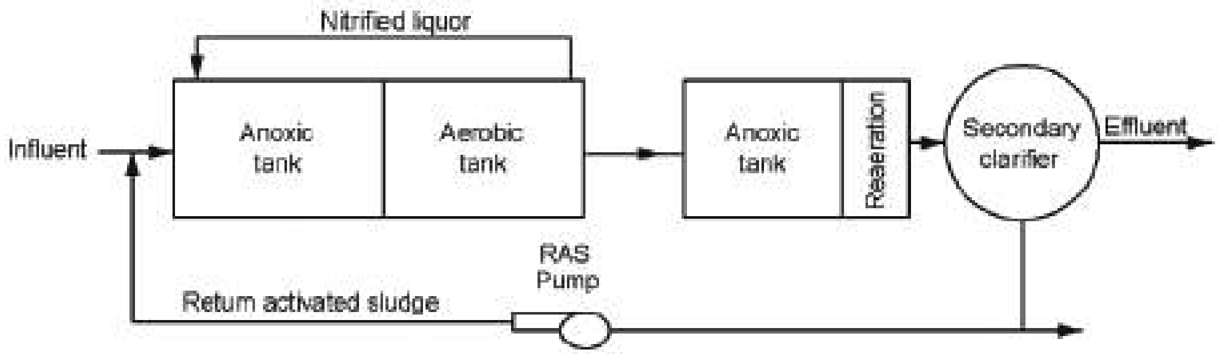
Disinfection as in Section 5.9 will be required to achieve the Faecal Coliform (FC) limit. The following treatment processes shown in Figure 5.53 (overleaf) could be used for achieving nutrient removal.

If effluent BOD and SS of less than 10 mg/L cannot be achieved by biological processes, then they shall be followed by chemically enhanced settling process. The option of primary clarification and anaerobic digestion of sludge and gas recovery can also be considered based on capacity of plant and actual design values.

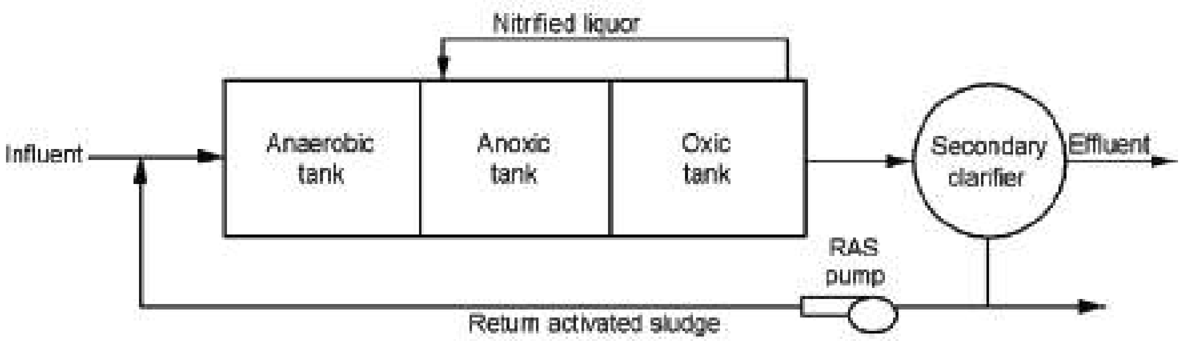
It also needs to be mentioned that discharge of treated sewage and untreated sewage mixing is to be totally avoided and hence interceptor sewers for transportation of the untreated sewage up to the STP should be practiced.



Case 1: Modified Ludzack Ettinger (MLE) Process with chemical enhanced settling & Chlorination



Case 2: Bardenpho process with Chlorination



Case 3: Bardenpho process with Chlorination

Figure 5.53 Nutrient removal processes

5.12 ELECTRICAL AND INSTRUMENTATION

5.12.1 General

The electrical and instrumentation system in a sewage pumping station or sewage treatment plant is the same as any other system in other infrastructure projects. However, in addition, the following must be looked into.

a) Corrosive gas: Hydrogen Sulphide is a corrosive gas, which may be present in the air in some locations like sewage wells, pump dry pits and many of the treatment plant units. This gas can be more corrosive when humidity is very high like in coastal zones where it can be high. These gases can attack and damage the electrical contacts, in conductors and switches.

b) Inflammable gas: Methane is an inflammable gas, which may be present in the air in many units of the sewage pumping stations and treatment plants.

c) Variations of raw sewage flows: This causes the motors and switches to start and stop frequently, more than the recommended operations.

These needs, along with other general requirements of an electrical system have to be satisfied while deciding on the rating, sizing and designing of the electrical and instrumentation system.

These can be broadly classified into the following groups;

i) Power receiving and transforming equipment (Substation & Transformers)
ii) Motors and motor controllers (Starters, Cabling, etc.)
iii) Generators for captive / back-up power supply
iv) Uninterruptible power supply for protection and control circuits
v) Instrumentation facilities
vi) Supervisory control and data acquisition system (SCADA)

5.12.2 Power Receiving and Transformation Equipment

Indoor / Outdoor substations are to be provided for higher voltage transformers in the upstream of indoor switchgear. A typical substation has the following major units.

1. Supply System:
   a) Low Tension Supply up to 150 HP (50Hz - 3Phase - 433V between phases)
   b) High Tension Supply above 150 HP through Indoor/Outdoor Substation (50 Hz-3Phase-11 KV/22 KV/33 KV in between phases)
2. Lightning arresters
3. Group operated switches (GOS) on both sides of the Circuit Breaker for 1000 KVA and above
4. Circuit Breakers

5. Drop out fuses for outdoor substations
6. Overhead bus bars, conductors and insulators
7. Transformers
8. Current transformer and potential transformer for power measurement
9. Current transformer and potential transformers for protection in substations of above 1000 KVA
10. Fencing and Yard-lighting
11. Earthing

5.12.2.1 Location

The Load switch/Substation is normally located at one end of the plant, preferably in the vicinity of the largest load centre in the plant and the easy and unhindered accessibility of the power supply agency should be considered. An overhead line or underground cable from the LT or HT supply system of the power supply company may have to be brought and terminated at the Load switch/station. In the case of a sewage treatment plant, normally the raw sewage pumping station is usually the largest power centre, followed by the aerators/air compressors and recirculation pumps. The substation location is to be at an elevated place to avoid any flooding risks. Due consideration shall be given to easy entry and easy exit for vehicles at all times.

5.12.2.2 Rating and Supply Voltage

The rating of transformers is to be worked out by summing up all electrical loads that would work at a time, and adding a margin to it. The transformer is specified to work at its peak efficiency at about 75% of its rating so that a cushion is always available for any expansion, temporary loads, starting loads, and de-rating factors like possibility of higher ambient temperature, altitude, etc. The supply voltage of the substation will be decided by the total load current and the nearest supply voltage available.

5.12.2.3 Lightning Arresters

These are to be provided at the commencement of a substation conductor to draw away any power spike due to atmospheric and switching surges, and to protect the downstream equipment. The lightning arresters are earthed to conduct away the surge currents to the earth via earthing conductors and through earth pits.

5.12.2.4 Group Operating Switches (GOS)

These are to be provided as isolating facility to major equipment, such as a transformer or HT switchgear and come in different versions like single throw, double throw, earthed and unearthed for different applications and at a safe clearance height from the ground. These are made up from three separate switch units for each of the 3 phases, adding a common operating lever/rod, for operation from ground level. They are suitable for opening only when there is no load on the downstream side. In short, the downstream major loads should be switched off before switching off the supply by a GOS and the GOS should be locked in the open position to enable repair works to be done safely.

5.12.2.5 Circuit Breaker (CB)

These are insisted as an isolating mechanism for stations with transformers above 750 kVA by the power supply/Inspectorate authorities. However, it is common to provide Circuit Breakers even for 500 kVA transformer substations in the interest of a safe and reliable operation. The main difference between a GOS and CB is that, while GOS can be operated in the no-load condition only, a CB can be operated when the station load is on and more importantly, the CB has provision for tripping through a remote signal. This remote signal is useful for switching off supply when a fault or abnormal condition is sensed by the electrical metering / protection equipment. For small stations, there will be a single CB as main switchgear and one or more transformers will have GOS. For large substations there can be a main and additional transformer control CBs too. The sequence of operation of the CBs in such a case will be decided by suitably grading the currents at which the switchgear will trip. This is done at the protective relay taps. The tripping time of CBs is very short and is specified in milliseconds and these are of several types based on the insulating medium such as oil, vacuum, SF-6 gas, etc. Further, they are also classified on the voltage class as 3.6 kV, 7.2 kV, 12 kV, 24 kV, 36 kV, 72.4 kV, 123 kV etc.

5.12.2.6 Drop Out Fuses (DOF)

These are a means of protection to a fault current for small transformers up to 500 kVA. They are to be suspended from the pole and conductor connecting to the transformer primary. When the current exceeds a specified limit, the fuse wire in it melts and disconnects the supply and at the same time dropping off indicating the fault. They are normally connected just before the transformer primary connection. The fuse wires are made of an alloy of lead and tin and with inverse time characteristics. This means that the higher the short-circuit current, the faster will be the fuse blow-out time.

5.12.2.7 Overhead Bus Bars

These are conductors to carry power from each component to the next component in the power circuit in an outdoor station. They are to be rated to carry the maximum rated current continuously and the short-circuit current for a short time without damage.

5.12.2.8 Transformer

This is the most important component in the substation. It receives the electrical power at a higher voltage and steps it down to a lower service voltage. The transformers have insulating oil in their tanks where the HT and LT coils are wound around a core. The oil serves as an insulating and a cooling medium to disperse the heat that is generated during the operation. The oil is in turn cooled by circulating air around radiator fins. The connections on HT and LT side shall be through overhead bushings, cable termination or a combination of these. Being a large equipment and not readily available, it is preferable to have two transformers instead of one in a plant substation. For a standard ASP of conventional design, the normal power consumption is as in Table 5.21 overleaf.

There are essentially two types of transformers, Oil and Cast Resin. The oil cooled transformers are covered under IS: 2026 & IEC 60076. The cast resin transformers are covered under IS: 11171. It is an improved version compared to the oil cooled transformers up to 1600 kVA capacity. They are capable to withstand mechanical and thermal stresses caused by short circuit currents.

Table 5.21 Requirement of power for different capacities of STP (Activated sludge process)

STP capacity MLD	Power in kVA		Recommended Back-up supply (Essential loads, max 8 hours), kVA	Modular units		Approx. Daily Energy consumed kWh
	Lower value	Upper value		Number of Modules	MLD of each	
2.5	250		250	1	2.5	1,200
5	315		250	2	2.5	2,500
10	400	500	300	2	5	4,800
20	750	900	500	2	10	9,000
30	1250	1500	750	3	10	12,500
40	1500	1750	1000	2	20	16,000
50	2000	2500	1500	2	25	20,000
60	2250	2750	1800	3	20	24,500
75	3000	3500	2250	3	25	28,000

Note : The above power considers:

a) A peak load of 200% for 4 hours and 150% for 6 hours

b) A nominal lifting of 6 m at the beginning of process. If additional lifting is required suitable revision will be required.

Their cores are made with cold-rolled grain-oriented high-grade steel and copper windings. The resin is filled and cured.

The maintenance is easy due to non existence of transformer oil.

The standard accessories are lifting lugs, mounting channels, Ratings and Diagram Plate. HV and LV Bushings, Terminal Connectors and Temperature sensing devices etc.

The specifications are:

Frequency 50 Hz,

Primary Voltages: 33000/22000/11000 V;

Secondary Voltages: 230 / 430V;

Tappings: ±5% in steps of 2.5% or any % as per site requirement;

Impedance 2% to 5%;

Basic Impulse Levels: 60, 75, 95 or as per requirement.

5.12.2.9 Parallel Operation

More than one transformer can be used in a substation to share the power supply and to keep the transformer size within handling limits or to act as standby. This is termed as Parallel operation. It is usual to have two transformers of equal kVA capacity in parallel, where both can share the load, or one can be on load (on duty) and the second as standby (off duty). Even multiple transformers can be used, where one group share the load (on duty) at a time and the other group is standby (off duty). The off duty transformers are kept charged on the primary side and open on the secondary, to avoid the transformer becoming cold and to ensure that the unit is ready to take on duty at any time. Parallel operating transformers should have same voltage ratio, compatible vector group and sequence, same impedance, and usually same rating.

5.12.2.10 Instrument Transformer for Metering

These are used in substations to measure the primary current, voltages and other electrical parameters and monitoring these parameters gives an indication of the health of the system.

5.12.2.11 Instrument Transformers for Protection

These are used to measure accurately the load and system condition at any time and send control signals at any preset abnormal condition to annunciate or trip the main switchgear. The proper functioning of the protective system will save a lot of time, cost and interruptions by pre-empting a dangerous situation.

5.12.2.12 Substation Fencing and Lighting

Proper fencing with separate entry and exit gates shall be provided to prevent any unauthorised entry by persons or animals, etc. Usually chain link fencing is sufficient. The gates should always be kept under lock and the key should be in a prominent place.

5.12.2.13 Earthing and Lightning Protection

These are essential to the substation to safeguard the equipment and operating staff from any electrical shock. In addition, the earth fault sensing relays act based on the leakage current through the earth path which should have minimum resistance to current flow at such times. An underground grid of conducting mat is to be formed in the entire substation area, which is connected to several earth pits forming a low resistive path to any leakage current. Similarly, the top of the substation structures are to be connected by a wire grid to intercept lightning strokes and conduct it safely to the ground.

All the above equipments should be provided strictly as per regulations and guidelines under the Indian Electricity Act 2007, and appropriate power regulations.

5.12.3 Prime Movers - Motors and Motor Controllers

The pumps used in sewerage are driven either by electric motors or by diesel engines.

5.12.3.1 Motors

Electric motors are the most widely used and are mainly of three types, Induction (AC) motors, synchronous (AC) motors and DC motors. Amongst these, the Induction motors are the most common. Synchronous motors merit consideration when large HP, low speed motors are required. DC motors are used occasionally, especially for the speed variation duties.

5.12.3.2 Selection Criteria

The type of motor has to be selected considering various criteria such as the constructional features desired, environmental conditions, type of duty, simplicity and ruggedness of construction, endurance life cycle, capital and operating costs.

5.12.3.3 Constructional Features of Induction Motors

The most commonly used induction motors are of 'Squirrel cage' type. Normally, the starting torque requirement of centrifugal pumps is quite low for which this squirrel cage motors are suitable. Slip ring or wound-rotor motors are used where required starting torque is high as in positive displacement pumps or for centrifugal pumps handling thick sludge. The rotor is a circular cylindrical stack of laminated iron stampings with high magnetic permeability, shorted at both ends by a ring of aluminium or copper, and the stator is a cage of copper winding. The cage itself is again made of circular stampings with slots, which are aligned to allow copper winding to be inserted. The insulated windings are grouped into series and parallel sets based on the design and connected suitably to bring out the terminals. Normally, six terminals, two for each phase are brought out to the terminal box where the incoming cable can be connected, to form either STAR or DELTA connection. The rotor is positioned on bearings to rotate freely within the stator cage.

When the stator core is magnetized due to current flow in the winding a rotating magnetic field is created which pulls the rotor to rotate, following the rotating field. When power is disconnected, the stator core loses its magnetism, thereby the rotor stops slowly.

5.12.3.4 Method of Starting

Squirrel cage motors when started direct-on-line (with DOL starters) draw starting current about 6 times the full load (FL) current and in case of large motors, this could affect the system voltage and other running loads and may trip the system too.

If the starting current has to be within the regulatory limits specified by the power supply authorities, the squirrel cage motors shall be provided with a starter, which reduces the starting current.

5.12.3.5 Voltage Ratings

Table 5.22 (overleaf) gives the standard voltages and corresponding range of motor ratings.

For motors of ratings 225 kW and above, where HT voltages are 3.3 KV, 6.6 KV and 11 KV can be chosen, the choice shall be made by working out relative economics of investment and running costs, taking into consideration costs of transformer, motor, switchgear, cables and others.

Table 5.22 General motor ratings for various voltage ranges

Supply	Voltage	Range of Motor rating in kW	
		Minimum	Maximum
1 φ AC	230 V	0.3	2.5
3 φ AC	415 V	-	250
	3.3 KV	225	750
	6.6 KV	400	-
	11 KV	600	-
DC	230 V	-	150

Source: CPHEEO, 1993 N.B. When no minimum is given, very small motors are feasible. When no maximum is given very large motors are feasible.

5.12.3.6 Types of Enclosure

The type of enclosure for the motor shall be as in Table 5.23.

Table 5.23 Protective enclosures and types of environment

Type	Enclo--sure type	Description of environment
Screen-protected drip proof	IP.11	Indoor, clean(dust free) environment
Totally enclosed, air cooled	IP.44	Protected against solid objects over 1 mm (tools, wires, and small wires), Protection against water sprayed from all directions - limited ingress permitted. Indoor, dust-prone areas
Totally Enclosed, Fan cooled,	IP.54	Protected against dust limited ingress (no harmful deposit), Protection against water sprayed from all directions - limited ingress permitted.
Outdoor application	IP.55	Protected against dust limited ingress (no harmful deposit), Protected against low pressure jets of water from all directions - limited ingress.
Submersible application	IP 68	Protected against ingress of any dust while exposed and sewage or sludge while submerged

Source:CPHEEO, 1993

Ingress Protection (IP) ratings are developed by the European Committee for Electro Technical Standardization (CENELEC)/(NEMA IEC 60529 Degrees of Protection Provided by Enclosures IP Code), specify the environmental protection as an IP rating with two numbers where the first digit is the protection from ingress of solid objects or materials and the second digit is the protection from ingress of liquids (water).

5.12.3.7 Class of Duty

All motors shall be suitable for continuous duty i.e. class S1 as specified in IS: 325 and additionally, it is recommended that motors should be suitable for maximum six equally spaced starts per hour and the motor shall be suitable for at least one hot restart.

5.12.3.8 Insulation

Class B insulation is generally satisfactory, since it permits temperature rise up to 80°C. At cool places having maximum ambient temperature of 30°C or less, motors with Class E insulation can also be considered. At hot places having maximum ambient temperature of above 40°C, motors with Class F insulation shall be considered. The present practice for HT Motors is to specify with class F (155°C) insulation with temperature limited to Class B (130°C).

5.12.3.9 Margin in Brake Kilowatts (BkW)

Motors are rated as per the output shaft horsepower (Brake kilowatts, BKW) but their rating shall be selected as to provide margins over the BKW required by the pump at its operating point as in Table 5.24.

Table 5.24 Margins for motor-ratings

Required BkW of Pump	Multiplying factor to decide motor-rating
Up to 1.5	1.5
1.5 to 3.7	1.4
3.7 to 7.5	1.3
7.5 to 15	1.2
15 to 75	1.15
Above 75	1.1

Source: CPHEEO, 1993

5.12.3.10 Electrical Switchgear and Control Equipment

The electrical equipment selected shall be adequate, reliable and safe, and the adequacy shall be determined by the continuous current required for the station-load and the available short-circuit characteristics of the power supply.

The reliability depends upon the capability of the electrical system to deliver power, when and where it is required, under normal as well as abnormal conditions.

Safety involves the protection to the plant personnel and the safeguarding of the equipment under all conditions of O&M.

These three aspects shall not be sacrificed for the sake of initial economy. The electrical system shall be designed with such flexibility as to permit one or more components to be taken out of service at any time without interrupting the continuous operation of the station. A proper selection of voltages in the electrical system is one of the most important decisions that will affect the overall system-characteristics and the plant-performance. The station bus bar voltage shall be at the level that is most suitable for the pump-motors, which constitute the major part of the load.

5.12.3.11 Switch Gear

The functions of a switchgear in a power distribution system include normal switching on, normal switching off, fault-tripping operations and equipment protection. Motor-starting function can sometimes be vested in the switchgear, but only when the frequency of starting and stopping is low or in applications where the motors are of such magnitude that no other equipment is suitable.

5.12.3.12 Switchboards

Various configurations of switchboards can be used in sewerage. Due to the distances between various components, a single switchboard is not feasible. Therefore, a master switchboard, with about 8 to 10 feeders will be located near the substation. The various feeders shall branch out to major load-centres in the plant where they will terminate in smaller sub-switchboards. This will help in suitable grading of fuses and protective relays so that the entire power supply is not interrupted due to a small fault in a remote section.

5.12.3.13 Starters

A starter for electric motor is a control device to start and stop a motor. The starting, protection during starting and running, stopping and any operational control during running of the motor are handled by the motor starter. Since the motor accelerates from zero speed to full speed in a short time, and sometimes drives a heavy load during starting itself, a proper starting method is required to protect the motor from electrical and mechanical stresses and possible failure. These can be of different types, viz., direct on line (DOL), Star-Delta, autotransformer and stator-rotor. Fully automatic starters are preferable. The stator rotor starter is meant for slip-ring motor and the others are used for squirrel-cage motors. A new generation of starters termed soft starters have been developed in the past decade with either reactance coil, or thyrister circuit to give a smooth ‘ramp’ of starting torque and limit the starting current to as low as 3 times the full load current. Such starters put less strain on the motor during starting and thereby extend the motor’s life and reduce failure rate and are recommended.

The general guidelines regarding the use of starters for squirrel cage motors are given in Table 5.25 overleaf. Ingress Protection (IP) is applicable for starters too. Outdoor starters such as those for aerators and raw sewage screens shall be in weatherproof enclosure. Even flameproof enclosure (IP65) may be required where there could be methane gas exposure, like locations near digesters. All copper contacts shall be covered with heat shrinkable sleeves or petroleum jelly to protect them from the corrosive atmosphere prevalent in the STP to protect the copper parts from oxidation and resultant loose contact and damage to the connections.

Table 5.25 Guidelines for starters for squirrel-cage motors

Type of Starter	Percentage of Voltage Reduction	Maximum Starting Current	Ratio of starting torque to locked rotor torques %
DOL	Nil	6 × FLC	100
Star-delta	58%	2 × FLC	33
Auto-Transformer	tap 50%	1.68 × FLC	25
	tap 65%	2.7 × FLC	42
	tap 80%	4 × FLC	64
Soft Start	40% to 65%	3 × FLC	
Variable Frequency Drive	40% to 90%	2.5 × FLC to 5 × FLC	20 to 80

FLC= Full Load Current.

Note: As per torque speed characteristics of the motor, the torque of the motor at the chosen percentage of reduced voltage shall be adequate to accelerate the pump to the full speed.

The starting torque of a pump is usually low at about 20% to 25% of the full load torque. Therefore, even a voltage reduction of 60% to 65% during starting can accelerate the pump.

Source: CPHEEO, 1993

5.12.3.14 Capacitors

The electrical motors are inductive loads, which reduce the power factor. The induction motors work at a power factor of around 0.84 to 0.87 lag. Many power supply agreements stipulate a minimum power factor of load to avoid penalty, which is usually in the range of 0.9 lag. Therefore, for improvement of power factor to acceptable levels, appropriate capacitors shall be provided. Capacitors conforming to IS: 2834 are static units that can be located in the motor control panel or as a bank for a group of loads, or even at the substation level. Normally the capacitors are connected at the motor starter or switchgear so that it will come into the circuit simultaneously with the motor and go off the circuit when the motor is switched off.

In case of existence of backup power supply, the capacitor bank shall be connected to main Panel with automatic switching contactors to cut in or cut out capacitor units (APFC). Capacitors are provided to improve the overall power factor to around 0.97 lag. An improved power factor as near to unity as possible will reduce the reactive load, which is also otherwise metered as demand charges in a HT installation.

An overall power factor of unity can also be achieved by sensing HV side power factor and adding capacitors on the HV side. There shall be HT or LT capacitors based on the operating voltage of the motors.

5.12.3.15 Speed Control of Motors

The speed of an electric motor is dependent on the supply frequency and the number of magnetic poles created in the stator winding. It is given by the formula:

$$N = 120 \times f/P \tag{5.43}$$

Where

f = Supply frequency in cycles/second and

P = Number of magnetic poles created in the stator winding.

A normal 4-pole motor runs at a nominal speed of 1500 rpm. By selecting different pairs of poles from 2 to 8, nominal speeds between 300 rpm and 750 rpm can be obtained.

However, where the process demands a speed variation, variable speed drives are applied whereby the frequency is varied to change the motor speed. These are also called variable frequency drives (VFDs) and are made part of the motor starting circuit to act as a starting control and as a speed controller.

Other types of speed control like phase shifting type are also available. In the case of pumps in sewage duty, the change in plant load over a day may need different flow rate pumps for which variable speed motors may be considered. Variable speed pumps are efficient where the friction loss component is dominating as compared to the static lift. In most cases of STPs, the static heads are the dominant factor since the pipeline lengths are low and this reason combined with the sensitive electronic equipment associated with the power circuit and the corrosive environment make the application of variable speed motors for pump application as very few.

The variable flow can be more easily achieved by running one more pump or one less pump. VFDs are useful to control the speed of the motors controlling the surface aerators, air compressors or air blowers, as the case may be. This is required to save electrical energy to maintain only the required dissolved oxygen (DO) level at all times. An instrumentation controlled interactive automation is advisable in large installations.

5.12.3.16 Cables

The flow of power from transformer to switchgear and from there to starter and to motor and other related equipment like capacitors are through power cables and Table 5.26 (overleaf) gives guidance on these.

Consequent to progress in PVC and XLPE cable manufacturing technology, paper insulated, lead-sheathed, jelly-filled and other forms of cables are now discontinued in general and only PVC for LT and PVC/XLPE for HT cabling shall be used and are to be of aluminium conductor and armoured to protect from underground hazards. The size of the cable should be selected such that the total drop in the voltage when calculated as the product of current and resistance of the cable shall not exceed 3%.

The values of the resistance of the cable are available from the cable-manufacturers.

In selecting the size of the cable, the following points (overleaf) shall be considered.

Table 5.26 Types of cables for different voltages

No	Range of Voltage	Type of Cable to be used	IS Ref.
1.	1 phase - 230 V 3 phase - 415 V	PVC insulated, PVC Sheathed	IS: 1554
2.	Up to 6.6 KV	PVC insulated, PVC Sheathed	IS: 1554
4	11 KV 3 phase	XLPE- Cross Linked, Polyethylene insulated, PVC sheathed Cable	IS: 7098
5	1 phase – 230 V	Paper insulated, lead sheathed	IS: 692

Source: CPHEEO, 1993

a) The current carrying capacity shall be appropriate for the lowest voltage, the lowest power factor and the worst condition of installation i.e., duct-condition

b) The cable shall also be suitable for carrying the short circuit current for the duration of the fault. The duration of the fault should preferably be restricted to 0.1 seconds by proper relay setting.

c) Appropriate rating factors shall apply when laid in groups (bunched) and/or laid below ground.

d) For laying cables, IS: 1255 shall be followed.

5.12.3.17 Controls

The controls shall be simple, direct and reliable. Large pumping systems may have controls that automatically start and stop the pump-units and associated valves and auxiliaries. A proper hand-operated selector switch may also be provided to avoid over-working of any one pumping unit.

Liquid level controls generally employ floats, ceramic floats being preferred to metal floats as the latter are affected by the chemical action of the sewage.

All floats are subject to accumulation of grease and scum and shall be periodically revamped.

As a recent development, various other level sensors, such as, ultrasonic type, radar type, capacitance type, etc., are also being used.

The various functions, which a control-panel has to serve and the corresponding provisions to be made in the panel are detailed below:

1. For receiving the supply- Circuit breaker or switch and fuse units

2. For distribution- Bus bar, Switch fuses units, Circuit breakers

3. For controls - Starters, Level-sensors, Flow-sensors, Time-delay relays,

4. As protections - Under voltage, over current, earth fault and Motor Protection Relays.

5. For indications and readings — Phase indicating lamps, Voltmeters, Ammeters, Frequency meter, Power factor meter, Temperature scanners. Indicators for state of relays, Indicators for levels, Indicators of valve positions, if valves are power actuated.

The scope and extent of provisions to be made on the panel would depend upon the size and importance of the pumping station. The space clearances shall be ensured as per Indian Electricity Act, rules and regulations.

5.12.3.18 Submersible Pump Motors

The submersible pumps are widely used and are monoblock pump sets with motor enclosed in a watertight compartment and the submergence in sewage serving the cooling purpose of the motor.

Since the motor casing is continuously cooled by the sewage, the motor can work at higher current loads without being overheated. The cable connection is also through a watertight gland.

The sewage submersible pump testing shall conform to IS: 9137 and IS: 1520.

The sewage submersible motor test shall conform to IS: 325 and IS: 4029.

Some salient features of submersible sewage pump are listed below:

a) The pumps are provided with maintenance-free anti friction bearings permanently grease-filled to sustain the axial and radial forces.

b) Impeller is of non-clog design capable of handling unscreened sewage and solids up to 75 mm size without clogging.

c) The mechanical seals have silicon carbide facing, and are effective in both directions of rotation.

d) The pump has a bolt-free connection facility for easy lifting and lowering. A flanged clamp with a vertical mating face is fixed to the beginning of delivery pipe at the low level of wet well. The pump delivery has a corresponding mating clamp with rubber ring on the vertical mating face. When the pump is lowered, the two mating faces get joined with the rubber ring to seal. Fixed clamp is given a slight taper as to make the joint water-tight as the pump is fully lowered and its own weight makes it tightly sealed.

e) All fasteners are of stainless steel.

f) The motor is a class F IP 68 protection enclosure with resin-impregnated windings.

g) Normally motors up to 65 kW are dry type with cooling fines and the cooling is effected by the surrounding fluid being pumped. For higher rated motors in-built closed loop liquid (water glycol coolant) cooling system are used. A separate impeller mounted on the pump shaft itself shall effect coolant flow within motor. An in-built heat exchanger without cooling fines, like the cooling jacket shall be provided for cooling of the coolant by the pumped liquid.

5.12.3.19 Single Line Diagram

A Single Line Diagram (SLD) depicts the power circuit from the supply point to the Switchgear, and up to the individual equipment. This also indicates the tappings for metering and protection, Circuit Breakers, Transformers, Generators, indications of where cables are connected, and important rating/specifications for the power equipment. Several electrical symbols are used in the SLD for notation of the various power equipment. All the Graphical symbols used for the various electrical components are standardized through 'IEC 60617- Graphical symbols for Diagrams' which contain some 1750 symbols. A typical single line diagram is shown in Figure 5-54 overleaf.

5.12.4 Backup Power Supply

All STPs shall be equipped with diesel generators. The rating shall be able to run continuously and give power supply to all the essential equipment of the plant (e.g., pumps, aerators, laboratory, etc.) plus lighting and the location shall be as near as possible to the major load centre or near the main LT panel to which the power will be supplied. The number of units shall be split into two or three generators instead of one large generator so that in case of requirement during lean load period, only one generator may be run to cater to the reduced load. Further, any one unit can be taken up for maintenance without the risk of total absence of standby facility.

Where the raw water pumping or terminal pumping station within the STP is considerably away from the other power-intensive units, two separate standby power supply systems can also be installed, one set for the pumping station and connected loads , and another for the rest of the plant. The generator shall have its own switchboard with an incomer and one or more feeder switches. The incomer shall receive supply from the generator through an incoming circuit breaker; the feeders will supply power to the recipient switchboard. This is required to protect the generator from any fault in the recipient load and protect the downstream load from any abnormal output from the generator.

5.12.4.1 Parallel Operation of Generators

When more than one generator is to be operated to share the load, the units have to be 'synchronized' before they are allowed to share the load. The synchronization means equal voltage, equal frequency and equal phase sequence. For this, a synchroscope shall be installed in the generator panel or the coupling panel, which will indicate the right time to close the circuit breaker.

5.12.4.2 Fuel Storage

Adequate fuel has to be stocked so that the bulk fuel storage normally caters to seven day's demand of fuel. Since the diesel is inflammable, adequate precautions shall be taken for storage, protection and upkeep of the storage facility.

5.12.4.3 Cost of Captive Power

Larger the generator size, lower is the cost per unit generated. However, larger units also consume more fuel during idle running. The cost of captive power generation is usually more than the cost of power obtained from electricity supply company.

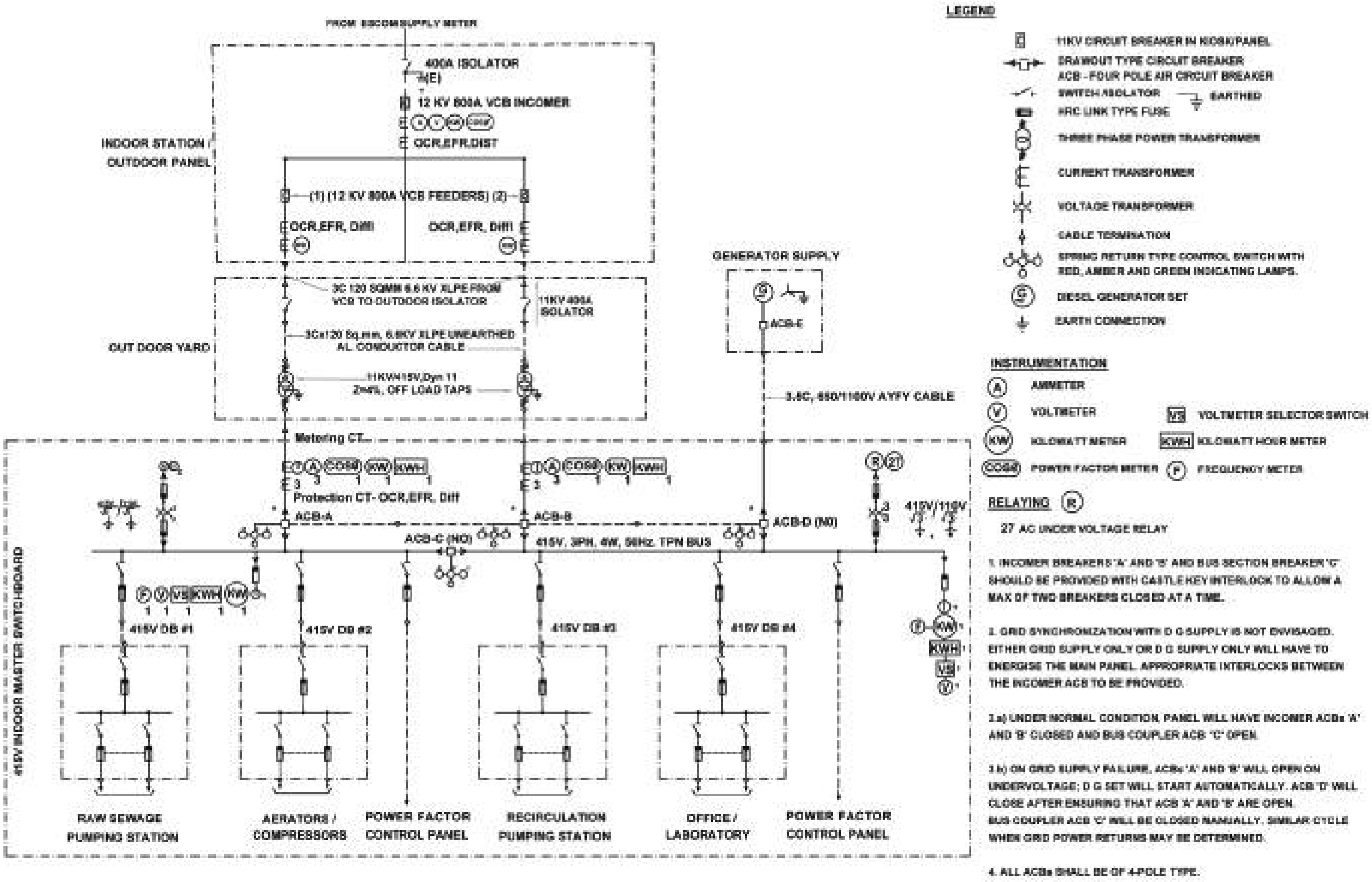


Figure 5.54 Typical Single Line Diagram of 11 KV yard and master L.T. switchboard

On an average, one unit of energy will need between 0.25 to 0.4 litres of diesel, depending on its size. In addition, the generator also needs lubricating oil, maintenance, etc., and therefore, the use of generator shall be resorted to, only when necessary.

5.12.4.4 Electricity Room and Generator Room

The Electrical Switchgear room shall be well illuminated and well ventilated. The ceiling height is stipulated to be at least 1.8 m above the highest point of the switchgear to the lowest point of the ceiling. Generally, this is not less than 3.2 m, where use of tripod is not envisaged for moving/ erection of the panels. Even higher levels of ceiling are common to ensure better air ventilation, natural lighting and equipment handling needs. The clearance between side and rear of panels and the wall shall be not less than 700 mm in case of LT Panels and 1000 mm in case of HT panels.

Ample space is required to be available in front of panels for operation, monitoring and repairs. Where draw-out type breakers are employed, the draw-out distance has also to be considered. Entry of cable ducts to the rooms should be done in such a way as to prevent water entry through the duct. The duct can be of brickwork with internal plastering and angle supports to keep cables clear of the duct floor. Multi-tier cable racks are used to lay power and control cables.

Galvanized steel cable trays are also advisable as the ducts are mostly covered and can be humid due to the low ventilation. Earthing strips from switchgear should be taken through the cable ducts with clear identification. Battery rooms should be well ventilated with exhaust fan and have acid-resistant floor and wall tiles. The operator’s table if located in the switchgear room shall be strategically located as to facilitate proper supervision.

5.12.4.5 Clearance from Statutory Authorities

For 11 KV and higher voltage substations, clearance to the layout drawings shall be obtained from The Jurisdictional Electrical Inspector of the Government before commencement of work. The written concurrence shall have to be obtained from the Electrical inspector again prior to commissioning, for compliance to the rules and safety aspects.

In case of generators, approval of the location and layout shall be obtained prior to erection and again before commissioning.

5.12.5 Automatic Mains Failure Panel

Failure of mains supply to the plant can occur at any time. In order to ensure that the treatment process is not unduly interrupted, an automatic changeover panel is installed. The automatic changeover panel is also called automatic mains failure panel (AMF) and it ensures that

a) When the main power supply is interrupted, the generator will start automatically after a certain time lapse to resume power supply to the plant.

b) When the main power supply is back, mains power supply will be resumed instantaneously after cutting-off the power from the generator. However, the engine should run for some more time on no load to facilitate the cooling down of the engine.

A set of changeover power contactors are normally employed for transfer of source between Mains and backup, ensuring that there is no faulty operation. AMF Panels are available from 15 KVA onwards. For successful operation of AMF Panel, the generator set and its controls have to be in good working condition, ready to start any time.

5.12.5.1 Sequence of Operation of AMF Panel

The sequence of operation of the AMF panel is described briefly below:

1. The mains supply source, which supplies power to the plant shall be constantly monitored by a mains voltage monitor. It will also monitor the readiness of the generator to start.

2. When mains voltage fails or drops below 70% to 80% of the rated voltage, the automatic control system shall give a starting signal to the diesel generator set.

3. The diesel engine will start. Once the diesel generator set reaches its operating speed and the alternator attains its operating voltage, a change over switching operation will occur through a set of relays whereby the mains supply switch will open and the backup supply switch will close. Thus, the load is transferred to the generator set. During this start-up and change over period, for a short time there will be no power to the plant during this operation.

4. Upon the return of the normal source voltage to the rated voltage or 90% of rated voltage for at least one minute (or any such stabilizing time), the changeover relays will activate opening of backup supply switch and closing of mains supply switch. In many cases this is done with such a short time gap that the plant runs uninterrupted.

5. The diesel engine continues to run on no load for some time. After a time-gap the engine stops. The automatic control system then resets itself and in readiness to start the engine in the case of the failure of the normal source.

6. In the event of failure of the diesel generator set to start/ deliver the power due to faulty starting within a specified period, there shall be an audio alarm. On hearing the alarm, the situation shall be investigated and remedial measures shall be taken without any delay. The changeover logic will be locked not to operate, until the generator problem is set right.

It is inevitable that all the electrical equipment will stop for a short while during the transition from the mains supply to the generator supply.

By the use of AMF Panel for automatic changeover of supply from grid supply to generator supply and vice versa, the interruption to operations is minimized. The station operator only needs to restart the components of the plant during the first changeover, and only monitor the smooth transfer during the second changeover.

A manual / automatic selection switch is also provided for manual operation in case of any problem.

The safety requirements of AMF panel are furnished in NEMA standard ICSL 2447.

5.12.5.2 Inbuilt Timers for Safety of the Operation

There are various inbuilt timers for the safety of the operation of the AMF. They are

- Start Delay or Blackout Timer: Time delay from mains failure to Diesel Genset starting
- Warm up Timer: Time delay from Genset running to load transfer on DG supply
- Mains Ok Timer: Time delay from mains sensing to load transfer on restoration of mains supply
- Cool down Timer: Time delay after load transfer on mains to DG shut down
- LOP Bypass Timer: Time delay during which LOP signal is NOT sensed
- Fault Relay Timer: Maximum time duration for sounding hooter after fault is sensed (If reset key is not pressed).

5.12.6 Uninterruptible Power Supply

The uninterruptible power supply (UPS) is an auxiliary piece of equipment, which provides back up power in electrical circuitry by drawing from a backup rechargeable battery or batteries for smaller drawals and through diesel driven generators for higher drawals. It is like an ambulance to take a patient to a hospital with oxygen and drips, etc., during the transit. Modern systems use a “double conversion” method of accepting AC input, rectifying to DC for passing through the rechargeable battery and then inverting back to AC for powering the protected equipment through a line-interactive UPS.

Most UPS below 1 kVA rating is line-interactive type where there is an additional multi-tap variable-voltage autotransformer which can boost or buck the powered coils of wire to regulate the output voltage fairly steady. However, the battery is charged only in high voltage mode but not in low voltage mode. For bigger systems, a synchronous motor/alternator can be brought in through a choke and energy stored in a flywheel. So that when the mains power trips, an eddy-current regulation maintains the power on the load as long as the energy of the flywheel can withstand.

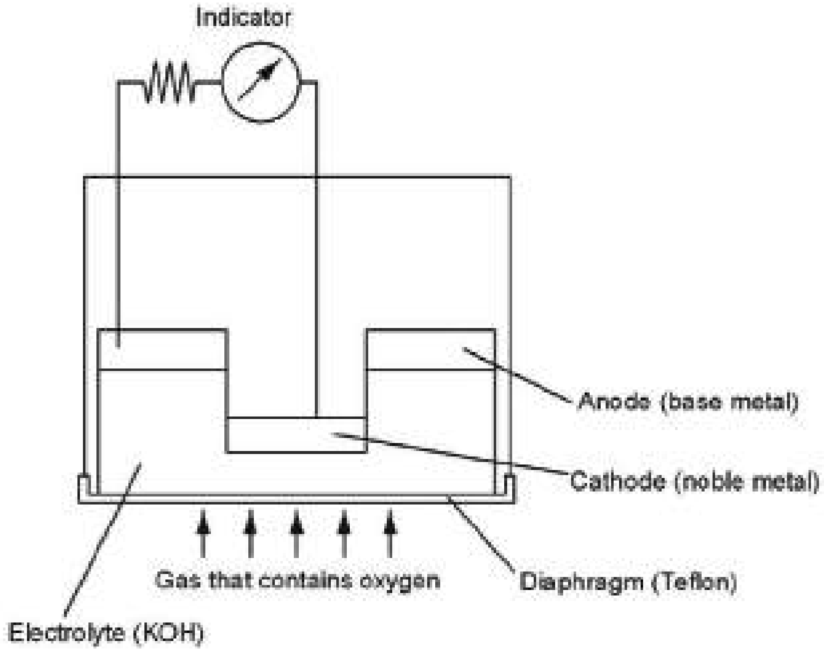
The UPS can also be combined with a diesel generator to bring on the standby power after a brief delay and is referred to as DUPS and power during this delay to start the generator supplied by another UPS. Generally, the local electricity authority has provisions to supply power by a dedicated feeder instead of supplying the power from a public distribution grid. These feeders are given to hospitals, water treatment plants and fire fighting services etc., and effort shall be made to draw power for STPs from such dedicated feeders and have duplicate feeders to ensure that power supply to the STP is really uninterruptible.

However, for personal computers, SCADA systems, etc., it is necessary to provide an appropriate line interactive UPS.

In STPs, this type of power back up shall be ensured for at least the biological aeration systems and the return sludge pumps.

5.12.7 Instrumentation Facilities

The most important instrumentation needed in an STP is for the sensing of dissolved oxygen in biological aeration tanks to make sure that the microbes do not die off for want of oxygen. This is measured through a galvanic cell controlled probe principle as in Figure 5-55 and a hand held meter with a probe as in Figure 5.56 and the probe can also be used in the laboratory in a BOD bottle.



Source: Yokogawa Electric homepage

Figure 5.55 Principle of a dissolved oxygen probe

Source: M/s YSI catalogue

Figure 5.56 Hand held DO meter with probe for field use as well as laboratory use in BOD bottle

5.12.7.1 Principle of Working of DO Probe

When the probe is dipped into the mixed liquor of the aeration tank or into the BOD bottle containing the sample, the dissolved oxygen permeates through an oxygen permeable membrane covering the tip of the probe.

Then it enters the electrolytic solution in which an anode (base metal) and cathode (noble metal) are adjacent to each other, a current proportional to the quantity of DO is generated and is measured by the electrical circuitry and pre-calibrated to display the DO concentration directly.

5.12.7.2 Advantages and Disadvantages

The advantages are the detecting system is compact and portable with the cable length between the meter and the probe being available even up to 30 m. The disadvantage is the probe has to be cleaned almost every week and fresh membrane disc replaced with new electrolyte solution and the tip of the electrodes gently scraped to remove adhesions and oxidative residues.

5.12.7.3 Standard Procedures

The meter can go on, but the probe needs frequent attention. In the field, the D O can be measured at any depth and at any co-ordinates of the aeration tank to get an idea of the uniformity of D O and in case there are very low values in a zone, it may indicate that the air diffusers might have got choked. In order to do so, the probes can be tied securely to light weight but rigid pipes and immersed at the chosen location by standing on the platform. However, in the case of aeration tanks using surface aerators, these should be used only near the sidewalls and floor near the walls as otherwise the cable may get entwined in the swirl of the mixed liquor and may even draw the operator into the tank.

5.12.7.4 Minimum Velocity

There has to be a velocity of the liquid across the probe surface to induce the required hydraulic shear. In aeration tanks this is automatically obtained. In BOD bottles, a magnetic stirring glass capsule iron needle is first dropped into the BOD bottle and the bottle mounted on a magnetic stirrer so that the liquid inside the bottle is stirred to induce the required velocity. Alternatively self stirring probes are also available which have a rotating fine brush eccentrically to the probe axis which serves to agitate. The illustrations are shown in Figure 5.57 and Figure 5.58 overleaf.

Source M/S YSI catalogue

Figure 5-.57 Setup with magnetic mixer and mixed needle for agitation

Source M/S YSI catalogue

Figure 5.58 Eccentric axis self stirring probe with the D O probe to generate velocity in the liquid

5.12.7.5 Slime Build up on Probes in Aeration Tanks

The probe if left into the aeration tank will have a tendency to build the slime onto the probe membrane and hence the probe has to be taken out and scrubbed gently and put back every day. This can be got over by using the self-stirring probes as in Figure 5.58

5.12.7.6 Instrumentation

Concerning instrumentation, the D O probe can be fixed at a desired location inside the aeration tank and the output signal of 4 to 20 mA can be relayed to the meter in the operator room and in turn can be hooked onto a desktop computer. The DO can be either measured and displayed 24 × 7 or checked at random by the operator.

5.12.8 Instrumentation Facilities – BOD

There are sophisticated BOD measuring analyzers, which use the principle of measuring the rate of initial D O depletion for about 10 to 15 minutes inside the BOD bottle. This is done by the D O probes fixed to the bottle instead of the usual ground glass corks and kept inside the incubator and thereafter extrapolating the same to the desired conventional 5 days or the recent 3 days while the incubator maintains the required temperature of 20ºC or 27ºC respectively. The extrapolation software is microprocessor based and is pre-set for a given sewage by initial calibration and recalibrated as often as needed by correlating with actual BOD values measured after the 5 days and 3 days.

These instruments help in getting a quick idea, instead of waiting for at least 3 days, if there are problems at the field STP like dropping of DO, poor settling due to sludge bulking requiring immediate adjustments of ratio of sewage to return sludge, changing excess sludge bleed, etc. These are stated to be versatile enough for direct field use. The only issue is when procured at a high cost, the repairs are to be borne by the O&M only and if these are part of 5 year procurement with replacements included, it may be worth. The portable BOD instrument is shown in Figure 5-59 overleaf.

5.12.9 Instrumentation Facilities – pH

In STPs, it will be useful to install a remote monitoring system for pH and residual D O in the biological aeration reactor subject to the day-to-day preventive maintenance against slime build up as in sub section 5.12.7.5.

Source: M/s HANNA

Figure 5-59 Portable BOD test instrument

5.12.10 Non-instrumentation Method of Quick BOD Measurement

The COD test can be completed in two hours with a one-hour reflux and a graph can be constructed for a given sewage showing the COD Vs. BOD at various stages of treatment and a COD reading obtained after two hours can be used to correlate the likely BOD values for a quick field check.

5.12.11 Flow Measurement

5.12.11.1 Magnetic Flow Meters

Magnetic flow meters work on the principle of electromagnetic induction. The induced voltage generated by an electrical conductor in a magnetic field is directly proportional to the conductor's velocity. Thus, the sewage is the conductor and these meters are suitable at almost all piping like, raw sewage, settled Sewage, primary sludge, return activated sludge, waste activated sludge and treated sewage. These are non-invasive and used in almost all pipelines, but of course, initial calibration is needed. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system.

5.12.11.2 Ultrasonic Flow Meters

When ultrasonic impulses are released onto a pipe surface carrying sewage, the impulses are deflected along the flow direction based on the velocity of the flow before they impinge on the opposite sidewall of the pipe. The time taken is measured and is correlated to the velocity and then to the diameter of the pipeline and hence the flow rate is arrived at. Like magnetic flow meters, these are also non-invasive and used in almost all pipelines but of course, initial calibration will be needed. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system.

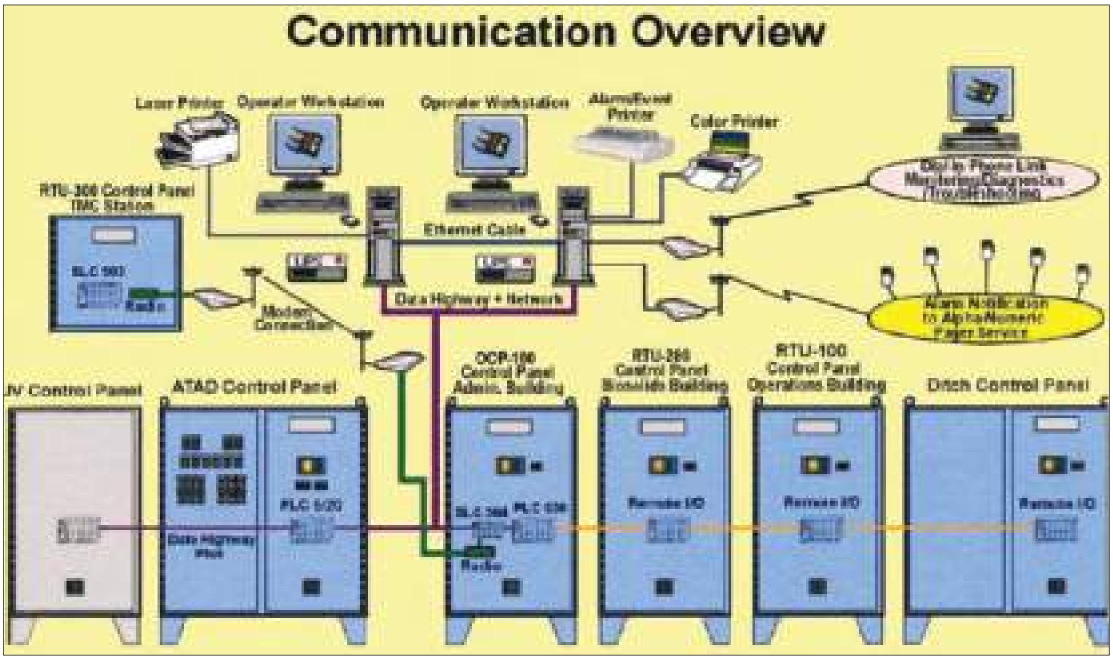
5.12.11.3 Ultrasonic Level Sensors

These also work on the same principle as above and the time taken to reach the water surface and get back to the sonic emission probe mounted on top of a channel is used to measure the depth of the liquid surface. By integrating with the depth of the floor of the channel from the probe, the depth of sewage flow is arrived at. These are useful in Parshall flumes in raw sewage channels. The output is the standard 4 to 20 mA signal, which is relayed to the central monitoring system. This signal from ultrasonic level sensor can be fed to a microprocessor, which can be programmed to give output as flow in an open channel like the Parshall flume.

5.12.12 Supervisory Control and Data Acquisition System (SCADA)

SCADA is an acronym for Supervisory Control and Data Acquisition. This presents the data as a viewable and controllable system on the screen of a computer. The data thus collected can be stored and analyzed for optimization of the process and for better real time process control. This assists plant-operating personnel by monitoring and announcing off normal conditions and failures of equipment. This allows the operators to perform calculations based on the sensor inputs. Using the stored data daily, weekly and monthly reports can be prepared. It also allows the operator to know the state of a process and an alarm associated with it. It is also possible for an authorised operator to intervene and operate equipment at a remote location through a remote terminal unit (RTU) under this SCADA network.

A typical SCADA communication overview is shown in Figure 5.-60.



Source: Kruger

Figure 5.60 Typical SCADA communication overview

5.12.12.1 Components of SCADA System

The various components of a SCADA system are

a) Personal computers: These are used by the operator to view the data acquired and allow the operator to control and improve optimization.

b) Programmable Logic Controllers: They control the outputs based on the inputs being monitored in the required sequential steps and it also communicates with the personal computers.

c) Modems: They are used to transfer data from the sewage-treatment plant site, to the centralized control station.

d) Remote terminal units (RTU): They allow the central SCADA to communicate with the various instruments at the sewage treatment plant. It controls, acquires and transfers data from the process equipment at the site, in conjunction with the central SCADA.

5.12.12.2 SCADA Software

There is standard SCADA software available which can be installed in application servers at the plant site, but they should be capable of controlling and monitoring the various instruments. The data acquired from the RTU should be displayed in the SCADA screen and the logs of each site station measurement should be transferable using data export to data base processing software like Oracle, Microsoft Excel, etc. It should also support internet connectivity for data transfer.

5.12.12.3 SCADA Security Level

In order to prevent misuse and to restrict access to a site station measurement there should be privileges to the various users of the SCADA. Typically, there are three levels, which are (1) Operator Level, (2) Engineer Level and (3) Manager Level.

5.12.12.3.1 RTU Security Level

The RTU communication port should have a configurable access level for its security. The minimum access levels are described in the following sub-section. These access levels are required to control read and write access to that port. Hence, once all the RTU ports are configured with these varying access levels depending on the requirement, then it becomes secure against unauthorised usage. It can be re-configured only after unlocking the RTU common port.

5.12.12.3.2 Minimum Access Levels of Programming and Configuration Interface

The interface shall have the minimum varying levels of user access that can be configured by the system controller. They are:

a) Unlimited Access: This will allow the user to read and write all RTU configuration parameters such as local, network and system registers, Hardware input and output registers, event logs and logic programs.

b) Access without configuration: This will allow reading and writing of all RTU configuration parameters except system registers and ladder logic.

c) Access limited to only reading the RTU parameters

d) Access limited to RTU port configuration.

5.12.12.4 SCADA User Interface

The SCADA system shall permit the user to access displays via printing device and/or soft key menus with a choice of function keys, cursor, control keys or any key on the keyboard. The system shall support operator access to multiple displays at one time, including split screens where the operator may view more than one process area at a time and permit pop-up displays.

The operator shall be able to have access to context sensitive help at any time during operation of the system. The operator shall be able to access multiple data sources/items with a single tag name.

5.12.12.4.1 Command/Control Functions

The system shall allow the user to control a specific set point or to adjust a set of points depending on the operating limits. Control of individual set points shall be enabled based upon a user's security level.

5.12.12.4.2 Display Capability

The system shall allow the user to view animated graphics for process templates including valves, meters, etc. The system shall support the capability for the operator to view scanned images and be possible to animate these images.

5.12.12.4.3 Text Description

The system shall support use of true-type scale able fonts that may be scaled according to the desired size of the text. The fonts shall be loaded by the operating system.

Text shall be able to blink based upon any user definable condition occurring in the system such as an alarm on a particular set point.

5.12.12.5 Alarm Capabilities

5.12.12.5.1 Alarm Display Capability

The system shall support alarm display capability on the display. Current alarms shall be available as an alarm summary object and a chronological summary of alarms shall be available.

It shall be possible to inform the operator of an alarm condition via an audible tone, a pop-up display or any combination of animation types on the screen.

Alarm acknowledgement may be performed on all alarms, alarms in a single group, and alarms in a collection of groups in defined in alarm group hierarchy or on a point-by-point basis.

5.12.12.5.2 Alarm File Capability

Alarms shall be logged to a file for future viewing or review of alarm history data. The user shall have the capability to review the file for cause and event analysis. The alarms that are logged shall be configurable from a choice of the parameters listed during configuration.

5.12.12.5.3 Alarm Printing Capability

Alarms shall be allowed to be printed and the format shall be configurable and made up of any of the parameters listed during configuration.

5.13 DISTRIBUTED CONTROL SYSTEM

5.13.1 Description

For a fairly large sewerage system with many pumping stations, STPs, etc., it is possible to have a centralized control station to monitor, data logging, interfere and control the various operations by a Distributed Control System (DCS), with a network of PLCs and RTUs.

The DCS developed in 1975 was first used by pulp and paper mills and is rather widely used off late. It consists of distributed microprocessor based single loop controllers connected to a shared video-based operator station. It has substituted analogue controllers with Direct Digital Control (DDC). The video based operator station eliminated the need of large instrument panels, which was important prerequisite for operators.

DCS is the best choice for a system with many operational units (say, sewage pumping stations) at various locations away from the main control station (say, at the STP), linked through various methods of digital communication. However, this is complex and expensive, but provides various controls, functions and other reliability features.

Presently, DCSs are available in miniaturized versions for multitasking, multivariable, multi-loop controller used for process control. It is a functionally and geographically distributed system. Equipment making up a DCS is separated by function and is installed in multiple working areas.

The operator can view information transmitted from various units and displayed on a video display unit (VDU) and can change control conditions from a computer key board.

5.14 PROGRAMMABLE LOGIC CONTROL

5.14.1 Description

For a single operating unit (say, sewage pumping station), the Programmable Logic Controller (PLC) is the very vastly employed automatic controller, helpful in eliminating continuous human monitoring and control.

Various matrix of operations are pre-programmed with various inputs from instruments, such as level transducer, pressure transducer, flow transducer, motor bearing temperature monitor, vibration monitor, etc.

PLC is to change, than re-lay panels. This would reduce the installation and operational cost of the control system compared with the electro-mechanical relay system. PLC offers the advantages such as, ease of programming and re-programming, programming language is based on re-lay wiring symbols familiar to most operating personnel, high reliability and minimal maintenance, small physical size, ability to communicate with other computer systems, moderate low initial investment cost and availability of modular designs.

5.15 CORROSION PROTECTION AND CONTROL

5.15.1 General

Corrosion is the phenomenon of the interaction of a material with the environment (water, soil or air) resulting in its deterioration. There are many types of corrosion, the major types being galvanic, concentration cell, stray current, stress and bacterial.

Sewage collection and treatment systems are more prone to corrosion in view of the nature of the sewage. Raw sewage contains solids like grit that cause abrasion in sewers, pumps and their components, thus removing the protective coatings. This exposes the metal and accelerates the corrosion process. Hence, corrosion control becomes important in SST. It is particularly severe in areas where sewage strength is high, sulphate content of water is substantial and average temperature is above 20ºC. The corrosion problem in SST can be categorized as (1) Corrosion of material of sewers and (2) Corrosion of metal and concrete in treatment plants. The corrosion of sewers and relevant prevention and control are described in detail in section 3.69 in this manual. Corrosion in case of treatment systems is described below.

5.15.2 Corrosion of Treatment Systems

The important units from the corrosion point of view are civil tanks for raw sewage collection, clarifiers and sludge digesters. In addition, all metal parts like in the pumps, valves, screens and grit chamber equipment are also important.

5.15.2.1 Raw Sewage Collection and pH Neutralization

In general, raw sewage pH will always be near neutral in pH and is not corrosive. Large variations in acidic or alkaline pH can result only when huge industrial wastewaters get into sewers. The manual clearly recommends in section 3.8 that allowing industrial effluents into sewers shall be discouraged. If such a contingency is foreseen, it may be only for a brief period before the entry is detected and cut off. In such cases, sodium hydroxide shall be used if the pH is corrosive and hydrochloric acid shall be used if the pH is alkaline. The neutralizing chemicals would need to be stored in acid or alkali resistant containers and the solutions led to the neutralizing tank by non-corrosive, thick walled PVC piping.

5.15.2.2 Clarifiers

The floor bottom of the clarifiers is scraped by mechanical scrapers in order to divert the sludge to the central sludge pit. These scraper arms and the squeegees are constantly immersed in sewage and are not subjected to severe corrosion because they are not exposed to the air.

The specification for the steel used for the underwater mechanisms should be carefully drawn to ensure maximum protection from corrosion. It is normally specified that all the steel below liquid level shall be at least 6 mm thick. It is a good practice to keep all chains, bearings or brackets above the liquid surface. All castings in the drive mechanism should be of high-grade cast iron. It is also possible to give cathodic protection to the scraper mechanism either by sacrificial anode or by impressed current. The choice of either of the method will depend upon the comparative costs. In any case, the cost of such a protective measure will not be higher than the cost of good quality acid resistant paint.

5.15.2.3 Sludge Digestion

Corrosion problem in sludge digestion tank is described in section 6.11.1 in this manual.

5.15.2.4 Activated Sludge

In the activated sludge plant, oxygen is provided to the sewage either by surface aeration or by compressed air system. In the compressed aeration system, air supply is normally provided through mild steel pipelines. Though the air is filtered and moisture is removed before sending to compressors, still there can be problems and these can be minimized by use of air supply pipelines of non-corrosive material.

In surface aeration, proper material selection and coating are necessary for protection of the exposed parts of the aerator blades. It may be mentioned here that the protective coating has to be applied at regular intervals, since it is found that such coatings have very short life. PVC lining may not be easy to provide due to the shape of rotor while fibre glass lining can be relatively easier. For floating aerators, it is desirable to have corrosion resistant lining, such as fibre glass for the floats.

5.15.2.5 Attached / Fluidized / Immobilized Media Systems

In these systems, the mechanical components include the header, the distribution arm and the distribution nozzles. The header and the distribution arm are normally of mild steel and should be protected from corrosion by proper painting.

The corrosion and the resulting blockage of distribution nozzles are of common occurrence. This can be avoided by selection of corrosion resistant materials such as brass or PVC for nozzles.

5.15.3 Sewage and Sewage Pumps

For pumps and pumping equipment, proper materials selection is of paramount importance. The pump casing is normally of close-grained cast iron capable of resisting erosion of abrasive material in the waste. For handling sewage and other corrosive wastes, the impeller is generally made of high-grade phosphor bronze or equivalent materials. The wearing rings for impeller should be of good corrosion resistant materials such as bronze. The shafts are normally made of high tensile steel and replaceable shaft sleeves are recommended.

For pump and pumping equipment, painting is the usual protective measure. Both, the interior and exterior surfaces of pumps should be painted after rust, scale and deposits are removed by sand blasting, wire brushing or rubbing with sand paper

5.15.4 Preventive Maintenance

It will be seen from the above that anti-corrosive paints, coating and linings have to be used in various equipment to prevent corrosion. The paints, coatings and linings require periodical renewal, Proper maintenance demands that a schedule be drawn up so that the operator may abide by it and undertake repainting or cleaning at appropriate intervals without waiting for corrosion to become obvious.

5.15.5 Piping Requirements in Treatment Plants

Piping requirement in sewage treatment plants range from sewage and sludge conduits, drains and water lines to chemical process piping, if any. Materials for various pipeline applications are given in Table 5.27 (overleaf). In order to facilitate identification of piping, particularly in the large plants, it is suggested that the different lines shall be colour-coded. The contents and direction of flow shall be stencilled on the piping in a contrasting colour. Typical colour code appears below (WEF, MOP8, 2010):

– Orange: energized equipment and flammable gas lines
– Blue: potable water
– Yellow: chlorine
– Black: raw sludge
– Brown: treated bio solids
– Purple: radiation hazards
– Green: compressed air
– Jade green: non-potable process or flushing water
– Grey: sewage
– Orange with blue letters: steam
– White: traffic and housekeeping operations
– Red: fire protection equipment and digester gas lines

5.15.6 Modification of Materials

Normally, choice of materials shall be suitable under the circumstances likely to be encountered and commensurate with economy. If justified economically, corrosion resistant construction material can be used initially, as this may not require any additional protective coating frequently.

Stainless steel, aluminium and plastics are examples of materials of this nature. It is possible that the use of such corrosion-resistant materials would be cost-effective in the long run.

However, in treatment plants, it is found that it is usually less expensive to use ordinary structural steel to which protective coatings are applied.

Table 5.27 Piping Materials in Sewerage and Sewage Treatment

Typical Application	Concentration in %	Materials
Influent	0.5 to 2	C, CL, RCP, RC, VC
Secondary Solids	0.5 to 2	C, HP, PE, CI
Primary solids	0.2 to1	C, G, T, HP, PE, D, CI
Thickened Sludge	4 to 10	C, HP, PE, T, CI, D
Digested Sludge	3 to 10	C, HP, PE, T, CI
Chemically treated sludge	8 to 26	C, HP, PE, H, CI
Dewatered sludge	8 to 25	C, CI
Heat Exchanger	< 0.1	S
Spray irrigation	< 0.1	C, CI, T, A, HP, PE
Chemical Process Piping		C, CI, S, G, T, HP, PE, H, D
Aluminium Sulphate	15 to 22	C, D, H, HP, PE, T, S
Calcium Hydroxide	63 to 73	C, CI, D, G, H, HP, PE, S, T
Calcium Hydroxide	85 to 99	C, CI, D, G, H, HP, PE, S, T
Sulphuric Acid	93	S, G
Ferric Chloride	59 to 98	C, H
Sodium Hydroxide	73	C, S, H
Carbon Slurry	20 to 30	G

C - Carbon Steel
G - Glass Lined
T - Teflon lined
PE – Polyethylene
CI - Cast Iron
H - Plastic or rubber hose
RC - Reinforced concrete
S - Stainless Steel
A - Aluminium
P - High Density Polyethylene
D - Ductile iron
RCP- Reinforced Plastic mortar
VC - Vitrified clay

Source: CPHEEO, 1993

5.16 REHABILITATION OF SEWAGE TREATMENT FACILITIES

5.16.1 Criteria for Rehabilitation of STPs

In simple terms, the word rehabilitation refers to a situation that needs cure. Thus, rehabilitation shall be specific to non-performance and not a wholesale rehabilitation. Rehabilitation may not include capacity upgradation, but it can be other way about. In any case, except for the electrical control panels and cabling, all other rehabilitation efforts in STP should be taken up inter-alia between all unit operations, process technology and mechanical equipment all at the same time, instead of piece meal. Instances like the burn out of a motor winding are exceptions to be attended immediately and will come under the terminology of repairs and not rehabilitation. To this end, the primary responsibility of deciding whether it is rehabilitation or repair or upgradation is to be owned up at designated levels of authority before proposals are initiated.

As the overall administration of the STP is a complex discipline involving academicians, technocrats and financial managers, it is necessary to bring about the programme under a joint responsibility as in Figure 5.61 overleaf.

5.16.1.1 Upgradation of Facilities

Upgradation of STPs would normally arise in regard to capacity upgradation and there are technological strategies in hand in the present context. Some possible options are:

1. A conventional ASP can be upgraded without increasing the footprint by opting for an MBBR to be inscribed in the aeration tank and duplicating the hydraulic piping and pump sets and the primary clarifiers can be modified as rim flow clarifiers and secondary clarifiers inscribed with tube settlers.

2. The effluent quality of the existing ASP can be upgraded by adding tertiary coagulation and/or sand filtration, ultrafiltration.

3. Existing UASB can be upgraded by dismantling final polishing units and replacing it with activated sludge process or its variants such as SBR (A A Khan et al., 2012), SAF, MBBR, etc.

4. Stabilization ponds can be upgraded to aerated lagoons on extended aeration mode with bund partitions, to carve out the sedimentation zones.

5. Aerated lagoons can be upgraded with bio-towers inscribed at one end and boosting the delivery head of pump sets or replacements.

6. Large diameter old trickling filters can be upgraded into conventional ASP by constructing a radial wall and allowing the mixed liquor to flow annularly and equipped with floating surface aerators.

7. Conventional aeration tanks can be partitioned to change the process to contact stabilization and maximize treatment capacity for the same volume. The partitioning has to be by inscribing a separate structure for contact aeration preferably circular and without opening out the reinforcements of the old tank base slab or sidewalls.

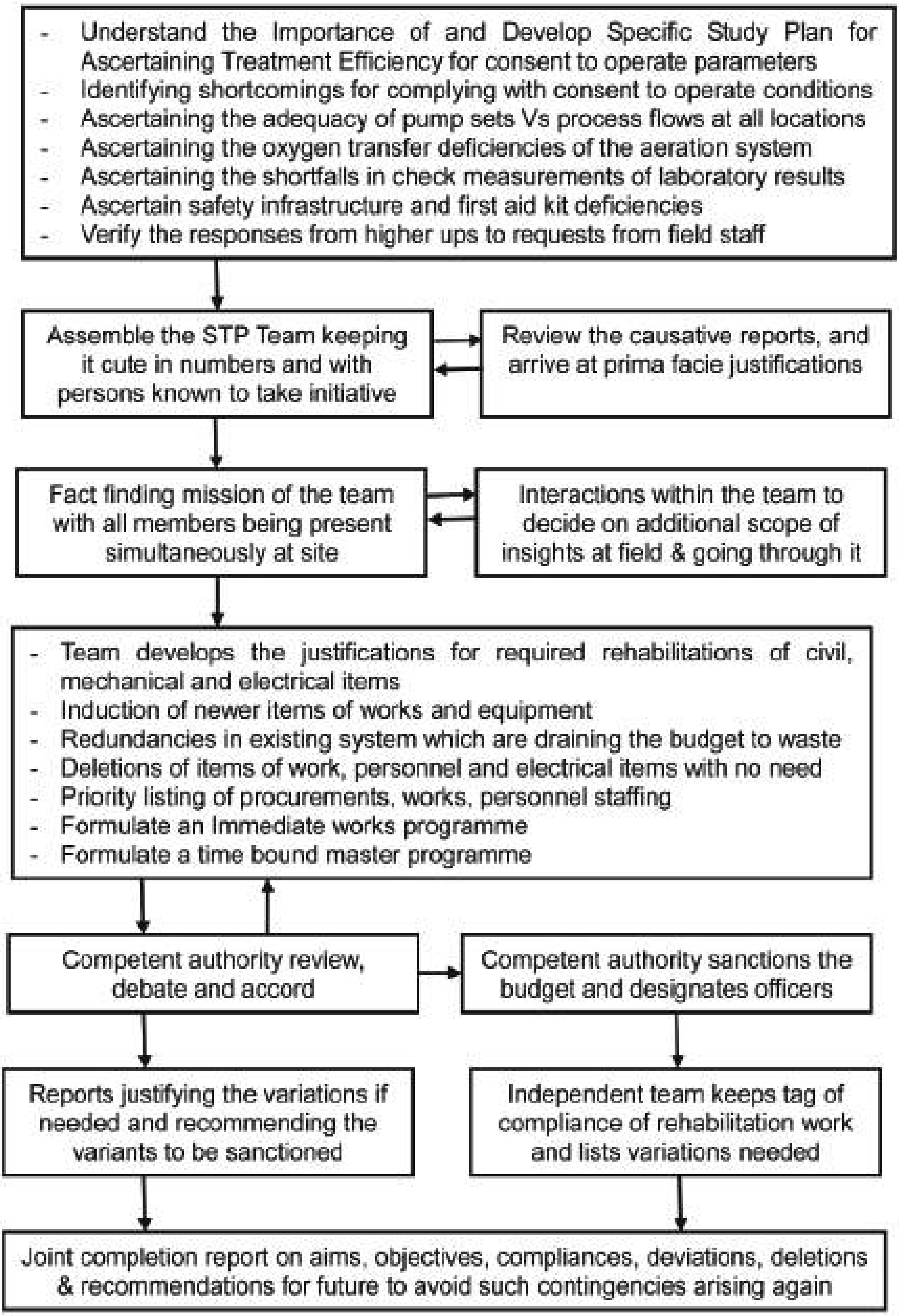


Figure 5.61 Algorithm for rehabilitation programme of STPs

8. The Melbourne Eastern STP upgrade is often cited in literature. It is from a conventional primary-aeration-secondary to online add on tertiary by ozonation-biological filters-ozonation-UV-chlorination before selling the reclaimed water to retailers who in turn sell it to end users. The biological filters used here are an unique component to biodegrade the residual organic matter and reduce ammonia, oil and grease, foam, litter and solids. The final water is to be used in farms, market gardens, vineyards, golf courses and sporting grounds in the city and at the plant, for every day operations to clean screens, to wash down work areas, for cooling and to water the grounds. The excess treated sewage will be discharged to the ocean where water contact sports will continue to take place. The significance of this to the Indian situation lies in the residual levels of phosphorous and nitrogen because of the higher concentrations in sewage due to lower per capita water supply and hence, mere add on units like the above may not be adequate and specific nutrient removal enhancements will also be needed.

9. Each situation needs to be approached individually and the question of following another location’s experience has to be tempered with reasoning.

10. The other issue is the procurement of the recent patented technologies, which of course have to be secured against reasonable competition anyway.

5.16.1.2 Energy Saving Measures

In the case of new STPs, it is preferable to design the treatment process to include primary treatment and secondary treatment with an F/M ratio in the conventional regime whereby the blended sludge becomes available for biomethanation in digesters and generation of electricity there from. This saves the electrical power needed for the STP. The 40 MLD ASP at Chennai is reported as performing on these lines. The economics of such gas generation and utilization is influenced by the capacity of the plant and the raw sewage BOD and VSS.

Hence, in the preparation of DPRs, proposals for such utilization of electrical generation should be carefully weighed by a net present value (NPV) of the costs of (a) Hydrogen Sulphide stripping, Gas/ dual fuel engines, repairs and renewals of these additions, their inherent O&M and electrical power needs, establishment costs and (b) cost of electricity that will otherwise be payable to the local electricity authority over the design period cited in Table 2.1.

5.17 CARBON CREDIT

This term qualifies the holder to emit one ton of carbon dioxide into the atmosphere and is awarded to institutions or countries that have reduced their greenhouse gases below their emission quota, which literally means emission standards. These carbon credits can be traded in the international market at their current market price. There are firms that have earned carbon credits and offer them to other firms who are interested in lowering their carbon emissions on a voluntary basis. They purchase the credits from an investment fund or a carbon development company that has aggregated the credits from individual projects. Buyers and sellers can also use an exchange platform to trade, such as the Carbon Trade Exchange, which is like a stock exchange for carbon credits. The global and Indian position are presented in Figure 5.62 and Figure 5.63 overleaf.

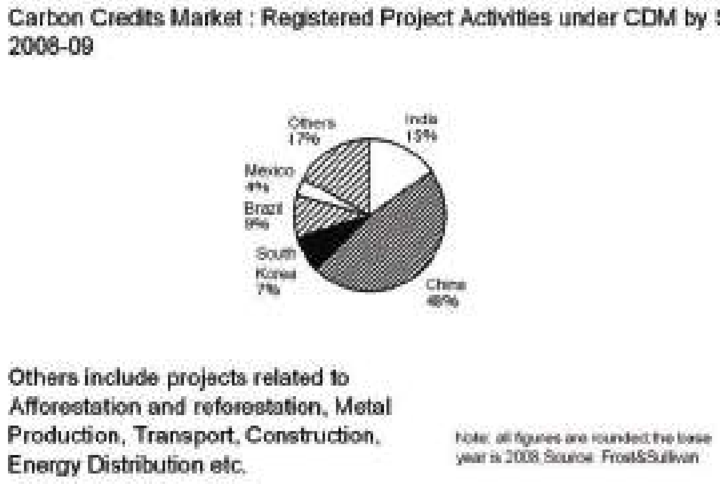


Figure 5.62 Country wise carbon credit market as of year 2009

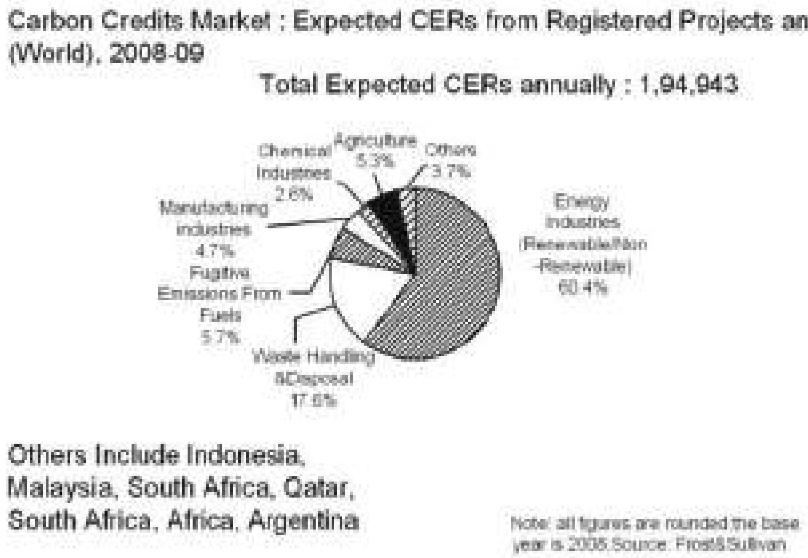


Figure 5.63 Project wise carbon credit market as of year 2009

India has the second-highest carbon credit transacted volumes in the world. India is the pioneer in the biomethanation from STPs as demonstrated in the Dadar STP in Mumbai back in 1970s itself. The gas was directly piped to nearby institutions for their fuel and revenue was earned. The availability of duel fuel (diesel as well as biogas) as well as exclusive gas based engines came up subsequently in the 1980s and the generation plant was put up in Okhla STP in Delhi. With passage of time, the STPs have, by now perfected the technology of biomethanation and generate electrical energy and thus have accumulated the carbon credits.

Surat is perhaps the largest producer of biomethanation from STPs and treats about 600 MLD, out of which 3.5 MW of power is reportedly generated. This translates to 50,000 carbon credit units per year.

The magnitude of the carbon credits programme can be understood from the news article that India has bagged the world's largest carbon credit project that will help replace 400 million incandescent light bulbs with compact energy saving (CFL) bulbs at cheap prices in a year while preventing 40 million tonnes of carbon from entering the atmosphere annually.

The project will allow the government, investors, distribution companies (discoms) and CFL manufacturers to sell CFLs at Rs.15 each, instead of the Rs.100 they currently cost on an average, and have been approved by the UN under the global carbon credit scheme called Clean Development Mechanism. Thus, the need to plan STPs with inbuilt biomethanation and energy recovery is imperative. An illustrative comparison of the potential for energy recovery and the power actually needed for a STP with conventional ASP is shown in Table 5.28.

Table 5.28 Estimated and actual achieved electrical energy recovery

Design Flow	40 MLD
Primary Sludge - Suspended Solids	450 mg/l
Primary Sludge VSS as %	60%
SS Removal Efficiency	60%
Quantity of Primary Sludge	10890 kg/day
Percentage of VS in Primary Sludge	60%
Quantity of VS in Primary Sludge	6480 kg/day
Quantity of excess sludge at 0.35 kg/kg of BOD removed	3150 kg/day
Percentage of VS in Excess Sludge	85%
Quantity of VS in Excess Sludge	2677.5 kg/day
Total VS	9157.5 kg/day
VS Destruction Efficiency	55%
Bio-Gas Yield	0.75 Ncum/kg VS destroyed
Bio-Gas Generated	3777 m^3/day
Calorific Value	6.0 kWh/cum
Power Generated by Gas Engine	7931 kW/day
Power Required	7850 kW/day

Average of past two years

Bio-gas Generated	3200 m^3/day
Electrical Energy Generated	6600 kWh

It may be seen that as long as the designed sewage flow and the designed raw sewage BOD are available, the plant has the ability to not only be self-sufficient in power, but also capable of generating additional energy for nearby institutions and sell at an unfailing pattern and earn revenue as well. The caution needed at the time of design will however be, to opt for minimum of two gas engines and install only one to start with until the sewage quantities, qualities and biomethanation kinetics are established. After one engine is established, the second engine can be suitably sized to exploit extra power production if it becomes possible and make it a commercial proposition by feeding the local electricity grid, instead of drawing from it.

The 40 MLD conventional ASP at Chennai has been built with biogas utilization facilities by way of digester gas collection, wet scrubbing and dual fuel engine.

5.18 RECENT TECHNOLOGIES IN SEWAGE TREATMENT

There are many new technologies emerging for the treatment and reuse of sewage and sludge.

The objectives of sewage treatment are

(a) to metabolise the organic matter so as to produce an effluent which can be disposed in the environment without causing health hazards or nuisance and

(b) to produce a sludge which can be used as a soil filler if it comes out of a biological treatment or as a soil-sludge immobilized product like walkway paver blocks or compound wall bricks and thus preserve the environment from dumping waste products.

The discharge standards are formulated and specified for each case by the statutory authority of Pollution Control Board (PCB). Where there is reuse, the PCB requires a zero liquid waste discharge, which as far as the sludge is concerned, addresses its reuse or secure landfill.

Thus, the degree of treatment is dictated by the type of end use and the chemical characteristics specific to these.

Thus, there cannot be a fixed type of treatment technology for a reuse situation even within the same category of industries or situations. It is here that the emerging trends in technology play a crucial part and come in handy.

For example, an industry in a land locked area may find a roof top MBR as the best choice in view of its most precious land area at ground level which it can use for increasing the production, whereas for a municipality, such a concept does not arise in the first place.

However, even for the publicly funded STPs, the recent trend is to include nutrient removal and this has brought up a recent trend in extending well beyond conventional secondary treatment.

Thus, it is necessary to recognize and understand the emerging trends. At the same time, it needs to be also recognized that an emerging trend in India might have long been a standard trend elsewhere. At the same time, India cannot plumb for a trend of technology merely because it is in use elsewhere, especially in view of the introductory remarks in this Chapter 5 explaining why such blanket adoptions need local validation.

Thus, for the purpose if this chapter, any technology that is working successfully for more than 5 years on the same scale or larger in other countries, than at which it is intended to be used here in India and has potential to be sustainable in the Indian context, is considered as emerging trends in this section.

5.18.1 Objective Oriented Emerging Technologies

The type, objectives, process name, and outline of new sewage treatment technologies are summarised in Table 5.29 overleaf.

Table 5.29 Types of sewage treatment technologies

Type	Objective	Process	Description
Tertiary	Nitrogen (N) removal	Recycled biological Nitrification/ Denitrification process	Concurrent with carbonaceous BOD removal nitrification is achieved by additional oxygen input and the mixed liquor and return sludge are recycled first into an anoxic tank receiving the raw sewage where the nitrates are denitrified by the microbes and partial BOD removal and almost complete removal of nitrates are obtained thus eliminating nitrogen altogether.
		Wuhrmann Process	This is a single sludge nitrification system with a downstream anoxic reactor. The influent enters into the aerobic tank where nitrification develops together with BOD removal and the nitrified mixed liquor passes to the anoxic reactor where the sludge is kept in suspension by moderate stirring, but no aeration. The denitrification takes place by microbes in their quest for oxygen. The classical difficulty here is the food requirements of the microbes which has otherwise been already removed in the aerobic reactor and hence, this process is suited only for sewages with little incoming nitrogen.
		Step-Feed Multistage Biological Nitrogen Removal Process	This process has been developed to enable efficient nitrogen removal from sewage without a major renovation The unit which consists of an anoxic tank and an oxic tank is arranged in several serial stages. The primary effluent is split and fed equally into an anoxic tank of each stage, and BOD load to MLSS in each stage is equalized. This method enables efficient nitrogen removal and easy maintenance of the process. The nitrified liquor internal recirculation from oxic to anoxic tank is carried out as needed.

Type	Objective	Process	Description
		Ammonia oxidation	Chlorination to mono, di & tri chloramines eventually releases nitrogen gas into atmosphere. It is a simple process, but the storage and handling of chlorination are a real challenge.
		Ammonia Stripping	Ammonia in raw sewage is present as ammonium carbonate and is a dissolved salt. By raising the pH to near 9.3, this is de-ionized and the ammonia becomes dissolved ammonia gas. It is then stripped in counter current towers where the sewage is sprayed from the top and air is blown from the bottom whereby the three phase mass transfer takes place and the resulting air ammonia gas mixture rises into the plume. The air volume and plume velocity are adjusted to keep the released ammonia concentration within threshold limits .
		Ion Exchange	Clinoptinolite is a resin occurring in some parts of the world which has the ability to exchange the ammonium ion and has applications in very small units needing cleaner operations and where secure landfill of spent resin is possible.
	Phosphorus (P) removal	Anaerobic-Oxic Activated Sludge Process	Under controlled conditions of anaerobiasis as absence of oxygen, luxury phosphorous uptake by the microbes is reported, but the mechanism is not fully understood. However, simultaneous denitrification can also occur from return activated sludge. Process operation with a plug flow regime & non-aerated but gently mixed zone in the first part of the activated sludge are stated as critical conditions to remove considerable phosphorus. However, the process control is influenced by varying phosphorous concentrations and sometimes it may be necessary to chemically precipitate the residual phosphorous by chemical addition.

Type	Objective	Process	Description
		Chemical precipitation	Aluminium and/or Ferric salts precipitate the phosphorous. This is easy to control.
	(N) & (P) Simultaneous removal	Bardenpho process	In this four-stage process, the first two stages are identical to the Modified Ludzck Ettinger (MLE) process of an anoxic zone followed by an aeration zone with a nitrate-rich recycle from the aeration to the anoxic zone. The third stage is a secondary anoxic zone for further denitrification to the portion of the flow that is not already recycled in first two-stages. A source like Methanol may be added to this third stage for microbial carbon. The fourth and final is a re-aeration zone to strip any nitrogen gas and increase the DO concentration.
		Anaerobic-Anoxic-Oxic Process	This process is a combination of the biological phosphorus removal process and the biological nitrogen removal process. It consists of tanks arranged in the sequence of anaerobic tank, anoxic tank and oxic tank. Influent and return activated sludge flow into the anaerobic tank while nitrified liquor is recycled with a circulating pump from the oxic (nitrification) tank to the anoxic (denitrification) tank. Ammonia nitrogen is oxidized to nitrite or nitrate in the oxic tank, and then nitrite or nitrate is denitrified to nitrogen gas in the anoxic tank. Aluminium and Ferric salts are added to chemically utilize the ionized Aluminium or iron & precipitate the phosphorous. This is a straightforward one for control.
Secondary	Polishing BOD, SS, Pathogen removal	Soil aquifer treatment	The soil organisms bring about further microbial activity is polishing these parameters and is a slow long term process. It is unique to the soil conditions, climatology, precipitation, flooding and inundation that may bring in agricultural residues of insecticides & pesticides.

Type	Objective	Process	Description
		Sequencing Batch Reactor (SBR)	This process utilizes a fill-and-draw reactor with complete mixing during the batch reaction step (after filling) and where the subsequent steps of aeration and clarification occur in the same tank. All SBR systems have common steps as (1) fill, (2) react (aeration), (3) sedimentation, (4) decant and (5) idle. For continuous-flow applications, at least two SBR tanks are provided in parallel so that one receives the sewage while the other completes its treatment cycle. Several process modifications have been made in the time durations associated with each step to achieve nitrogen and phosphorus removal.
		Moving Bed Biofilm Reactor (MBBR)	This process is based on several synthetic biofilm carrier elements (patented and non-patented) developed for use in the aeration tank of the ASP and are suspended in the activated sludge mixed liquor in the aeration tank. These processes are intended to enhance the activated sludge process by providing a greater surface area to unit volume of the aeration tank for the additional surfaces for increased microbial population and metabolism and hence, the biomass concentration in the tank and offers the potential to reduce the basin volumes. They are used to improve the volumetric nitrification rates and to accomplish the denitrification in aeration tanks by anoxic zones within the biofilm thickness.
		Fixed bed bio film activated sludge process (FFASP)	This process consists of a series of aerated reactors, filtration units and final polishing units. Plants with extensive root systems are placed on a supporting mesh slightly below the water level in the open aerated reactors and their roots dangling about 1.5 metres into the sewage is claimed to provide a healthy habitat for bacteria and a whole range of other organisms such as protozoa, zooplankton, worms, snails and even fish. As sewage flows through these different ecosystems in each tank, a series of self managing, cascading ecologies are stated to provide a highly robust and efficient system.

Type	Objective	Process	Description
		Immobilized Biofilm Process (Eco Bio Block, EBB)	This process is used for cleaning up polluted water sources such as sewage drains, polluted rivers, ponds, lakes, etc. The blocks are produced by mixing effective microbes with zeolites (volcanic porous stones), and alkaline cement. Once EBB is placed in polluted water, the effective microbes would multiply, treat the wastes effectively in a faster manner and clean the water body without causing any harm to plants and fish. EBB does not require energy, manpower and maintenance to perform the cleaning process. There is no operational cost practically. Since it is an online treatment, additional large land space is not required. However, the hydraulics of the drains needs validation with EBB in place
Advanced processes	Minute Solids in sus-pension colloidal materials, dissolved organic matter, TDS microbes, etc. removal	Membrane filtration	Membrane Filtration is used to remove minute solids, colloidal material, dissolved organic matter, etc. from secondary effluents using several kinds of membranes. According to separating particles size, membranes are classified as follows: Micro filtration (MF) for 0.08 to 2.0 microns Ultra filtration (UF) for 0.005 to 0.2 microns Nano filtration (NF) for 0.001 to 0.01 microns Reverse osmosis (RO) for 0.0001 to 0.001 microns The RO membrane is essentially a desalination adaptation for removal of TDS
		Membrane Bioreactor	This process is a sort of aeration tank and secondary clarifier being a two in one. The secondary clarifier is avoided by filtration of mixed liquor by membrane modules either immersed into the aeration tank mixed liquor or externally fitted and the mixed liquor routed through these. Essentially it has a suspended solids free treated sewage and retains higher MLSS and reduces the volume of aeration tanks.

Note: These are not a comprehensive listing as many newer processes keep evolving at any given time. However, in respect of biological processes, the descriptions above are meant for understanding the basics of the stated processes.

5.18.2 Recycled Nitrification / Denitrification Process

5.18.2.1 Description

Single-stage systems are those in which nutrient removal is achieved in a single basin and clarifier. Removal of nitrogen is achieved by combined nitrification (under aerobic conditions) and denitrification (under anoxic conditions). A single-stage system using one anoxic zone can achieve an effluent total nitrogen concentration of 4 to 11 mg/l as nitrogen. This is commonly called as Modified Ludzack-Ettinger (MLE) Process. The aspects of this are as follows:-

a) Nitrification is influenced by temperature, pH, DO and toxic or inhibiting substances.

b) Nitrification is possible between 5 to 50ºC. The optimum range is between 25 to 35ºC.

c) Nitrification is possible between pH 6.5 to 8.0 and the optimum condition is around pH 7.2.

d) The recommended level of dissolved oxygen is 2 mg/L

e) In order to promote biological reaction and to prevent deposits of activated sludge organisms, mixers are to be installed in the anoxic tank. The schematic drawing is shown in Figure 5-64.

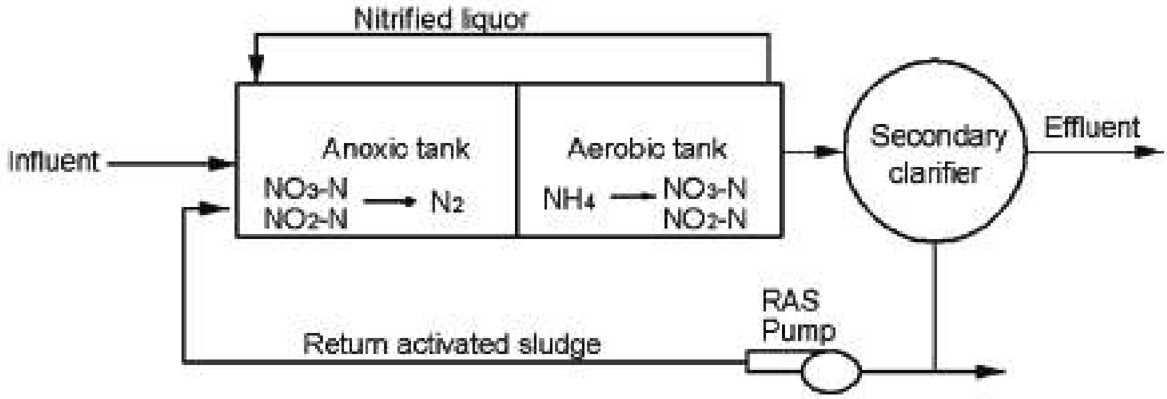


Figure 5.64 Configuration of recycled nitrification/denitrification process

5.18.2.2 Application Examples

The performance of full-scale recycled nitrification / denitrification process of Yoshinogawa STP, Yoshinogawa regional sewerage system of Nara Prefecture, Japan is shown in Tables 5-30 and 5-31.

Table 5.30 Operational conditions of Yoshinogawa STP, Japan (FY 2009)

Index	Influent Flow (m^3/d)	HRT (hrs)		F/M	MLSS (mg/l)	Sludge Age (days)	Internal Recirculation (%)	Return Sludge (%)
		Anoxic	Oxic					
Average	50	3.3	8.4	0.083	2,130	17.0	75	51
Range	8,287-11,265	2.9-3.6	7.3-9.1	.058-0.122	,810-2,370	13.2-20.3		50-54

Source:Regional Sewerage Centre of Nara Prefecture, Japan, 2009

Table 5.31 Performance of Yoshinogawa STP, Japan (FY 2009

	BOD, mg/L			SS, mg/L			TN, mg/L		
Influent / Effluent / % Removal	173	3.3	98.1	186	6	96.8	28.6	7	75.5

Source: Regional Sewerage Centre of Nara Prefecture, Japan, 2009

5.18.2.3 The K&C Valley STP at Bengaluru

This is the first STP in India for BOD removal and biological nitrification and denitrification under JICA funding for Bangalore and functioning to its 30 MLD capacity. The criteria and performance are compared in Table 5.32 with the design criteria as in the often referenced textbook “Wastewater Engineering” by Metcalf & Eddy bringing out the need for similarly criteria for other projects as well.

Table 5.32 Design criteria for biological nitrification-denitrification from results of the 31 MLD average flow Koramangala & Chellagatta (K&C) Valley STP at Bengaluru

No.	Design Parameter	(A)	(B)
1.	Raw sewage BOD, mg/l	350	140
2.	Final BOD, mg/l	17	9
3.	Raw TKN, mg/l	37	35
4.	Final TKN, mg/l	8	6
5.	Raw sewage suspended solids, mg/l	400	70
6.	Final suspended solids, mg/l	26	10
7.	HRT in anoxic zone at average flow, hrs	1.6	2.5
8.	HRT in aeration zone at average flow, hrs	20.5	9
9.	MLSS in aeration zone, mg/l	3500	3000
10.	Average DO in aeration tank	2	2
11.	F/M in aeration	0.12	0.16
12.	Ratio of RAS to plant flow	1.0	0.6
13.	Ratio of MLSS return to plant flow	2.0	3.1
14.	Surface loading in clarifier at average flow in m^3/m^2/day	12	24
15.	HRT in clarifier at average flow + RAS, hrs	3.5	NG
16.	SWD of clarifier, m	3.5	NG
17.	Solids loading in clarifier kg MLSS/m^2/day	84	115
18.	Oxygen kg for kg BOD applied	1.2	1.48

(A)- Performance of K&C Valley STP at Bengaluru, (B)- As per Example 8-2 & 8-5 of Wastewater Engineering-Metcalf & Eddy-4th Edition, NG-Not Given

5.18.2.4 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. As for usual municipal sewage, up to 85% of nitrogen removal can be expected in this process

ii. This process allows controlling the discharge of nitrogen to the receiving natural waters, which could create eutrophication problems. In this case the reduction of phosphorus is also required

iii. To limit the consumption of oxygen in the water bodies, because it requires approximately 4.57 mg of oxygen to oxidize 1 mg of nitrogen

iv. To facilitate the reuse of treated water in certain activities which require waters with low levels of nitrogen

b. Disadvantages

i. Denitrification is possible only if a good nitrification has been achieved

ii. Nitrification is possible only if adequate bicarbonate alkalinity is available, otherwise bicarbonate alkalinity is to be added

iii. Precise control of floor level gentle mixing in anoxic tank and residual DO control in oxic tank are required

iv. Toxic and inhibiting substances may affect the activity of the nitrifying bacteria, this of course being common irrespective of the process used

5.18.2.5 Typical Design Parameters

The design criteria in Table 5.32 of the Koramangala & Chellagatta (K&C) Valley STP at Bengaluru is to be relied upon until further validations become available for Indian conditions.

5.18.2.6 Applicability

This process can remove 70% to 85% of the Total Nitrogen in the sewage. It is used for nitrogen control where inhibitory industrial waste is not present. It is easier to install in new plants and also upgrade existing ASPs as the additions are stand alone and are only a half hour anoxic tank, MLSS return and additional air supply.

5.18.3 Wuhrmann Process

5.18.3.1 Description

The Wuhrmann process configuration, as shown in Figure 5.65 (overleaf), is a single-stage nitrification system with the addition of an unaerated anoxic reactor. By this reason this process is also called post-denitrification.

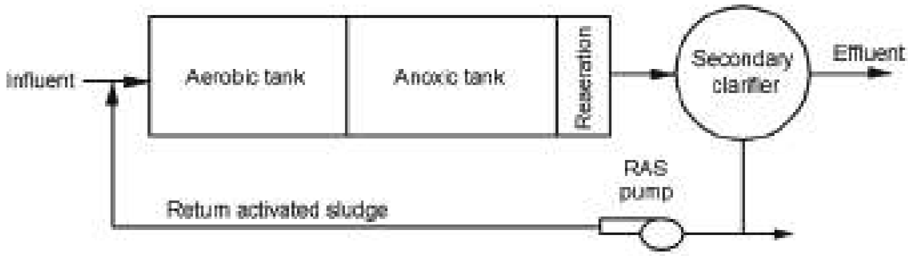


Figure 5.65 Configuration of Wuhrmann process

The possible lack of available carbonaceous substrate in the reactor significantly limits the denitrification rate of this configuration. To solve this problem an updated solution has been proposed by Ludzack and Ettinger in which the anoxic tank is the first in the sequence and is followed by the aerobic tank. Sludge from the secondary sedimentation is also recirculated to the inlet and mixed with the influent as shown in Figure 5-66. This process has a disadvantage in that a fraction of the nitrate generated in the aerobic tank is sent to the secondary sedimentation without denitrification.

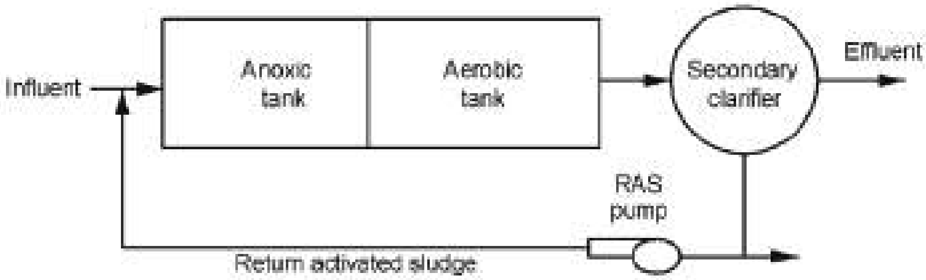
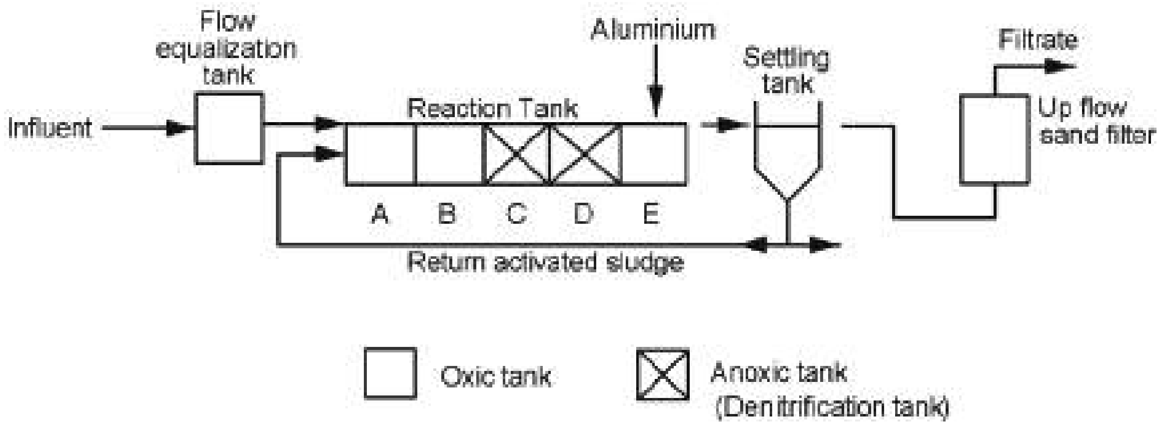


Figure 5.66 Configuration of Ludzack & Ettinger process – Wuhrmann modified

5.18.3.2 Application Examples

The process diagram and performance of full-scale Wuhrmann processes of Hamamatsu City sewage treatment plant, Japan are shown in Figure 5.67 and Table 5.33.



Source:Moriyama et al., 1988

Figure 5.67 Hamamatsu City sewage treatment plant process diagram

Table 5.33 Performance of Hamamatsu City sewage treatment plant

Parameter	Values	Parameter	Values
Water temperature (°C)	13.5 - 16.0	Influent	
Flow rate (m^3/day)	919 - 1,008	BOD	100 - 400
Return sludge ratio (%)	86 - 105	T-N	30 - 50
Retention time (hours)	23.0 - 25.1	NH_4^+-N	20 - 30
Waste sludge (m^3/day)	10.6 - 19.2	Effluent	
MLSS (mg/l)	3,670 - 3,880	BOD average	15
SRT (days)	22.7 - 37.6	T-N	5 - 10
BOD - SS loading (kg/kg/day)	0.15 - 0.23		
T N - SS loading (kg/kg/day)	0.028 - 0.037		
Sludge aeration time (days)	9.1 - 15.0		

Source:Moriyama et al., 1988

5.18.3.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Process can virtually remove all nitrogen

ii. The influent enters into the first reactor, where nitrification develops, together with removal of almost all the biodegradable, organic material

iii. The nitrified mixed liquor passes to the second reactor, where the reduction of nitrate takes place

b. Disadvantages

i. In practice, re-aeration step is usually needed after anoxic zone

ii. For high N-removal, anoxic zone is very large

iii. The anoxic tank sometimes requires the addition of organic matter to allow denitrification. So, some organic matter in excess is added to the treatment process whose objective is to reduce the organic matter content

5.18.3.4 Typical Design Parameters

Typical criteria for nitrification-denitrification systems, Pre-denitrification (Modified Ludzack Ettinger process), Post-denitrification (Wuhrmann process), and Combined pre-and post-denitrification (Bardenpho process) are as in Table 5.34 (overleaf).

Table 5.34 Typical criteria for nitrification-denitrification systems (20°C MLSS temperature)

Parameter	Pre-denitrification	Post-denitrification	Combined Pre- and Post-denitrification
System SRT, days	6-30	6-30	8-40
HRT, hours			
First anoxic zone	2-8	-	2-6
First aerobic zone	6-12	6-12	6-12
Second anoxic zone	-	2-6	2-5
Reaeration zone	-	-	0.5-1.0
MLSS, mg/l	1500-4000	1500-4000	2000-5000
Nitrate recycle	2-4 Q*		2-4 Q
RAS flow	0.5-1 Q	0.5-1 Q	100% (Q design)
Dissolved oxygen, mg/l			
Anoxic zones	0	0	0
Aerobic zones	1-4	1-4	1-4
Mixing requirements			
Anoxic zones, kw/10^3 m^3	4-10	4-10	4-10
Aerobic zones, kw/10^3 m^3	20-40	20-40	20-40
Airflow, aerobic, m^3/min 10^3 m^3	10-30	10-30	10-40

* Energy input is an important parameter; however, the manufacturer should be consulted for determining the number and placement of mixers.
Propeller and turbine mixers have been used successfully.

Source:WEF, 2010

If there is no data for DO inf, then assume a value of 2mg/l at 15ºC or less; 1 mg/l at 15ºC to 20ºC; and 0.5 mg/l at temperatures greater than 20ºC. The DO_{NR} can be assumed equal to the dissolved oxygen of the mixed liquor in the vicinity of where the nitrate recycle pump is located. They may be equal to the dissolved oxygen at the end of the aerobic zone. The dissolved oxygen of the RAS is difficult to determine without sampling the RAS or the clarifier blanket. In the absence of any data, it may be assumed that it is half the dissolved oxygen level at the end of the aerobic zone. Calculate the nitrite-nitrogen and nitrate-nitrogen load entering the pre-anoxic zone (kg/d). The criteria about nitrification, denitrification and oxygen demand are shown below:

a. Total oxygen required, as part of organic material, is 4.57 mg O_2/mgN for nitrification of which 2.86 mg O_2/mgN is released in denitrification.

b. Denitrification reduces the oxygen demand by 23g O_2/hab/d (8×2.86).

c. For an assumed contribution of nitrogen of 10 g N/person/d and an estimated requirement for sludge production of 2 g N/person/d (i.e., 20% of the influent TKN), the nitrification potential is 8g N/person/d.

d. For complete denitrification, the nitrate mass to be denitrified equals 8 g N/hab/d.

e. If methanol is used as external organic material, the consumption is 2.5 g CH_3OH/g N.

f. The per capita contribution of nitrogen varies from 2 to 5g/p/d of organic nitrogen and 3 to 7g/p/d of ammonia. The contribution of NO_3 is negligible. However, industrial activity can contribute with big amounts of nitrogen compounds.

5.18.3.5 Applicability

The simple nitrification-denitrification systems shown in this section are one of the processes used for nitrogen removal in sewage treatment as an option to control eutrophication. These processes can remove 70% - 80% of the total nitrogen in the sewage.

5.18.4 Step-feed Multistage Biological Nitrogen Removal Process

5.18.4.1 Description

In step-feed process, the basin is divided into several stages and raw influent is introduced to each stage proportionately. All return micro-organisms (sludge) are introduced at the head of the basin.

By splitting the flow to several influent feed locations and directing recovered sludge to the beginning of the process, a higher solids retention time is achieved compared to plug-flow system with the same basin volume.

Common features include anoxic zones for denitrification, and oxic zones for oxidation of organic material, nitrification and phosphorus uptake. Nitrified mixed liquor is returned from the oxic zone to the anoxic zones for denitrification.

5.18.4.2 Application Examples

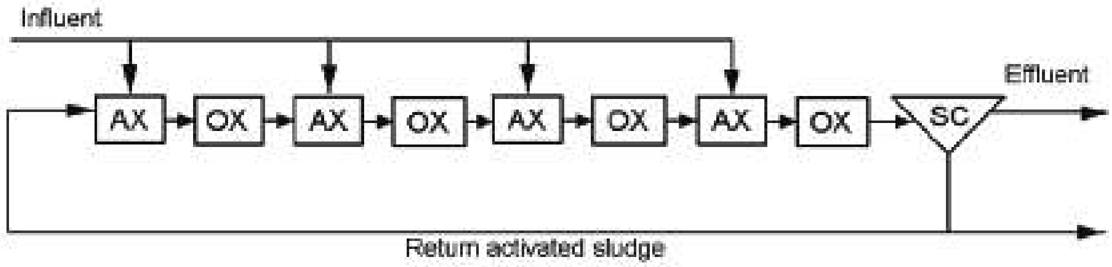
The application example of step-feed multistage biological nitrogen removal process of Stamford STP, Stamford, CT, USA is shown in Figure 5-68 overleaf.

The original STP was a traditional activated sludge system. It was updated to reduce nitrogen in the effluent to the bay.

The aeration tanks were divided in 4 phases with 25% each to operate with step feed. The incoming load was divided in 4 tanks and RAS was added in the first tank.

The yearly average discharge of TSS is 10mg/L and BOD is 6mg/l.

Nitrogen was reduced from 8-7.5mg/L to an average of 3mg/l.



Source: Christine deBarbalillo et al., 2002
(AX - Anoxic, OX - Oxic, SC - Secondary Clarifier)

Figure 5.68 Step feed schematic for four pass anoxic-oxic system in Stamford

5.18.4.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Better equalization of the waste load

ii. Operation flexibility

iii. More uniform oxygen demand along the aeration tank, with lower peak demand

iv. Allows operational control of the sludge age and hydraulic residence time

v. Can be used in preventing gross process failure due to hydraulic overloading or sludge bulking

vi. The sludge is reused several times down the tank, allowing for higher BOD treatment capacity

vii. This design reduces aeration tank size and aeration time, while BOD removal efficiency is maintained. The shorter aeration time reduces capital expenses.

b. Disadvantages

i. Requires good O&M of mechanical and control equipment to assure the right flow distribution and liquid recirculation

5.18.4.4 Typical Design Parameters

The main design criteria of this process are shown in Table 5-35 overleaf. The important considerations related to design of this process are as follows:

a. Anoxic tank with DO 0.2 mg/L and Oxic tank with DO 2.0 mg/l.

b. Denitrification Rate: 60% - 80%

c. It is a compact design. With the reduction in hydraulic retention time, the tank capacity is reduced to half that of a conventional circulating process.

d. It is energy-efficient. According to the need of nitrified liquid recirculation, energy consumption can be cut to about 70% of the usual recirculation process.

Table 5.35 Typical criteria for Step-feed multistage biological nitrogen removal process

SRT (days)	F/M as MLVSS	Volumetric Loading (kg $BOD_5/m^3/d$)	MLSS (mg/l)	HRT (V/Q) (hours)	Return Activated Sludge Qr/Q
5–15	0.2–0.4	0.64–0.96	2,000–3,500	3–5	0.25–0.75

Source: Metcalf & Eddy, 2003

5.18.4.5 Applicability

Existing facilities can be modified. Modification can be carried out without sacrificing existing treatment capacity, installing partitions in the reaction tank, creating anoxic and aerobic zones and adding distribution system for the step-feed of the primary effluent.

To be effective, all of the recirculated sludge needs to be returned to the front end of the tank, and not mixed with the influent sewage.

It addresses the common problem of filaments caused by high F/M at the front end of the tank and filaments caused by low F/M at the back end of the tank.

5.18.5 Anaerobic-oxic Activated Sludge Process

5.18.5.1 Description

Anaerobic-Oxic (Aerobic) activated sludge process is a biological phosphorus removal (BPR) process which removes phosphorus from sewage due to luxury uptake of phosphorus by activated sludge microorganisms. The schematic is shown in Figure 5.69.

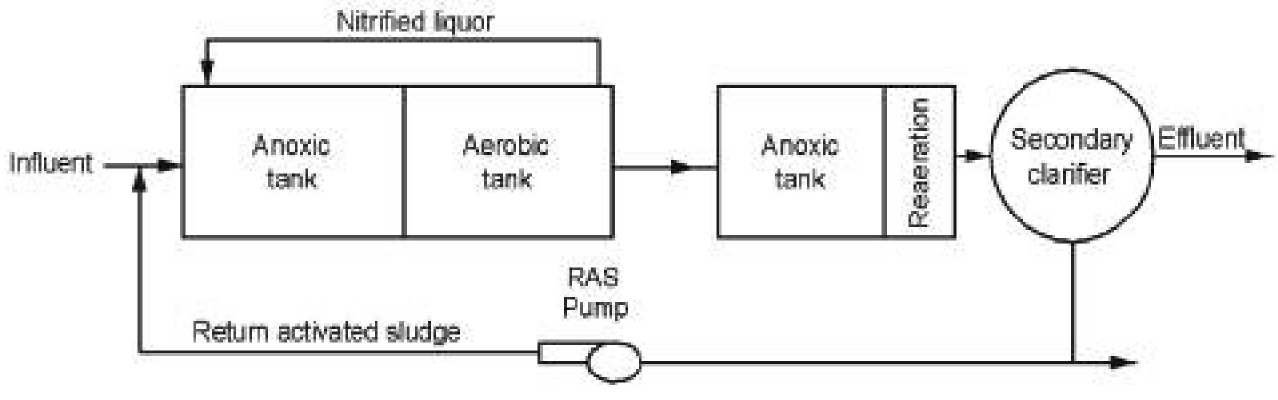


Figure 5.69 Configuration of anaerobic-oxic activated sludge process

Phosphorus is an important element of activated sludge microorganisms. Although the phosphorus content of activated sludge microorganisms is generally between 1.5 to 3%, it is possible to absorb phosphorus at a high concentration level between 2.5 to 5% by placing activated sludge under a special condition.

Specifically, activated sludge microorganisms release phosphorus into sewage in an anaerobic condition (where neither dissolved oxygen nor joint oxygen such as nitrate exists in water), and in an aerobic condition which follows the anaerobic condition, activated sludge microorganisms absorb more phosphorus than that emitted into sewage.

This phenomenon is called luxury uptake of phosphorus by activated sludge microorganisms.

This is an outstanding method for removing a considerable quantity of phosphorus from sewage without adding chemicals.

However, since the phosphorus removal efficiency of this system is affected by seasonal changes and climate such as rainfall, it is necessary to compensate for these changes by adding flocculating agent to tackle the in organic phosphorus.

5.18.5.2 Application Examples

Some examples of performance of the full-scale AO processes are shown in Table 5.36.

Table 5.36 Performance of AO processes

Site	HRT (hours)		MLSS (mg/l)	Total Phosphorous (mg/L)			Ref
	Anaerobic	Oxic		Influent	Effluent	Removal %	
Largo, USA	1.5	2.6	-	8.9	1.85	79.2	1
Pontiac, USA	1.8	6.7	2,430-2,820	3.0-4.1	0.4-0.8	80.0	2
Kawasaki JP	2.0	3.5	2,200	2.05-4.54	0.16-0.45	90.8	3
Fukuoka JP	3.3	9.7	1,620	8.2	0.63	92.3	3

Ref 1: USEPA, 1987
Ref 2: USEPA, 1987
Ref 3: Murata, 1992

5.18.5.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. For municipal sewage, about 80% of organic phosphorus removal efficiency can be expected.

ii. It is known that this process can control bulking of activated sludge caused by filamentous bacteria in addition to phosphorus removal.

b. Disadvantages

i. When the organic matter concentration (BOD or COD) of influent is low on rainy days, etc., phosphorus contained in the activated sludge microorganisms is not adequately released to liquid phase in the anaerobic tank; therefore, the removal efficiency of phosphorus deteriorates. It is desirable to maintain BOD/P in excess of 20 to 25 for good phosphorus removal.

ii. This process is susceptible to side stream containing high concentration of phosphorus from a sludge treatment system.

5.18.5.4 Typical Design Parameters

According to the performance of the actual AO processes and references on the AO process, typical design parameters of this process are integrated as shown in Table 5.37.

Table 5.37 Typical design parameters of AO process

F/M as MLVSS	SRT (days)	MLSS (mg/l)	HRT(hours)		Return activated sludge (% of influent)
			Anaerobic	Oxic	
0.2-0.7	2-25	2,000-4,000	0.5-1.5	1-3	25-40

Source: Pennsylvania Department of Environmental Protection

Important considerations related to design and maintenance of this process are as follows:

a On rainy days, etc., the removal efficiency of phosphorus deteriorates in this process. To remove phosphorus in a stable manner, equipment for adding chemical coagulant or filtration equipment, etc., need to be installed as auxiliary equipment.

b. Division wall between aerobic and anaerobic tank must be installed to prevent the adverse current of the activated sludge mixed liquor as far as possible. Moreover, since scum occurs in the anaerobic tank, it is desirable to install equipment that eliminates the generated scum.

c. To promote biological reaction and to prevent deposition of activated sludge, mixers are to be installed in the anaerobic tank.

d. The structure of the primary and final sedimentation tank should be the same as that used in a conventional activated sludge process.

e. In a sludge treatment facility, when waste activated sludge remains in the anaerobic condition, phosphorus contained in activated sludge microorganisms is released to the liquid phase.

Therefore, it is necessary to take measures to prevent such a phenomenon so that the phosphorus concentration in treated water does not increase in response to the influence of phosphorus load of the side stream.

5.18.5.5 Applicability

This process can treat BOD and SS of municipal sewage to a level equivalent to that of conventional activated sludge process, and the phosphorus removal efficiency of the A/O process depends primarily on the ratio of the BOD concentration to the phosphorus concentration in the influent. Effluent soluble phosphorus concentrations as low as 1 mg/L are possible when this ratio exceeds 10:1.

5.18.6 Bardenpho Process

5.18.6.1 Description

The first two stages of the four-stage Bardenpho process are identical to the Modified Ludzack Ettinger (MLE) system (anoxic zone followed by an aeration zone with a nitrate-rich recycle from the aeration to the anoxic zone). The third stage is a secondary anoxic zone to provide denitrification to the portion of the flow that is not recycled to the primary anoxic zone. Methanol or another carbon source can be added to this zone to enhance denitrification. The fourth and final zone is a re-aeration zone that serves to strip any nitrogen gas and increase the DO concentration before clarification. This process can achieve effluent TN levels of 3 to 5 mg/L (USEPA, 2009). The schematic is shown in Figure 5.70.

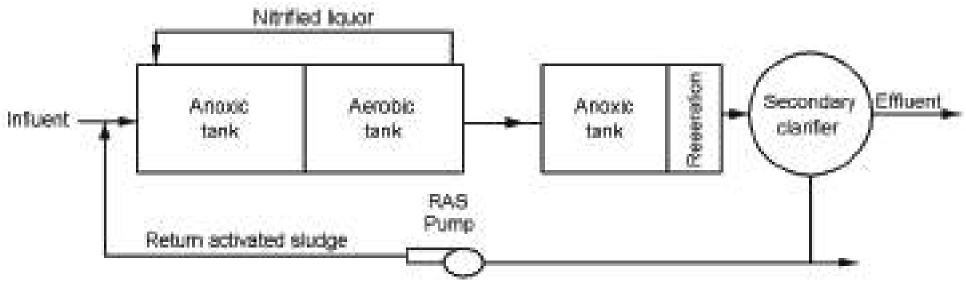


Figure 5.70 Configuration of Bardenpho process (4-stage)

The four stage Bardenpho process can be modified for the five stage system to remove both nitrogen and phosphorus. This system provides anaerobic, anoxic, and aerobic stages for phosphorus, nitrogen, and carbon removal. A second anoxic stage is provided for additional denitrification using nitrate produced in the aerobic stage as the electron acceptor, and the endogenous organic carbon as the electron donor. The final aerobic stage is used to strip residual nitrogen gas from solution and to minimize the release of phosphorus in the final clarifier. Mixed liquor from the first aerobic zone is recycled to the anoxic zone. The schematic is shown in Figure 5.71.

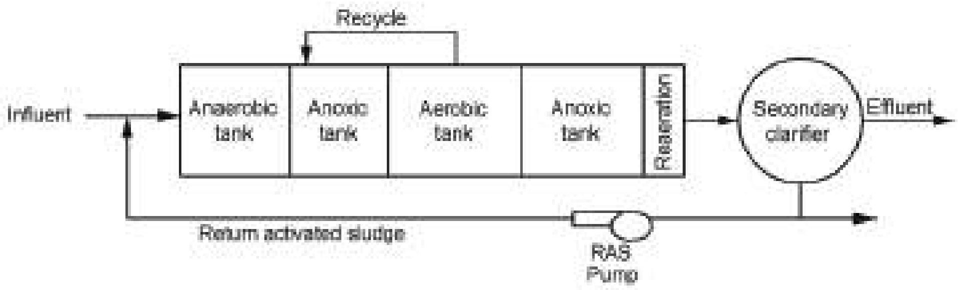


Figure 5.71 Configuration of modified Bardenpho process (5-stage)

5.18.6.2 Application Examples

The performance of full-scale Bardenpho process of 42 MLD Kelowna STP, the City of Kelowna, Canada is shown in Table 5.38.

Table 5.38 Performance of 42 MLD Bardenpho process, Kelowna STP, Canada

Parameter	Unit	Influent	Effluent
TSS	mg/l	350.0	2.0
BOD_5	mg/l	186.0	5.0
T-P	mg/l	8.50	0.14
T-N	mg/l	36.39	4.03
Faecal Coliform	cfu/100ml	N/A	0.2

Flow Capacity: 42 MLD

Process configuration: Bardenpho process + Dual media filtration process + UV disinfection process

Source: Environmental Operators Certification Program http://www.eocp.org/plants-kelowna.html

5.18.6.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. The Bardenpho process removes nitrogen to low concentrations.

ii. Addition of an anaerobic zone at the beginning of the process enables phosphorus removal.

iii. Since the nitrates in the RAS ranges from 1 to 3 mg/L, it does not seriously interfere with the mechanism for phosphorus removal as can happen in the 3 Stage Pho-redox process.

b. Disadvantages

i. Larger reactor volumes are required.

ii. Precise control of anaerobic, anoxic, and oxic conditions in the tanks is required.

iii. Toxic and inhibiting substances may affect the activity of the nitrifying bacteria.

5.18.6.4 Typical Design Parameters

Typical design parameters are shown in Table 5.39 overleaf.

5.18.6.5 Applicability

This process can achieve 3-5 mg/L T-N in unfiltered effluent.

Table 5.39 Typical design parameters of Bardenpho process

Process	SRT (d)	MLSS (mg/L)	Retention time (hrs)				Return Activated Sludge % of influent (%)	Internal Recycle % of influent (%)
			Total	Anaerobic	Anoxic	Aerobic		
Bardenpho (4-stage)	10-20	3,000-4,000	8-20	-	1-3 (1st stage) 2-4 (3rd stage)	4-12 (2nd stage) 0.5-1 (4th stage)	50-100	200-400
Bardenpho (5-stage)	10-20	3,000-4,000	9-22	0.5-1.5	1-3 (2nd stage) 2-4 (4th stage)	4-12 (3rd stage) 0.5-1 (5th stage)	50-100	200-400

5.18.7 Anaerobic-anoxic-oxic (A2O) Process (Biological Nitrogen and Phosphorus Removal Process)

5.18.7.1 Description

This process consists of tanks arranged in the sequence: anaerobic tank, anoxic tank and oxic tank. Influent and return activated sludge flow into the anaerobic tank while nitrified liquor is recycled with a circulating pump from the oxic (nitrification) tank to the anoxic (denitrification) tank.

Ammonia nitrogen is oxidized to nitrite or nitrate in the oxic tank, and then nitrite or nitrate is denitrified to nitrogen gas in the anoxic tank. Depending on the water quality of influent to the reaction tank, it may be necessary to add organic matter such as methanol and sodium hydroxide etc., to the reaction tank. Coagulant may be added to the reaction tank if more stable phosphorus removal is needed. The schematic is shown in Figure 5.72

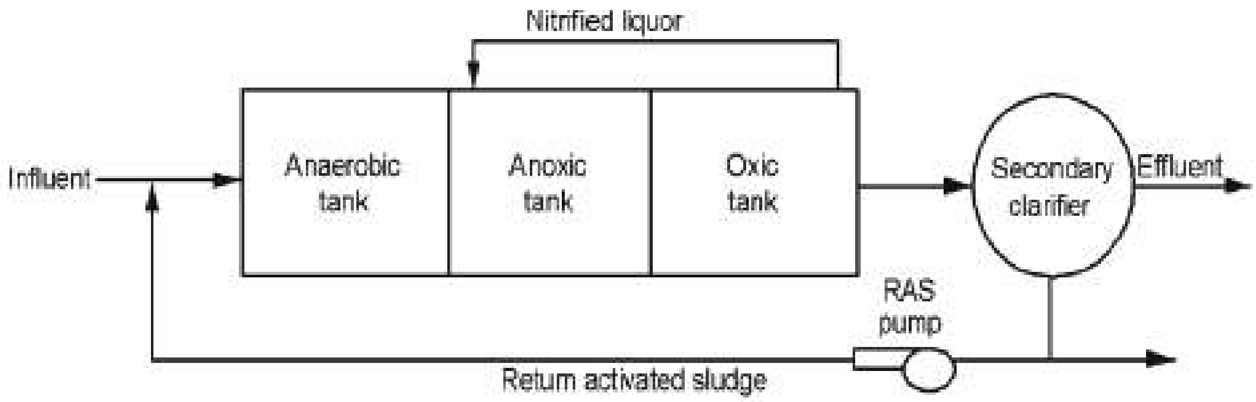


Figure 5.72 Configuration of anaerobic-anoxic-oxic activated sludge process

5.18.7.2 Application Examples

The performance of full-scale A2O process of Toba STP in Japan is shown in Table 5 40 overleaf.

Table 5.40 Performance of A2O process (Toba STP, Kyoto, Japan)

Run	Water Temp. (°C)	HRT (hr)			F/M	MLSS (mg/l)	TN or TP	Influent mg/l	Effluent mg/l	Removal (%)
		Anaerobic	Anoxic	Oxic						
1	17.6	1.3	2.7	6.0	0.12	1,570	TN	27.0	8.3	69
							TP	2.7	0.8	70
2	14.9	1.3	2.6	6.0	0.14	1,560	TN	26.9	10.0	63
							TP	2.8	1.1	61
3	20.6	1.3	2.7	6.1	0.12	1,810	TN	24.7	8.9	64
							TP	3.0	0.3	90
4	23.8	1.3	2.6	5.8	0.08	1,930	TN	17.5	6.9	61
							TP	1.9	1.5	21

Source: Murata, 1992

5.18.7.3 Advantages and Disadvantages

a. Advantages

i. Both nitrogen and phosphorus are removed simultaneously in this process.

ii. A portion of alkalinity consumed in the aerobic tank is recovered by denitrification reaction in the anaerobic reaction tank by recycling nitrified liquor from the aerobic tank to the anoxic tank.

b. Disadvantages

i. Generally, this process needs larger volume of reaction tank than that used in the standard activated sludge process.

ii. The process operating parameters of nitrogen removal, such as SRT conflict with that of phosphorus removal; therefore, the optimum SRT condition needs to be set to remove both nitrogen and phosphorus. Generally, the phosphorus removal efficiency is less than that of the AO process because higher SRT value is needed than in the AO process.

5.18.7.4 Typical Design Parameters

According to the performance of the actual A2O processes and references on the A2O process, typical design parameters of this process are integrated as shown in Table 5.41.

Table 5.41 Typical design parameters of A2O process

F/M as MLVSS	SRT (days)	MLSS (mg/L)	HRT(hours)			Return activated sludge (% of influent)	Nitrified Recycle (% of influent)
			Anaerobic	Anoxic	Oxic		
0.15-0.25	4-27	3,000-5,000	0.5-1.5	0.5-1.0	3.5-6.0	20-50	100-300

Source: Pennsylvania Department of Environmental Protection

5.18.7.5 Applicability

For municipal sewage of ordinary water quality, the nitrogen removal efficiency of this process is expected to be 60 to 70% and phosphorus removal efficiency is expected to be 70 to 80%.

5.18.8 Other Reported Processes

5.18.8.1 Sharon-Anammox Processes

The acronyms Sharon is from “Single High rate Ammonia Removal over Nitrite” and Anammox is from “Anaerobic Ammonia Oxidation.”. In this process, it is stated that a portion of the incoming ammonia is partially oxidized to nitrite and the balance ammonium is retained. These are subsequently converted into nitrogen gas under anaerobic conditions without the need to add an external carbon source. It is claimed that 40% less oxygen is adequate as compared to conventional biological nitrification, organic carbon source is not required, and sludge production is negligible. Its development into a functional plant is however, still in its nascent stage. A schematic of nitrogen transformations is in Figure 5.73.

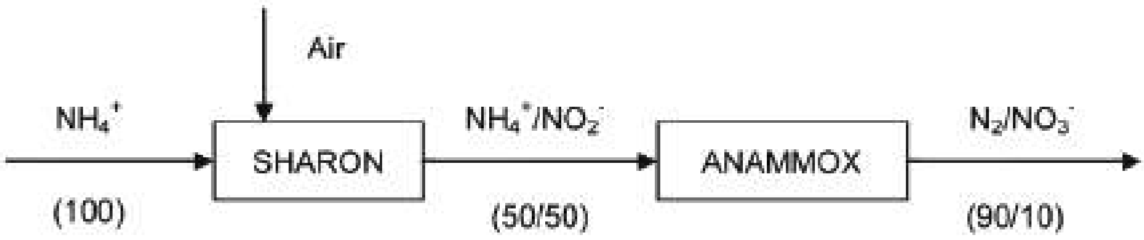


Figure 5.73 Schematic of Sharon-Anammox process for nitrogen removal

5.18.9 Guiding Principles of Biological Nutrient Removal Processes

Guiding principles of the biological nutrient removal processes which are described in the foregoing sub-sections are summarised as under:

a. Make sure whether nitrogen and phosphorous removal is required as per the discharge standards or the sewage and quality standards.

b. Decide whether removal of N or P or both is required.

c. All the design criteria are applicable for developed countries where detergents used are non-phosphate (biological) detergents. Check whether design criteria are directly applicable to India or not, where soaps and detergents containing chemical phosphates are commonly used.

d. Arising out of this, conduct a pilot-scale study to evolve guidelines that are to be followed especially for phosphorous.

e. If laboratory study is difficult, chemical removal (precipitation) of phosphorous by lime or alum is practicable.

f. In India, large scale STPs using such technologies have already been provided by the BWSSB in their STPs under the JBIC funding and these can be referred upon to draw factual data on design criteria vis a vis actual performance. Here again, this design is to be taken only as a guide and not replicated because biological nutrient removal is a complex process and has to be specific to the situation on hand based on BOD, nitrogen and phosphorous variations.

5.18.10 Membrane Filtration (MF, UF, NF, RO)

5.18.10.1 Description

Filtration membranes are classified according to the pore size or the size of solute they screen out as shown in Figure 5.74:

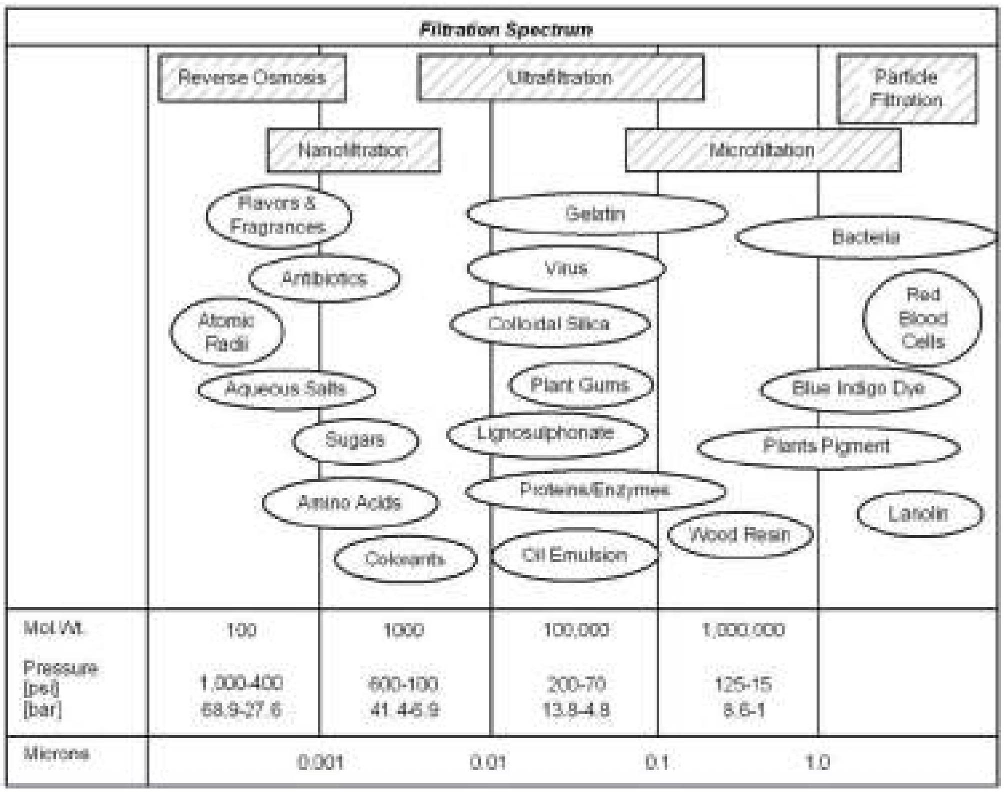


Figure 5.74 Membrane characteristics

MF - Microfiltration membranes are porous membranes with pore sizes between 0.1 and 1 micron (1 micron=1000 nanometre). They allow almost all dissolved solids to get through and retain only solids particles over the pore size.

UF - Ultrafiltration membranes are asymmetric or composite membranes with pore sizes around between 0.005 and 0.05 micron. They allow almost mineral salts and organic molecules to get through and retain only macromolecules

NF – Nanofiltration membranes are reverse osmosis with pore sizes around 0.001 micron. They retain multivalent ions and organic solutes that are larger than 0.001 micron.

RO - Reverse osmosis membranes are dense skin, asymmetric or composite membranes that let water get through and reject almost all salts.

5.18.10.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Membranes can reduce contaminants to the levels required by specific reuse application.

ii. Membranes may be added at any moment in existing sewage treatment plant and may be designed for the amount of treated water needed

b. Disadvantages

i. This process is only possible after appropriate treatment.

ii. High operation and maintenance costs

iii. Part of the water produced must be used for backwash

iv. Backwash water must be incorporated to the influent water

v. In case of RO, disposal of rejects with high dissolved solids is a problem, unless coastal discharge is available.

5.18.10.3 Typical Design Parameters

The appropriate combination of feed flow rate and TMP (Transmembrane Pressure) will maximize the flux while minimizing the impact of pumping and shear on the product. The appropriate combination of these two parameters will also minimize processing time and/or membrane area.

Membrane Area [m^2] = Process Volume [L] / (Flux [LMH]   Process Time [h])

Pump feed rate [L/min] = Feed flux [L/min/m^2]   Area [m^2]

LMH: L/m^2/h

These are fixed by membrane manufacturers.

5.18.10.4 Applicability

Membrane filtration is used for polishing water for specific uses like industry process water, or for aquifer infiltration.

In India, membrane filtration is widely used in the water and wastewater sectors. The recent sewage reclamation plants in India have all used membrane filtration to recover usable grade water from sewage in the final filtration section. It is used in various industries, airport complexes, and so on.

The difference between the UF, NF and RO is essentially the particle size removal since they depend on the pore size and colloidal particles can be removed by ultra filtration whereas dissolved salts will require reverse osmosis.

5.18.10.5 Guiding Principles

Membranes should be recognized more for the reject streams they generate and which are concentrated in pollutants. Unless it is addressed, the membrane technology will not be useful.

Spiral wound membranes need pre-treatment for complete elimination of SS, limiting COD to less than 100 mg/l and silt density index to less than 3 units besides elimination of all organics and refractory organic colour which are extremely important and complete sterilization of the feed which are challenges not easily surmountable.

Tube and disc arrangement of membranes, where the fluid passes over only one membrane film at a time and drains out in upward movement is relatively better as it can take feed with BOD, COD, SS etc. but these are costlier than spiral wound membranes.

5.18.11 Membrane Bioreactor

5.18.11.1 Description

The membrane bioreactor (MBR) process is a combination of activated sludge process and membrane separation process. Low pressure membranes (ultrafiltration or microfiltration) are commonly used. Membranes can be submerged in the biological reactor or located in a separate stage or compartment and are used for liquid-solid separation instead of the usual settling process. Primary sedimentation tank, final sedimentation tank and disinfection facilities are not installed in this process. The reaction tanks comprise an anoxic tank and an aerobic tank, and the membrane modules are immersed in the aerobic tank. Pre-treated, screened influent enters the membrane bioreactor, where biodegradation takes place. The mixed liquor is withdrawn by water head difference or suction pump through membrane modules in a reaction tank, being filtered and separated into biosolids and liquid. Surfaces of the membrane are continuously washed down during operation by the mixed flow of air and liquid generated by air diffusers installed at the bottom of the reaction tank.

The permeate from the membranes is the treated effluent.

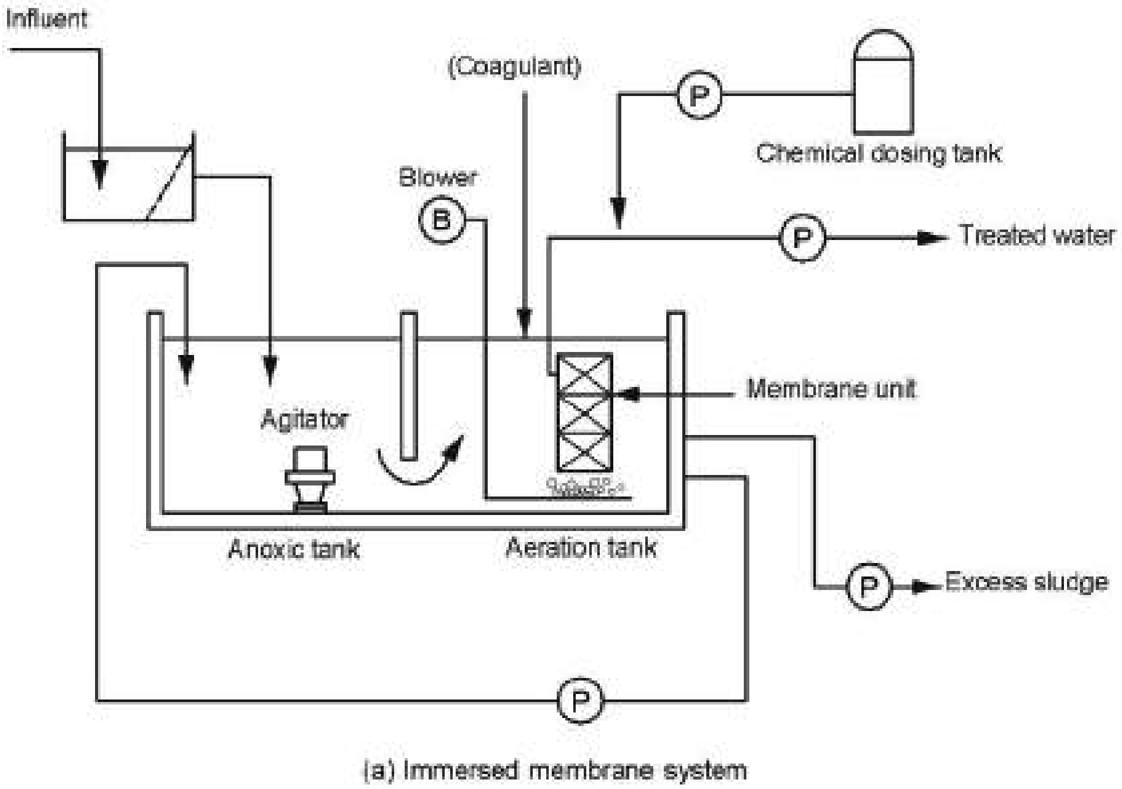
The schematic is shown in Figure 5-75 overleaf.

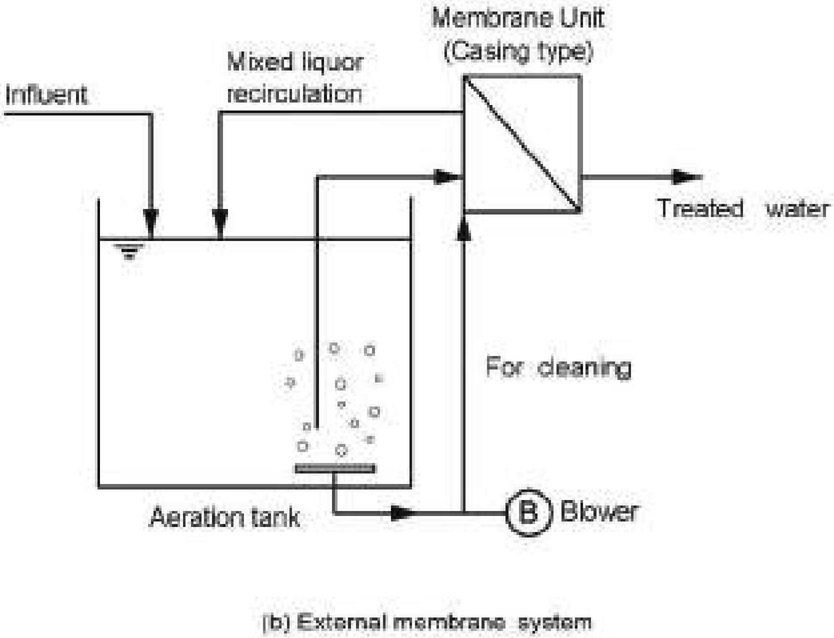
5.18.11.2 Application Examples

a. 4.54 MLD STP using MBR technology in the Games Village Complex, Delhi

The schematic flow diagram of 4.54 MLD STP using MBR technology in the Games Village Complex is shown in Figure 5.76 and the description of each process of this plant are shown in Table 5.42 The plant data are as follows:

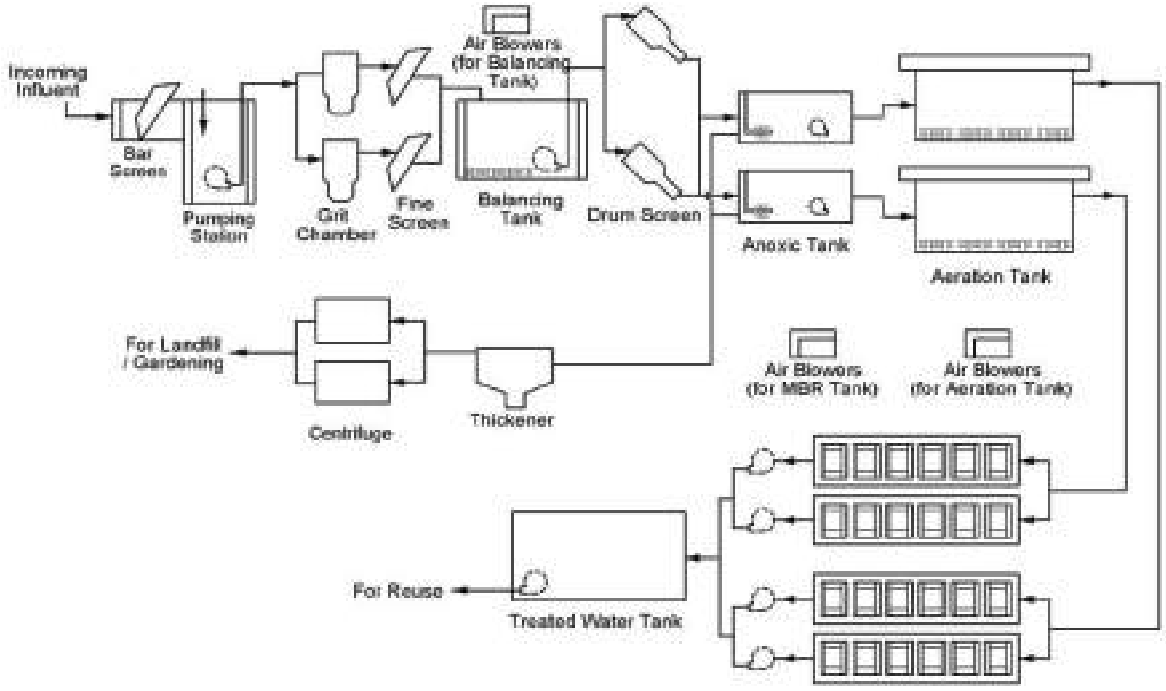
- Average flow: 4.54 MLD
- Peak Flow: 11.35 MLD
- Lean Flow: 2.0 MLD





Source:JSWA, 2003

Figure 5.75 Configuration of membrane bioreactor system



Source: Pamphlet issued by DJB

Figure 5.76 The schematic flow diagram of 4.54 MLD STP using MBR technology, Delhi

Table 5.42 Description of each process

Bar Screen	Fine Screen	Equalization/ Balancing Tank	Ultra Fine Screen	Anoxic Tank	Aeration Tank	Membrane Bio Reactor	Treated Water Holding Tank
1 Nos.	1 Nos.	1 Nos.	2 Nos.	2 Nos.	2 Nos.	4 Nos.	1 Nos.
Opening 20 mm	Opening 6 mm	2,900 m^3 (HRT 6 hrs)	Opening 1 mm,	370 m^3 (Total) (HRT 1.9 hrs)	830 m^3 (Total) (HRT 6.5 hrs)	473 m^3 (Total)	2,244 m^3

Source: Pamphlet issued by DJB

The performance of the MBR based STP at the Games village complex in Delhi is mentioned in Table 5.43 overleaf.

b. Nordkanal Sewage Treatment Plant, Germany

The operating conditions and the performance of full scale MBR process in Germany (Applied for the recycled nitrification-denitrification process) are shown in Table 5.44 and Table 5.45 overleaf.

The average flow of the MBR process is 1,024m^3/h.

Table 5.43 Performance of 4.54 MLD STP using MBR technology, Delhi

Parameter	Unit	Influent	Effluent
pH	-	7.0 - 7.6	6.8 – 7.8
Temperature	°C	18 - 38	-
TSS	mg/l	400	<1.0
BOD5	mg/l	250	<2
Total COD	mg/l	750	-
Total Kjeldahl Nitrogen	mg/l	40	<1.0
Total NH_4^+-N	mg/l	25	<0.5
Total Alkalinity	mg/l	305	-
Total Coliform	MPN/100 ml	-	2

Source: Pamphlet issued by DJB

Table 5.44 Operating conditions of MBR process (Nordkanal STP)

F/M	HRT(hours)		SRT (days)	MLSS (mg/l	Flux (l/sqm/day)	Nitrified Recycle (% of influent)
	Anoxic	Oxic				
0.04	3.1-4.9	4.8-7.6	25.0-28.6	9,200	288-552	400

Source: Moreau, 2010

Table 5.45 Performance of MBR process (Nordkanal STP)

	COD	NH_4^+-N	NO_3^--N	PO_4^{3-}-P	TSS
Influent, mg/l	997-1,210	38.9	-	8.7	11,600- 14,800
Effluent, mg/l	17.4-22.1	0	2.57-3.5	0.26-0.3	-
Removal, %	98.2	100	-	96.8(A)	-

(A) Ferric chloride addition
Source: Moreau, 2010

c. Arakawa Sewage Treatment Plant, Japan

The operating condition and the performance of pilot scale MBR process in Japan (Applied for the recycled nitrification-denitrification process) are shown in Table 5.46 and Table 5.47 overleaf. Average flow of the MBR process is 54m^3/h.

Table 5.46 Operating condition of MBR process (Arakawa STP)

HRT(hours)		SRT (days)	MLSS (mg/L)	Flux ($L/m^2/day$)	Applied Vacuum (kPa)	Nitrified Recycle (% of influent)
Anoxic	Oxic					
3.0	3.0	20.0	9,200	500-1,000	15-35	300

Source: JSWA, 2003

Table 5-47 Performance of MBR process (Arakawa STP)

		pH	T-BOD	S-BOD	TSS	TN	TP	TColiform	Al
Inf. mg/L	(A)	7.15	191	43.1	222	34.1	4.75	3.6E+05	2.48
	(R)	6.97-7.47	92-497	21.5-78.6	65-867	21.5-58.7	2.74-9.51	9.6E+04 - 7.3E+05	0.78-11.63
Eff. mg/L	(A)	7.17	1.0	-	<0.4	5.0	0.52	0.24	0.03
	(R)	6.80-7.50	0.5-2.3	-	<0.4-0.7	2.1-6.9	0.09-2.16	ND-11.00	0.02-0.06
Removal (%)	(A)	-	99.5	-	99.8	85.3	89.1	99.9	98.8

T-BOD: Total BOD, s-BOD: Soluble BOD, TSS: Total suspended solids, T-N: Total nitrogen, T-P: Total phosphorus, Al: Aluminium, T-coliform: Total coliform, ND: Non- detected, E: Exponent

Source: JSWA, 2003

d. MBR for park horticulture, Bangalore

The local authority has put up an automated MBR for horticulture of the city's famous Cubbon park. It uses the city sewage. It is under O&M by the contractor firm.

e. Porlock Sewage Treatment Plant, the United Kingdom

Porlock sewage treatment plant is the first sewage treatment plant introducing MBR process in the globe. The plant started operation in 1997 with capacity of 1.9 MLD.

The performance of the plant and number of membrane panels replaced are shown in Table 5-48 and Table 5-49 overleaf.

Table 5-48 Performance of MBR process

Parameter	BOD	COD	TSS
Influent(mg/l)	208	424	210
Effluent(mg/l)	<4	22	<2

Source: Churchhouse, 2003

Table 5.49 Number of membrane panels replaced

Period of membrane panels used (year)	Number of Membrane Panels		
	Installed (a)	replaced (b)	Ratio of b) / (a)
1	85,000	162	0.2
2	73,936	227	0.3
3	36,036	514	1.5
4	15,386	29	0.2
5	15,386	16	0.1
6	4,286	20	0.5
7	686	≤15	2.9

Source: JSWA,2003

f. Fukuzaki Sewage Treatment Plant, Japan

Fukuzaki sewage treatment plant is the first MBR plant in Japan.

The plant started operation in 2005 with capacity of 4.2 MLD. The plant also attempts to remove nitrogen and phosphorus.

The operating conditions and the performance of the MBR process are shown in Table 5.50 and Table 5.51.

Table 5.50 Operating condition of MBR process

MLSS, (mg/l)	Flux, (l/m^2/day)
8,000-12,000	400-600

Source: JSWA

Table 5.51 Performance of MBR process

	BOD		COD		T-N			T-P	
	Inf (mg/l)	Eff (mg/l)	Inf (mg/l)	Eff (mg/l)	Inf (mg/l)	Eff (mg/l)	Removal ratio (%)	Inf (mg/l)	Eff (mg/l)
FY2005	124	0.7	75.9	5.6	33.5	5.8	82.7	3.28	0.36
FY2006	159	0.9	85.2	5.4	38.3	4.9	87.2	3.61	0.20
FY2007	177	2.6	96.3	5.9	40.1	7.1	82.3	4.08	0.21
FY2008	177	3.8	109	6.0	46.1	8.3	82.0	4.66	0.08
FY2009	244	1.2	112	5.7	34.6	7.3	78.9	4.23	0.26
FY2010	263	1.5	98.4	5.5	33.1	7.3	78.0	3.49	0.29

Source: JSWA

5.18.11.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. This process does not need primary and final sedimentation tanks, and disinfection facilities; therefore, it requires smaller space than conventional biological systems (generally around 1/3 of ASP system).

ii. Since high MLSS concentration can be maintained in a reaction tank, MBRs operate at higher volumetric loading rates which result in lower hydraulic retention times. The low retention times mean that less space is required compared to a conventional system.

iii. High MLSS concentration in a reaction tank enables dewatering of the excess sludge withdrawn from the reaction tank directly without thickening.

iv. The effluent from MBRs is transparent containing almost no TSS. Organic matters (BOD) are well removed because of lower concentration of TSS compared with ASP process. Phosphorus can also be removed by adding coagulant in reaction tank.

v. Long solids residence times (SRTs) of MBR, because of high MLSS concentration, is prone to nitrification reaction in its process. Therefore, if anoxic zone is applied in reaction tank, nitrogen is expected to be removed by biological nitrification and de-nitrification reaction.

vi. E-coli is almost certainly blocked by MF membrane with pore size of less than 0.4 micro meters which is generally used for MBR system.

vii. Treated sewage by MBR can be reused for various purposes such as toilet flushing, gardening, etc. without additional treatment.

viii. O & M works are easy and free from control of bulking and sludge recirculation because final sedimentation tank is not required. Monitoring and control of treatment process can easily be automated.

b. Disadvantages

i. Oil and grease is to be fully removed as otherwise membranes will get be choked and become unusable.

ii. Needs higher capital and operating costs than conventional systems for the same throughput.

iii. Needs a flow equalization tank to regulate fluctuation of the influent flows.

iv. Needs fine screens for pre-treatment to protect membranes.

5.18.11.4 Typical Design Parameters

According to the performance of the actual MBR processes and references on the MBR process, typical design parameters of this process and effluent quality are integrated as shown in Table 5.52 and Table 5.53.

Table 5.52 Typical design parameters of MBR process

COD Loading ($kg/m^3/day$)	F/M as MLVSS	SRT (days)	MLSS (mg/l)	Flux ($l/m^2/day$)	Applied Vacuum (kPa)	DO (mg/l)
1.2-3.2	0.1-0.4	5-20	5,000-20,000	600-1,100	4-35	0.5-1.0

Source: Metcalf & Eddy, 2003

Table 5.53 Typical effluent quality

Effluent				
BOD, mg/l	COD, mg/l	NH_4^+-N, mg/l	T- N, mg/l	Turbidity
<5	<30	<1	<10	<1

Source: Metcalf & Eddy, 2003

Important considerations related to design and maintenance of this process are as follows:

a. Since constant volume of influent needs to be supplied to membrane modules, it is necessary to install a flow equalization tank to regulate the fluctuation in influent rate of flow.

b. There are two basic configurations of membrane: hollow fibre bundles and plate membranes. There is no difference in the removal efficiency of these two configurations.

c. Since the permeation flux of membranes decreases as water temperature falls, capacity of membranes should be designed considering influent temperature.

d. The area of opening of division wall between anoxic and oxic tanks must be adequate so that reverse mixing between these tanks do not occur. Moreover, the reaction tank should be covered so that foreign objects do not enter it.

e. Total amount of air required in aeration tank should be designed considering amount of air required for biological treatment and for scrubbing the surface of the membranes.

f. It is recommended that the installation include one additional membrane tank/unit beyond what the design would nominally call for considering operation and maintenance.

g. Some types of membrane require a separate washing tank for membrane modules.

h. MBR systems are configured with the membranes actually immersed in the biological reactor or, in a separate vessel through which mixed liquor from the biological reactor is filtered.

5.18.11.5 Applicability

For new installations, the use of MBR systems allows for higher sewage flow or improved treatment performance in a smaller space than a conventional biological system using activated sludge because there are no installations of secondary sedimentation tanks, sand filters and disinfection facilities. This process has been used in the past only in smaller-flow systems due to the high capital cost of the equipment and high operation and maintenance (O&M) costs.

Presently, they are being increasingly used in larger systems. MBR systems are also well suited for some industrial and commercial applications. The high-quality effluent produced by MBRs makes it particularly suitable for reuse applications and for surface water discharge applications requiring extensive nutrient (nitrogen and phosphorus) removal.

5.18.11.6 Guiding Principles

a. Maintenance of higher MLSS concentration is not easy. In practice, high MLSS concentration cannot be sustained. So smaller reaction tank is provided; and effluent BOD is not as desired. Adequate safety factor has to be considered for reaction tank.

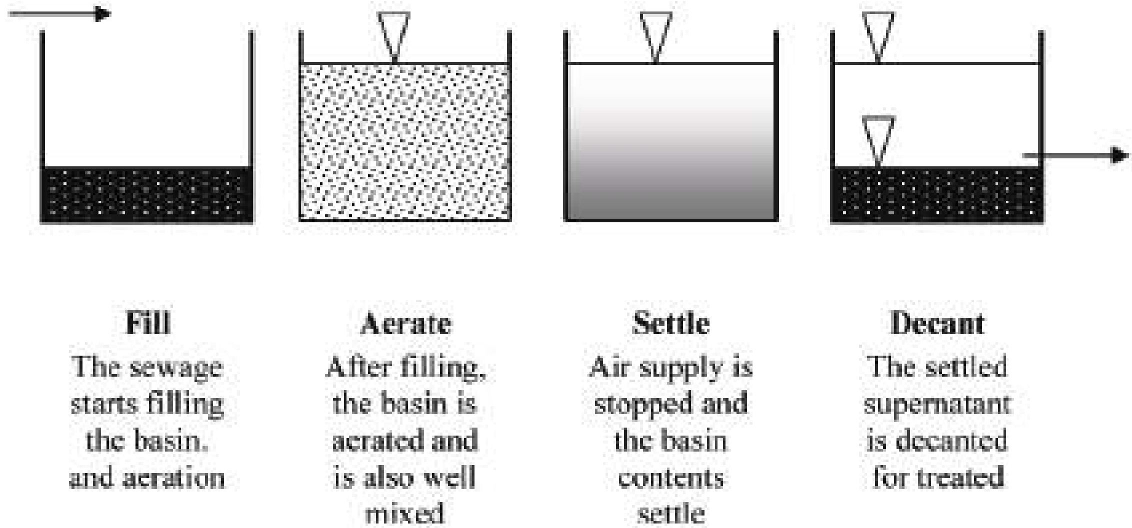
b. Manufacturer can estimate the value of MLSS for pilot equipment and then design the plant based on the estimated MLSS. Pilot equipment cost can be borne by the owner.

5.18.12 Sequencing Batch Reactors (SBR)

5.18.12.1 Description

In its functional process scheme, a Sequencing Batch Reactor (SBR) is the same as the activated sludge process. The only difference is in the activated sludge process, the sewage flows through a primary clarifier, an aeration tank and then through a secondary clarifier continuously whereas in the SBR, the aeration and settling are carried out in batch mode one after the other in the same tank.

An illustrative schematic diagram is in Figure 5.77 overleaf.



Note: All the four steps shown above take place in the same tank one following the other on a time interval basis but are shown separately in this figure for easy understanding.

Figure 5.77 Illustrative Schematic of SBR operation in the same Basin

Primary clarifiers do not seem to be provided. Consequently, at least two SBR basins are needed in parallel so that when one is in aeration, the other can be in settling and decanting of the supernatant. In fact the activated process can be referred to as continuous flow reactor (CFR). For this reason, the footprint on like to like basis of this type of SBR will be higher. In the CFR the suspended solids in the settling tank are constantly under simultaneous influence of opposing upward hydraulics of the overflowing treated sewage and gravitational setting of the suspended solids. In the SBR, this is got over by batch settling.

In fact, the CFR can also be designed with the settling tank alone in parallel modules and in batch settling alternatively. The SBR does have some advantages and they are addressed herein.

SBRs are typically configured and operated as multiple parallel basins. It aims to provide process and equipment performance, and variously include an instrumental control system that regulates timed sequences for filling, reaction, settling and effluent decanting. All these are referred to as one cycle of process control operation. It is the time duration between successive decanting sequences during which the liquid level moves from a lower water depth (bottom water level) to its fill depth (top water level) and back to its lower water depth (bottom water level). This volume progression takes place in repetitive sequences that permit reactive filling to be followed by solids liquid separation. The operational and process controls are governed as flows

a. A batch reactor consisting of a single tank equipped with an inlet for raw sewage, air diffusers, with associated compressors and piping for aeration; a sludge draw-off mechanism for waste sludge; a decant mechanism to remove the supernatant after settling; and a control mechanism to time and sequence the processes.

b. Decanting of the settled supernatant is carried out by equipment called as decanters. These consist of sharp edged weir plates over which the settled supernatant overflows similar to conventional clarifier weirs.

The scum baffles are provided before these weir plates similar to the primary clarifiers. The difference between clarifiers and these decanters is that in the case of clarifiers, the water surface remains constant and the weir plates are fixed permanently at that water surface. In the case of SBRs, the water surface will keep going down as the settled sewage is withdrawn because there is no inflow during this period. Hence, the weir plate has to move simultaneously down with the water surface and the collected settled sewage has to be discharged out of the SBR basin through a fixed pipe outlet. This is achieved by unique mechanisms called decanters. There are mainly three types of decanters namely (a) mechanized float controlled, (b) mechanized swing controlled and (c) hydraulically float controlled. These are shown in Figure 5.78 overleaf. The country has very limited experience on the performance of the various type of decanters. While selecting a decanter the competent authority may decide the type of decanter after ascertaining their field performance in the country or elsewhere in the world under similar conditions.

c. Wasting of surplus sludge typically occurs during the non mixed (aerated) stage. The sequence to take advantage of the higher concentrations of settled mixed liquor; wasting can equally take place in an aerated mixed condition.

d. SBR plants consist of a minimum of two reactors in a plant. When one reactor is in the fill and aeration mode, the other reactor can be in settling and decanting mode of the cycle.

e. In the reaction stage, the oxygen is supplied to the system within the time frame of the reaction cycle.

f. Each single SBR basin has the same floor area for all sequences in each cycle of operation.

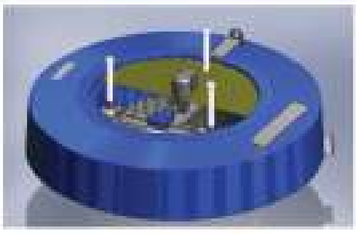
As with CFRs, there are a number of types of SBRs all of which are easily differentiated. The main differences relate to their cyclic sequencing operation. The SBR efficiency derives from a capacity to maintain good sludge settling through batch settling. As with CFRs, nitrogen removal by biological nitrification-denitrification as also biological phosphorous removal by upstream anaerobiasis can also be built into the SBRs. Generally, the SBRs are reported in F/M ratios bordering on the extended aeration mode for the full quantity of the treated sewage.

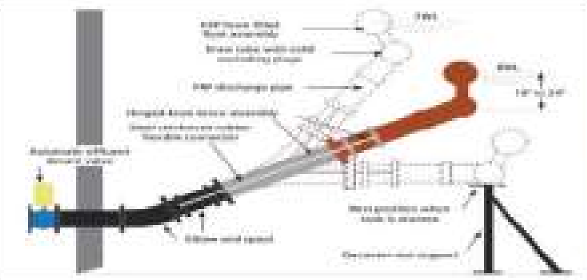
However, these can also be used with primary settling and F/M ratios like in conventional ASP in CFRs to generate biomethanation from primary and excess volatile sludges and electricity production from the methane and thus save on electricity costs. There are variants of this basic SBR technology. Some of these are as follows:

a. Intermittent Feed and Intermittent Decant SBR Process

In this process, the inflow and outflow are intermittent at the beginning and end of the treatment cycle. Interrupted inflow during the settle and decant sequencing provides the best possible environment for solids liquid separation. Operation with specific initial fill only sequencing to generate 30% to 50% bulk in basin biological selectivity mechanisms against filamentous sludge bulking; typically this is thirty percent to fifty percent of the fill-react sequence in a cycle.

A mechanized cum float arrangement. It is stated as a circular weir with a surrounding floatation device also acting as a scum baffle. The decanting rate is stated as controllable by the weir elevation by operating the three screw jacks with a single motor and depth adjustment of the weir by electric actuators and that the float is raised above the sewage during aeration and is rested on tripod supports for maintenance. Source: www.as-h.com





A hydraulic float arrangement. It is stated as using a draw tube with solid excluding plugs and kept buoyant supported by a FRP foam filled float parallel to the tube and both these integrally spanning the width of the basin at one shorter side and in multiple such decanters as needed and rested on supports erected in construction within the basin. The decant valve opening is stated to control the rate of decanting. Source: www.water.siemens.com

The mechanized swing decanter. It is stated as a motor controlled mechanism which holds the decanting weir and scum baffle and swings it down to the desired rate of decanting and adjustable at the motor and the rate of decent of the weir. Source: www.alibaba.com

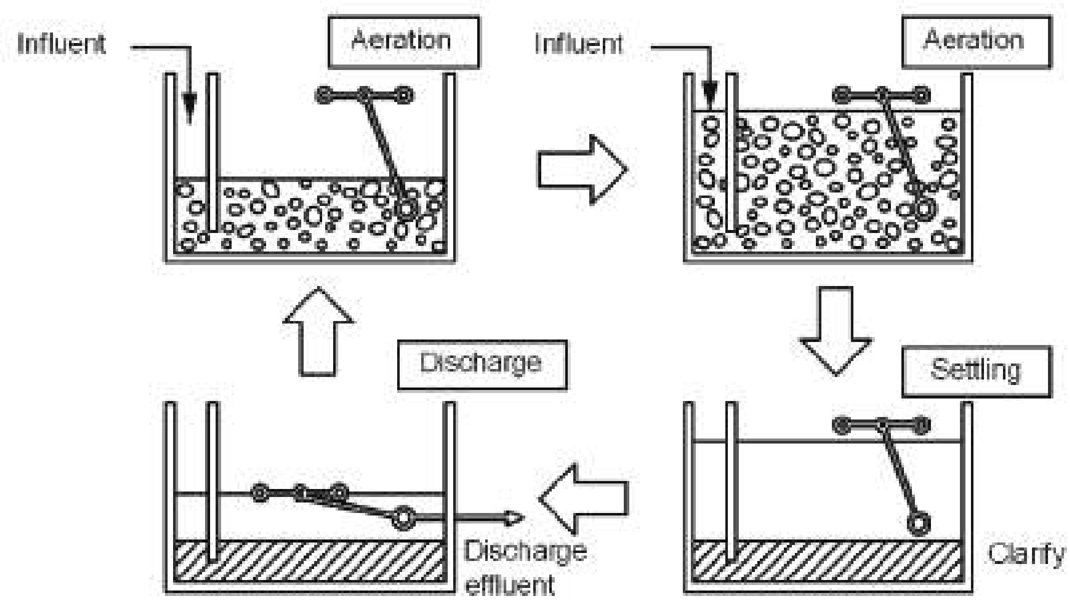
Float type decanter. The decanting pipe is stated to have floating pontoons and evenly distributed decanting holes to fitted with the company's patented closing function, preventing the sludge from entering the decanting pipe holes..

Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 5.78 Types of Decanters

Cyclic flow interruption during settle and decant sequences to positively prevent by pass flow of untreated sewage that can otherwise degrade effluent quality; all influent sewage receives aeration during a cycle. Operational protocols that maximize nitrogen removal through conventional sequenced aeration for nitrification followed by sequenced anoxic mixing for denitrification for which total cycle times are typically 6 hours. In addition appreciable denitrification also takes place during settling (Kazmi and Furumai, 2000).

This conventional SBR configuration uses sequences described as Fill, React, Settle, Decant, and Idle are shown in Figure 5.79.



Source:Nishihara Environment Co., Ltd.

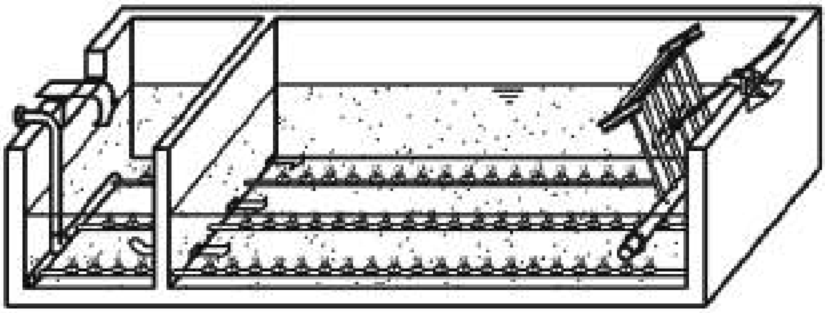
Figure 5.79 Typical operating cycles of intermittent SBR process

c. Continuous Flow SBR Process

It is stated that the need for at least two parallel modules and hence relatively larger foot print than CFR, has been sought to be got over in this process where a single reactor is divided as the pre-treat zone and downstream main-react zone and interconnected at floor level.

To start with, the liquid depth in both these zones is reduced by the depth by which the supernatant is decanted in the main-react zone. At this stage, the raw sewage continues to enter the pre-treat zone reacts with the returned MLSS and depending on the objectives, this can be run as anaerobic, anoxic or aeration or a sequential mix. The reacted MLSS flows into the main-react zone at the floor level and at an appropriate time, the air agitation is cut off permitting the solids liquid separation and the supernatant is drawn off by the decanter of the swing down or float type and sludge wasting takes place simultaneously. An advantage stated is the elimination of a separate return sludge pumpset.

An illustrative schematic is shown in Figure 5.80 overleaf.



Source:http://www.ittwww.com.au/brochures/abj_brochures.pdf

Figure 5.80 Typical configuration of Continuous Flow SBR

d. Selector Zone Incorporated SBR Process

Bioreactors operating in low F/M ratio and long sludge ages may sometimes experience problems due to filamentous organisms causing bulking sludge.

A customary solution is to provide a selector tank with a hydraulic retention time of about 30 minutes before the bioreactor and operate at a high F/M ratio which ensures growth of floc forming microorganisms while suppressing filamentous growth. The selector also helps in removing some of the organic phosphorus without chemical addition. This is also used in the case of SBR. The reactor is partitioned initially for the selector zone.

e. Chemically Enhanced SBR Phosphorous Removal Process

Addition of metal salts if necessary by supplementing the bicarbonate alkalinity is also another customary solution in removing phosphorous which remains in excess of the uptake.

This is also used in the case of SBR by adding the alkalinity as needed in the react cum fill stage and the metal salt in the subsequent react stage.

All sewage received in a cycle is similarly treated as in the conventional SBR. Purposely designed and controlled reaction environments of aerobic, anoxic and anaerobic are a feature of this variation of a conventional SBR.

5.18.12.2 Application Examples

a. Mundhwa Sewage Treatment Plant, Pune, India

The design details of the Mundhwa STP reproduced in Table 5.54 (overleaf) are extracted from the IIT Roorkee report August 2010.

The performance results are extracted from the IWWA 2011, It is stated that the raw sewage values of BOD, SS, TKN and PO4-P are 205 mg/l, 262 mg/l, 45 mg/l and 2.3 mg/l and the treated sewage values of BOD, SS, TKN are reduced to less than 10 mg/l, and PO4-P is reduced to 0.7 mg/l.

Table 5.54 Design criteria stated as used in the 45 MLD Cyclic Activated Sludge process based SBR at Mundhwa STP in Pune

Design Flow • Average flow • Peak factor • Peak flow	 45 MLD 2.25 times average flow 95 MLD
Influent Quality to SBR Basin • BOD • SS • TKN • TP • Faecal coliform	 250 mg/l 350 mg/l 45 mg/l 5 mg/l 10^6 MPN/100 ml
Effluent Quality of SBR Basin • BOD • SS • TKN • TP • Faecal coliform	 < 10 mg/l < 10 mg/l < 10 mg/l < 2 mg/l < 100 MPN/100 ml
No. of basins	4
Each basin area	1431 m^2
Basin foot print	127.5 m^2/1000 m^3/d
Decant volume	3377 m^3
Decanter loading	300 m^3/1000 m^3/d
Hourly flow rate	1875 m^3/h
Hourly flow rate to each basin	937 m^3/h
No. of cycles per day/basin • Filling and aeration • Settling phase • Decanting phase Total cycle time	8 90 min. 45 min. 45 min. 3 h
Hours of aeration time/day/basin	12 h
MLSS MLVSS F/M	4300 mg/l 3440 mg/l 0.08
HRT	16.30 h
SRT	14.86 days
Selector (Anoxic) Zone • Number of selector compartments/basin • Retention time in selector zone	1 50 min.
Disinfection • Chlorine dose after SBR Basin	 2.5 mg/l

Source: IIT Roorkee, August 2010

b. Culver Sewage Treatment Plant, USA

The operating conditions and the performance of the Culver SBR process in USA are shown in Table 5.55 and Table 5.56.

Table 5.55 Operating conditions of the Culver SBR process

Average flow (MLD)	Mode of Operation (minutes)							Number of Basins	MLSS (mg/L)	F/M (a)	SRT (days)
	Fill and Mix	Fill and Aeration	React	Settle	Decant	Idle	Total cycle time				
1.34	54	126	42	42	42	60	366	2	5,000	0.08-0.16	15.5

Source: USEPA, 1986, USEPA, 1983

Table 5.56 Performance of the Culver SBR process

	BOD	TSS	$NH4^{+}$-N + $NO3^{-}$-N	T - P
Influent (mg/L)	170	150	22.0	6.5
Effluent (mg/L)	10.5	5.5	2.4	0.75
% Removal	94	96	89	88

Source: USEPA, 1986, USEPA, 1983

5.18.12.3 Advantages and Disadvantages

a. Advantages

i. One single reactor basin provides all of the unit operations and processes that require two separate basins in a conventional activated sludge plant configuration that can provide an effluent quality suitable for reuse. Equalization, primary clarification (in most cases), biological treatment, and secondary clarification can be achieved in a single reactor vessel.

ii. This process can be operated and controlled with flexibility for efficient removal of organic matter, suspended solids, nitrogen, and phosphorus under all loading conditions. Provides enhanced organic phosphorus removal with or without chemical augmentation.

iii. This process can control the growth of filamentous bacteria and hence prevent bulking of activated sludge.

iv. This process saves capital cost by eliminating final sedimentation tanks. As secondary sedimentation tanks are not required in this process, footprint area needed is also minimal as simultaneous multiprocessing takes place in a single reactor basin (approximately $100m^2/1000m^3$ only needed for SBR Tanks).

v. Can be used with primary clarifiers and power generation configurations where the ratio of VSS:TSS is high.

vi. Allows for easy modular expansion for population growth, modular configurations and cyclic operation is easily managed to provide continuous inflow and outflow hydraulic profiles, dispensing with the need for outflow hydraulic balancing

b. Disadvantages

i. Compared to the conventional activated sludge system, a higher level of sophistication and maintenance can be associated with more automated switches and valves.

ii. Basin depth should be sufficient to provide an adequate clear water depth over the sludge blanket to prevent settled solids entrainment.

iii. In small single stream SBR systems approximately less than 10 MLD, effluent flow balancing may be needed for downstream processing, such as filtration or disinfection.

iv. Short-circuiting of influent conservative parameters (ammonia nitrogen, orthophosphate) under the non-interrupted inflow protocol may be a process failure consideration in some SBRs.

v. Larger capacity aeration system, relative to aeration time per cycle and per day, is required compared as to conventional activated sludge system.

vi. The potential for discharging floating or settled sludge during the decant phase with certain SBR configurations.

vii. Potential plugging of aeration devices during selected operating cycles depending on the aeration system used by the manufacturer.

viii. There should be sufficient allowance of clear water depth from the sludge blanket to minimize sludge carryover. The volume of water decanted should be limited to prevent scouring of solids.

ix. All the SBR plants must be designed to cater to the peak flows. A minimum of two tank system is required.

5.18.12.4 Typical Design Parameters

A compilation of typical process details that would feature in the use of SBR facilities is mentioned in Table 5.57. It has to be recognized that as with all similar technologies, these are only of informative value for India and it is mandatory that there is a demonstrated available level of Indian expertise and support services for the design of SBR systems and its operational methodology for India which is to be hereafter evolved with reference to a validation of design vs. actual performance of SBRs built in India.

One of the classical difficulties that pertain to establishing the design parameters for SBR is the biomass metabolism. In the ASP and CFR, it takes place under “steady state conditions” where a steady BOD profile from inlet to outlet is existing in the aeration tank irrespective of time.

Table 5.57 Typical process parameters for SBR configurations (for unsettled sludge)

S. No.	Parameters	Units	Continuous Flow and Intermittent Decant	Intermittent Flow and Intermittent Decant
1	F/M ratio	d^{-1}	0.05 - 0.08	0.05 - 0.3
2	Sludge Age	d	15 - 20	4 - 20
3	Sludge Yield	kg dry solids/ kg BOD	0.75 - 0.85	0.75 - 1.0
4	MLSS	mg/L	3,000 - 4,000	3,500 – 5,000
5	Cycle Time	h	4 - 8	2.5 - 6
6	Settling Time	h	> 0.5	> 0.5
7	Decant Depth	m	1.5	2.5
8	Fill Volume Base	-	Peak Flow	Peak Flow
9	Process Oxygen			
	BOD	kg O_2/kg BOD	1.1	1.1
	TKN	kg O_2/kg TN	4.6	4.6
* For Phosphorous ≤ 1 mg/L, after bio-P removal, metal precipitant (Fe^{3+} or Al^{3+}) shall be added. Sludge yield factor and sludge age not applicable for primary settled sewage; typical primary TSS removal 60%, BOD 30%.				

In the case of SBR, the biomass metabolism takes place under “unsteady state conditions” where the BOD profile decreases with time during the batch time interval. Thus, calculating the oxygen requirements is a challenge depending on many factors. In actual practice, the oxygen requirement is calculated as though it is a steady state condition as in the CFR and then the rate of air delivery to the basin is calculated by delivering the entire volume of air in the actual aeration interval. This will need a much bigger air compressor and air diffuser system.

However the motor is controlled by VFD and thereby the delivery of the compressor is gradually adjusted down to suit the real need or maintained as it is based on maintaining the required residual D O in the basin at the end of aeration. It is a unique feature in the design of SBR aeration facilities.

However, it needs a focussed study to establish the actual oxygen uptake rate as a function of aeration interval to make future designs more realistic. Such data as validated by actual observations is not found in literature.

5.18.12.4.1 Disinfection tank volume

The capacity of the chlorine contact tank for the treated sewage of batch processes such as the SBR will be based on 30 minutes detention time of the rate of decant flow calculated as volume of decant flow divided by the duration of decanting of any one or multiple reactors decanting simultaneously.

5.18.12.5 Applicability

Municipal sewage is successfully treated in SBR systems. As with conventional activated sludge plants, SBRs can be used for all plant sizes. Current practice examples large scale facilities in municipal applications to about 270 MLD, (Goronszy, 2008), Especially where land availability is limited; plants can easily be installed on a multi-level basis, like the one reported to be in use at Thailand as in Figure 5.81.

Figure 5.81 Yannawa STP, Thailand

5.18.12.6 Guiding Principles

With proper anticipations in the design stage, the SBR process can be installed with good flexibility to adapt to future regulatory changes for effluent parameters such as for nutrient removal. As with conventional activated sludge variants, there are several SBR variants each of which requires their own design considerations. Design and operation for efficient nitrogen removal provides enhanced process stability, especially with operating temperatures greater than 20ºC. It is necessary to bear in mind that there are no procedures to design the SBR, like all other biological processes for obtaining a desired removal of coliform organisms. At best one can cite another functioning SBR but the result of that STP need not necessarily be the same for the new SBR even though it is designed on the same lines. It applies to almost all such biological processes. Hence, provision for variable chlorine dosage of the SBR basin effluent should be made.

5.18.13 Moving Bed Biofilm Reactors (MBBR)

5.18.13.1 Introduction

The moving bed biofilm reactor (MBBR) is based on the biofilm carrier elements. Several types of synthetic biofilm carrier elements have been developed. These biofilm carrier elements are floated in the mixed liquor in the aeration tank and are kept floating by the air from the diffusers. They have a tendency to accumulate at the top zones. Hence wall mounted mixers propel the media downwards so that they again float and are in circulation in the mixed liquor. They are retained by suitably sized sieves at the outlet.

This process is intended to enhance the activated sludge process by providing a greater biomass concentration in the aeration tank and thus offer the potential to reduce the basin volume requirements. They have also been used to improve the volumetric nitrification rates and to accomplish the denitrification in aeration tanks by having anoxic zones within the biofilm depth. Because of the complexity of the process and issues related to understanding the biofilm area and activity, the processes design is empirical. There are now more than 10 different variations of the processes in which a biofilm carrier material of various types are suspended in the aeration tank of the activated sludge process. There are many examples of such activated sludge treatment process with suspended biofilm carrier in the world. In this section, some of the more widely cited processes such as the Captor®, Linpor®, Pegasus®, and Kaldnes® are described and some design considerations and parameters are cited.

5.18.13.2 Description

a. Captor® and Linpor® process

In the Captor® and Linpor® processes, foam pads with a specific density of about 0.95 g/cm^3 are placed in the bioreactor in a free-floating fashion and retained by an effluent screen. The pad volume can account for 20% to 30% of the reactor volume. Dimensions of the carrier materials are presented in Table 5.58.

Table 5.58 Dimensions of the carrier materials

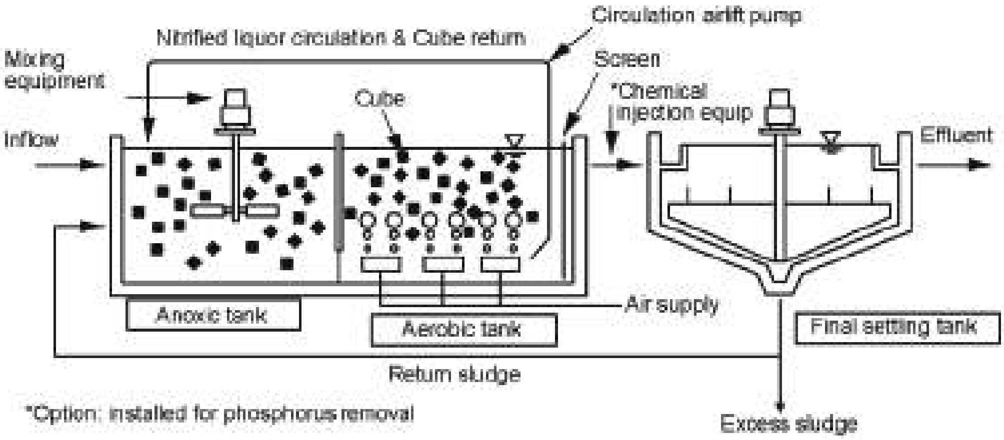
Name	Carrier specifications	
	Material	Dimension, mm
Captor®	Polyurethane	30 x 25 x 25
Linpor®	Polyurethane	10 to 13 cubes

Source: Metcalf & Eddy, 2003

Mixing from the diffused aeration circulates the foam pads in the system, but without additional mixing methods. They may tend to accumulate at the effluent end of the aeration basin and float at the surface. An air knife has been installed to clean the screen continuously and a pump is used to return the carrier material to the influent end of the reactor. Solids are removed by a conventional secondary clarifier and wasting is from the return line as in the ASP.

The principal advantage for the sponge carrier system is the ability to increase the loading of an existing plant without increasing the solids load on existing secondary clarifiers, as most of the biomass is retained in the aeration basin. Loading rates for BOD of 1.5 to 4.0 kg/m^3/d with the equivalent MLSS concentration of 5,000 to 9,000mg/L have been achieved with these processes. Based on the results with full-scale and pilot-scale tests with the sponge carrier installed, it appears that the nitrification can occur at the apparent lower SRT values, based on the suspended growth mixed liquor, than those for activated sludge without internal carrier. Compared to other biofilm carriers, Captor® and Linpor® Cubes are larger; meaning the openings of the screen used to block the outflow of the biofilm carriers can also be larger.

Therefore, there is less clogging of the screen due to scum. Furthermore, Linpor® Cubes are stated to have better durability and it is stated that even after a long period use, replacement is not necessary. The Linpor® system was developed by Linde AG in the mid-1970's, Linpor®-CN Process and Linpor® Cubes carrying sludge are shown in Figure 5.82 and Figure 5.83.



Source: Nishihara Environment Co., Ltd.

Figure 5.82 Linpor®-CN process flow diagram

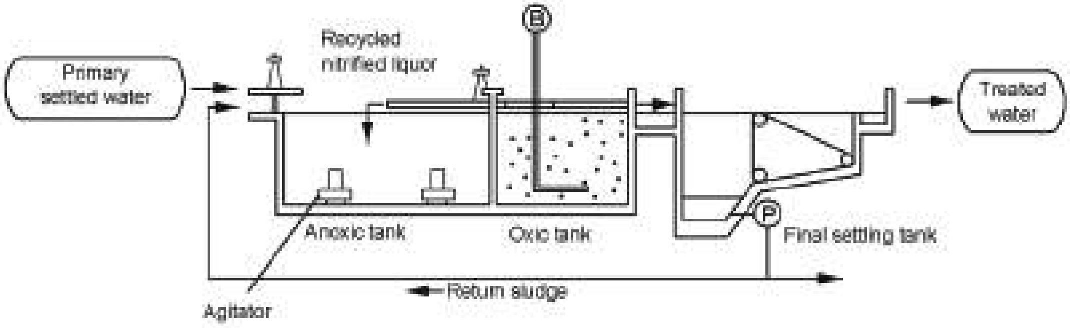
Source:Nishihara Environment Co., Ltd.

Figure 5.83 Linpor® Cubes carrying sludge (12×12×15 mm)

b. Pegasus®/Bio-cube process

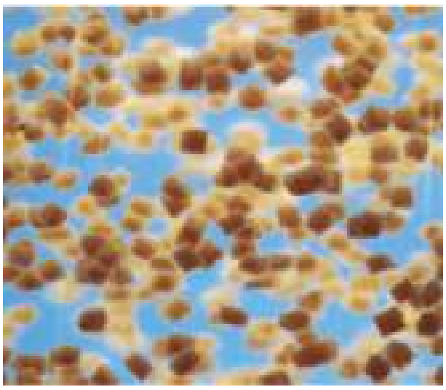
The Pegasus system is based on the immobilization of nitrifying bacteria in organic gel pellets called bio-n-cubes which are non-biodegradable organic matrix consisting of a mixture of polyethylene glycol and nitrifying activated sludge. The number of immobilised autotrophic bacteria is independent of the sludge age and thus higher than in comparable low-loaded activated sludge systems. The immobilization prevents the autotrophic bacteria from wash-out, improves the nitrification kinetics and results in a lower temperature dependency for the ammonia removal process. The bio-n-cubes are maintained in suspension in the aeration tank and wash-out is prevented by a retention grid at the outlet of the tank.

If the existing volume of the activated sludge tank enables the hydraulic retention time of the sewage to be at least five hours, total nitrogen removal is stated as implemented directly in the activated sludge by means of the Pegasus immobilised culture system. The biopellets used in the Pegasus process, called bio-n-cubes are relatively small in size and produced in a way to ensure a balance between oxygen transfer, biomass growth and suspendability. The annual wear rate of volume of bio-n-cubes is stated as 1%. The bio-n-cubes are proprietary products of Hitachi Plant Technologies, Ltd. The schematic is shown in Figure 5.84. The bio-cubes are shown in Figure 5.85. The parameters of the pellets used are given in Table 5.59.



Source:Hitachi Plant Technologies, Ltd., 2010

Figure 5.84 Pegasus® process flow diagram



Source: Hitachi Plant Technologies, Ltd 2010

Figure 5.85 Pegasus® bio-n-cubes (3×3×3 mm)

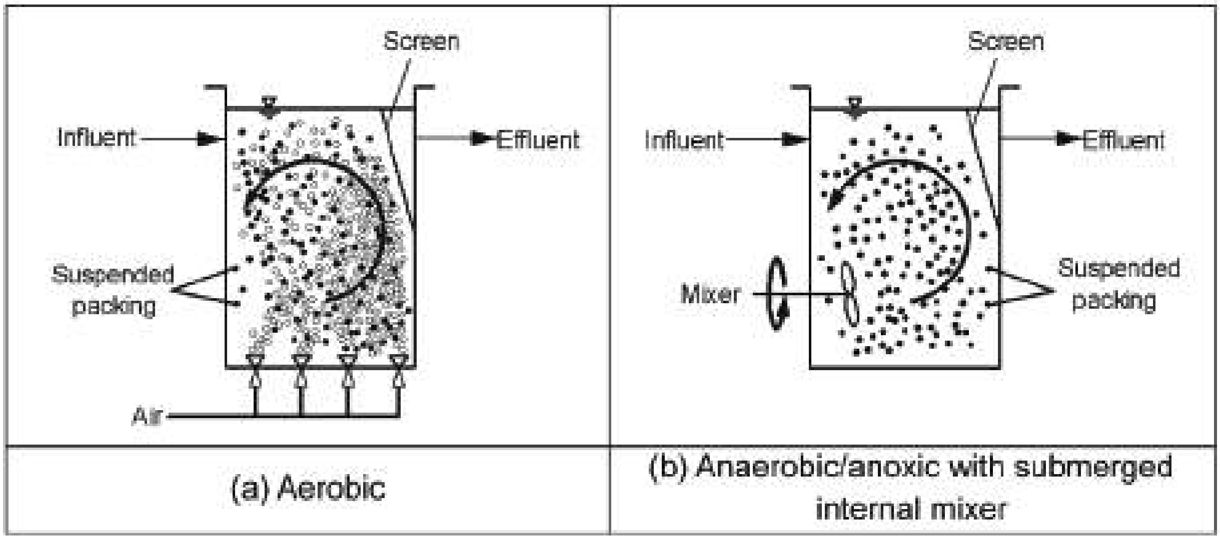
Table 5.59 Parameters of the pellets of the Pegasus process

Pellet material	Polyethylene glycol (PEG)
Pellet size	3×3×3 mm
PEG volume fraction	10 - 20%
Micro-organism fraction	2%
Density	1.03 g/cm^3
Surface area	700 m^2/m^3 reactor volume
Biofilm thickness	~ 60 μm

Source: Hitachi Plant Technologies, Ltd., 2010

c. Kaldnes® process

This technology was developed by the Norwegian firm of M/S Kaldnes Miljoteknologi. The processes consist of adding small cylindrical shaped polyethylene carrier elements (specific gravity of 0.96 g/m^3) in aerated or non-aerated tanks to support biofilm growth. The small cylinders are about 10 mm in diameter and 7 mm in height with a cross inside the cylinder and longitudinal fins on the outside. The biofilm carriers are maintained in the reactor by a perforated plate (5×25 mm slots) at the tank outlet. Air agitator or mixers are used to circulate the packing continuously. The packing may fill 25% to 50% of the tank volume. The specific surface area of the packing is about 500 m^2/m^3 of bulk packing volume. The typical reactors are shown in Figure 5.86. The biofilm carriers are shown in Figure 5.87. The schematic is shown in Figure 5-88 overleaf.

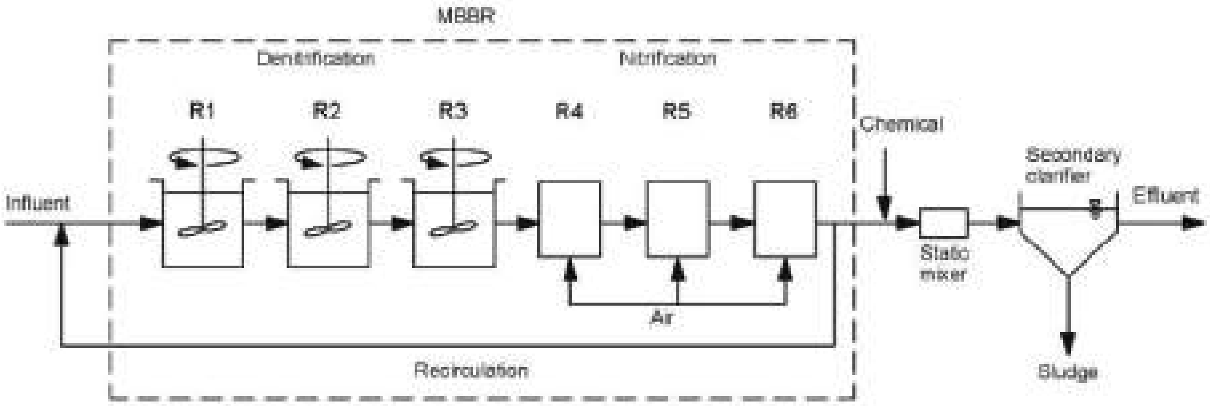


Source: Metcalf & Eddy, 2003

Figure 5.86 Typical reactors of MBBR process

Source:Stowa Webpage

Figure 5.87 Kaldnes biofilm carriers (About 10 mm diameter × 7 mm height)



Source: Metcalf & Eddy, 2003

Figure 5.88 Schematic flow diagram of MBBR process for removal of BOD and nutrients

d. FAB Technology

The fluidized aerobic bioreactor includes a tank in any shape filled up with small carrier elements. The elements are specially developed materials of controlled density such that they can be fluidized using an aeration device.

A biofilm develops on the elements, which move along with the effluent in the reactor. The movement within the reactor is generated by providing aeration with the help of diffusers placed at the bottom of the reactor. The thin biofilm on the elements enables the bacteria to act upon the biodegradable matter in the effluent and reduce BOD/COD content in the presence of oxygen from the air used for fluidization. The technology is shown in Figure 5.89.

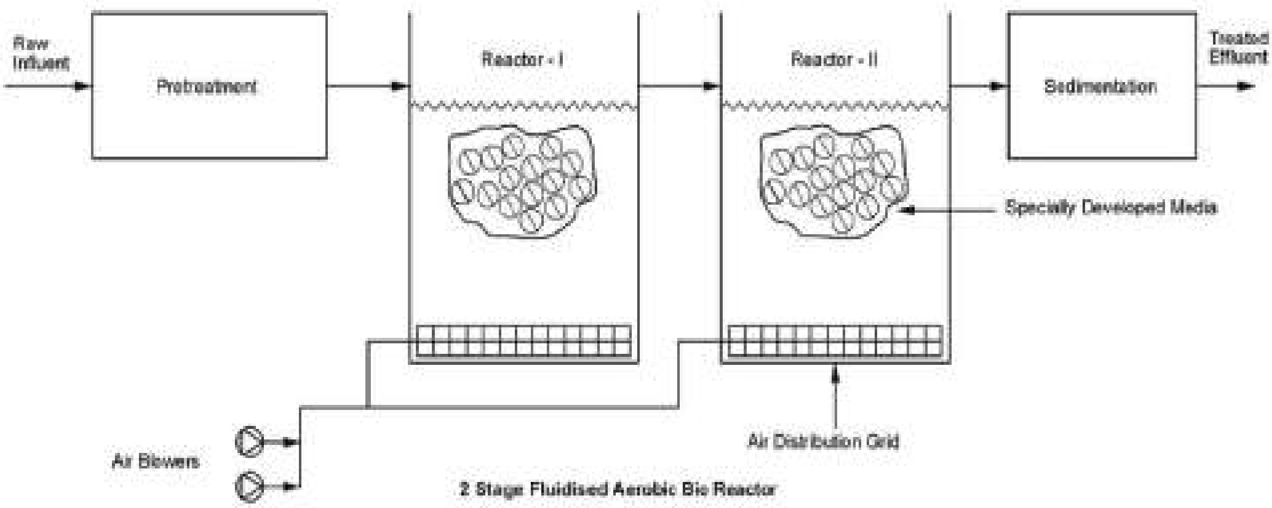


Figure 5.89 FAB technology

The merits of the FAB technology as stated by its promoter is mentioned in Table 5.60 overleaf.

Table 5.60 Merits of FAB technology as stated by its Promoter

Features	Benefits
Attached growth process	No sludge recycle No monitoring of MLSS Low sludge production
High Biofilm surface area	High loading rates Compact plants Small foot print
Fluidised Bed	Non clogging design Better oxygen transfer efficiency Reduced power consumption Reduces coliform Low maintenance Tank of any shape can be utilized

5.18.13.3 Application Examples

a. Captor® and Linpor® process

The design details and the performance data of the Linpor® CN Process for a biological nitrification-denitrification STP of Freising, Germany are mentioned in Table 5.61 and Table 5.62.

Table 5.61 Design Details of the biological nitrification-denitrification Linpor process (Freising, Germany)

	Flow rate (MLD)	MLSS(mg/L)			Carrier Volume (%)	F/M (1/d)
		Suspended	Fixed	Average		
Design	23.6	3,800	15,000	5,800	22	0.12
Operation	21.4	5,700	18,000	7,700	20	0.04

Source: Gilligan and Morper, 1999

Table 5.62 Performance of the biological nitrification-denitrification Linpor process (Freising, Germany)

	Design		Operation	
	Influent	Effluent	Influent	Effluent
BOD (mg/l)	222	-	106	3
COD (mg/l)	397	-	208	-
TKN (mg/l)	46.6	-	29	<0.6
NH_4^--N (mg/l)	35.9	<5	16	<0.1
NO_x^--N (mg/l)	2.6	-	-	9.9
Total-N (mg/l)	49.2	<18	29	9.9

Source: Gilligan and Morper, 1999

b. Pegasus®/Bio-cube process

The design details and the performance data of Pegasus process for a biological nitrification-denitrification STP of Munakata City, Japan are in Table 5.63 and Table 5.64.

Table 5.63 Design details of the biological nitrification-denitrification Pegasus process (Munakata City, Japan)

Maximum daily low (m3/day)		11,300
Primary settling tank	Surface-loading (m^3/m^2/day)	36
	Retention time (hours)	1.7
Anoxic tank	Capacity (m^3)	2,008
	Retention time (hours)	4.3
Oxic tank	Capacity (m^3)	1,436
	Retention time (hours)	3.0
	Pellet dosing ratio (%)	7.5
	Circulation ratio of nitrified liquor to influent flow (-)	1.5-3.0
Final settling tank	Surface-loading (m^3/m^2/day)	25
	Retention time (hours)	2.9
PAC feeder	Dosage of coagulant (L/1,000m^3)	70
	A/p (-)	1

Source: Hitachi Plant Technologies, Ltd.

Table 5.64 Performance of the biological nitrification-denitrification Pegasus process (Munakata City, Japan)

Water quality	Influent	Effluent from secondary settling tank	After rapid filtration
BOD (mg/l)	210	14.2	10
SS (mg/l)	250	20.5	5
T-N (mg/l)	40	10.2	10
T-P (mg/l)	6	0.4	0.3

Source: Hitachi Plant Technologies, Ltd.

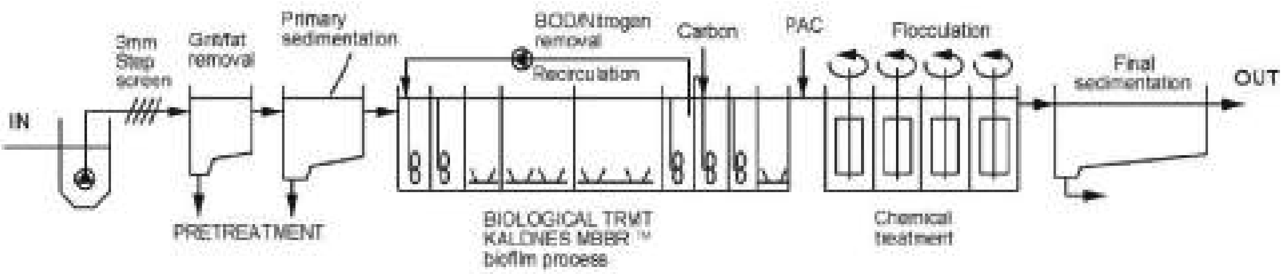
c. Kaldnes® process

The performance data and the schematic flow diagram of the Kaldnes process in Lillehammer STP of Norway, which applies the biological nitrification-denitrification and phosphorus removal by chemical precipitation, are shown in Table 5.65 and Figure 5.90.

Table 5.65 Performance of the Lillehammer Kaldnes process

	Influent (mg/l)	Effluent (mg/l)	Removal (%)
BOD_7	81.67	3.33	96
T-N	25.46	5.17	80
T-P	3.67	0.08	98

STP for Nitrogen and Phosphorus removal Lillehammer, Norway.
Dimensioning flow: 1,200 m^3/hr, Maximum flow: 1,900 m^3/hr
Design Temperature: 10ºC
Source: Kaldnes Web page



Source: Kaldnes Web page

Figure 5.90 Schematic flow diagram of Lillehammer Kaldnes process for removal of BOD and nutrients

5.18.13.4 Advantages and Disadvantages

The advantages and disadvantages of these MBBR processes are as follows:

a. Advantages

i. These processes can enhance the activated sludge process by providing a greater biomass concentration in the aeration tank and thus offer the potential to reduce the basin size requirements.

ii. Packing materials can maintain the concentration of nitrifying bacteria in the aerobic tank at a high level, and the nitrification reaction proceeds efficiently. Hence, these processes can improve the volumetric nitrification rates and accomplish the denitrification in aeration tanks by having anoxic zones within the biofilm depth.

iii. These processes can be used for the upgrading of existing STP, especially when space is an issue.

iv. These processes can upgrade the plant by reducing the solids loading on the existing sedimentation tank.

b. Disadvantages

i. Because of the complexity of the process and issues related to understanding the biofilm area and activity, the processes designs are empirical and based on prior pilot-plant or limited full-scale results.

ii. When upgrading existing treatment plants that operate without primary settling and rather large screen sizes, the carrier material should be chosen appropriately to prevent clogging.

iii. In the aeration tank of these processes, the concentration of dissolved oxygen (D O) has to be relatively high because the D O concentration is the limiting factor in the biofilm processes. A high driving force in terms of D O concentration across the biofilm is therefore required.

5.18.13.5 Typical Design Parameters

a. Linpor® process

Typical design parameters of the Linpor process are shown below:

i. MLSS suspended: 3,800 (mg/L)
ii. MLSS fixed: 15,000 (mg/L)
iii. MLSS total: 5,800 (mg/L)
iv. Carrier volume: 22 (%)
v. F/M: 0.12 (1/d)
(Gilligan and Morper, 1999)

b. Pegasus®/Bio-cube process

Typical design parameters of the Pegasus process are shown below:

i. HRT (Anoxic tank + Oxic tank): 6-8 (hours)
ii. Carrier volume: 10-20 (%)
iii. Circulation ratio of nitrified liquor to influent flow: 1.5-3.0 (-)
iv. Final sedimentation tank hydraulic application rate: about 1.0 (m/hr)
(Hitachi Plant Technologies, Ltd.)

c. Kaldnes® process

Typical design parameters of the Kaldnes process are shown below:

i. Anoxic detention time: 1.0-1.2 (hours)
ii. Aerobic detention time: 3.5-4.5 (hours)
iii. Biofilm area: 200-250 (m^2/m^3)
iv. BOD loading: 1.0-1.4 (kg/ m^3/d)
v. Final sedimentation tank hydraulic application rate: 0.5-0.8 (m/hr)
(Metcalf & Eddy, 2003)

5.18.13.6 Applicability

The MBBR processes are used in municipal sewage treatment for BOD removal and nutrients removal. These processes are frequently used for upgrading an existing plant, especially when space is an issue. The existing reaction tanks can be either retrofitted with Linpor®, Pegasus®, or Kaldnes® or similar other such technologies in the world. The one handicap is the inability to derive the design guidelines for an STP. This is because, depending on the media that is used the available area for microbes to grow in a given volume of the reaction tanks varies rather widely and this compounds the formulation of a fundamentally reliable design equation. At this moment, heavy reliance on the eventual builders of these MBBR type STPs and the realization as well as the intangibles of their guarantees seems unavoidable.

5.18.13.7 Guiding Principles

a. There are several types of MBBR processes and the design guidelines should be studied and evaluated consciously.

b. Life cycle of media of MBBRs is uncertain as of now and it appears that one possible method of sustaining competition may be to opt for a contract including the replacements for a specified number of years as part of the contract itself.

5.18.14 Fixed Bed Biofilm Activated Sludge Process

5.18.14.1 Description

The FBAS process is an essentially an activated sludge attached growth process where the plant roots provide the area for the biofilm to develop and grow. The aeration system is divided into a series of biological reactors where fixed biofilm is maintained in every stage of the process. Biodegradation of influent contaminants takes place mainly with the help of fixed biological cultures, where plant roots are used as biofilm carriers; additional textile media is used in the reactors as additional biofilm carriers. As a standard feature of the technology the reactors are covered by a shading structure or a greenhouse. As the influent travels through the cascade, the available nutrient quantity is consumed and as a result, the composition of the ecosystem fixed in the biofilm changes from reactor to reactor, gradually adapting itself to the decreasing nutrient concentration. In each cascade stage, a specially adapted ecosystem will form, thus maximizing the decomposition of contaminants. It is stated that 32 plants with such technologies have been set up in different countries including Hungary, China and France, etc. in last 10 years. However, it will be useful to demonstrate this project under Indian conditions.

(Organica™ technology) is stated to enhance the forces of nature to purify the sewage by harnessing the metabolic processes of living organisms that digest organic pollutants. In addition to the bacteria found in traditional activated sludge systems, the Organica STP are stated to be inoculated with 3,000 species of plants, animals, and microbes. The STP consist of a series of aerated reactors, filtration units and final polishing units. Plants with extensive root systems are placed on a supporting mesh slightly below the liquid level in the open aerobic reactors.

The roots of these plants, suspended 1.5 metres into the water, provide a healthy habitat for the bacteria and a whole range of other organisms such as protozoa, zooplankton, worms, snails, clams and even fish. As sewage flows through the technology train, different ecosystems develop in each tank. It is stated that there are two types of Organica treatment process. One is the Organica Fed Batch Reactor (FBR) process which combines conventional Sequence Batch Reactors (SBR) and continuous flow sewage treatment technologies. The other is the Organica cascade which is a continuous flow treatment process through a series of connected biological reactors.

5.18.14.2 Application Examples

a. Organica FBR (Fed Batch Reactor) process

i. Shenzhen, China

- Technology applied: Single train FBR process
- Footprint: 975 m^2
- Hydraulic Capacity: 400 m^3/day
- Community Served: 1,700 people

ii. Etyek, Hungary

- Technology applied: Two train FBR process
- Footprint: 570 m^2
- Hydraulic Capacity: 1,100 m^3/day
- Community Served: 10,000 people

iii. Le Lude, France

- Technology applied: Two train FBR process
- Footprint: 380 m^2
- Hydraulic Capacity: 815 m^3/day
- Community Served: 6,000 people

iv. Telki, Hungary

- Technology applied: Two train FBR process
- Footprint: 360 m^2
- Hydraulic Capacity: 800 m^3/day
- Community Served: 8,000 people

b. Organica cascade process

i. Budapest Hungary

- Technology applied: Cascade process
- Footprint: 340 m^2

- Hydraulic Capacity: 280 m^3/day
- Community Served: 6,000 people

ii. Szarvas, Hungary

- Technology applied: Cascade process
- Footprint: 540 m^2
- Hydraulic Capacity: 1,600 m^3/day
- Community Served: 10,000 people

Independent evaluation of the results of the performance is not readily traceable in literature.

5.18.14.3 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. The process is stated to require much lesser land area than conventional activated sludge

ii. The process is stated to be odourless and hence the STPs are stated to be easily built in urban area with no negative impact on the value of adjoining areas.

iii. It is stated that it can operate at a much lesser loading rates during initial days of setting up the plant in new habitations and it is stated that due to small area requirements, this technology can offer decentralized solutions and recycling water in local areas.

iv. The technology allows for design flexibility and can be adopted for nutrient removal such as phosphorous and nitrogen, which are today the major concern of pollution in rivers.

b. Disadvantages

i. In colder climates where the temperature drops to sub normal, the plants may have to be protected with a greenhouse otherwise the biota may freeze up.

ii. Because of higher automation, the technology is not attractive for smaller sizes of plants

iii. The technology requires more qualified operators than in other technologies.

iv. Yet to be validated on reasonable number and sizes of STPs in India

5.18.14.4 Applicability

Organica treatment plants are also on-site sewage treatment systems and the fate of disposal and regrowth of over grown plants is to be addressed with the possibility of biomethanation of the harvested over growths just like the biomethanation of fodder.

5.18.14.5 Guiding Principles

These require to have continuous feed for sustaining the microbes and as such are not recommended for places like hostels, which may lie vacant during holiday seasons and after exam seasons. These may be better for controlled housing colonies without industrial activity but only after an oil and grease trap and with potential for using the treated sewage for avenue trees by dedicated pipeline and root zone drip irrigation but are yet to be validated on reasonable number and sizes of STPs in India.

The toxic or otherwise of the inoculum of 3,000 species and their residues when discharged into aquatic environment like rivers, ponds etc are unknown at this stage and the treated sewage may have to be put through a toxicological clearance from a competent authority before the technology can be taken on board in JnNURM funded STPs.

5.18.15 Submerged Immobilized Biofilm Technology

5.18.15.1 Description

These are stated to be exfoliated bricks of volcanic ash which do not degrade by themselves but offer microbes have a chance to get into the crevices and stay there as immobilized habitats. These microbes bring about the aerobic anaerobic or facultative activity based on prevailing oxygen conditions or septic conditions. These are confined in application to small sized plants and polishing of sewage effluent from STPs. These are stated as patented makes. This can be recommended for outfalls of secondary effluent prior to discharge in the water bodies for polishing of effluents wherever required to meet the discharge standards.

An STP with Eco-Bio-Block (EBB) has been installed and functioning at Indian Institute of Science Education and Research (IISER), Mohali in small scale and the same needs to be evaluated. Further piloting is required. Based on the performance, the same may be recommended for on-site and decentralized wastewater management systems.

5.18.15.2 Application Examples

a. EBB performance test with Ministry of Land, Infrastructure, Transport and Tourism, Nobeoka City, Japan

EBB performance test was carried out by Nobeoka City with Ministry of Land, Infrastructure, Transport and Tourism using the EBB blocks at a river in Nobeoka City.

i. Study year: 2001

ii. Test area of river: 2m width, 30m length

iii. Type of EBB block: EBB 300 (300×300×60mm)

The performance of EBB blocks in treating the drain sewage is mentioned in Table 5.66 overleaf.

Table 5.66 Performance of EBB test in Nobeoka city

	Before EBB	After EBB	
		7days	14days
pH	6.9	6.6	6.5
DO (mg/l)	6.0	7.4	7.5
BOD (mg/l)	2.5	0.7	0.7
COD_{Mn} (mg/l)	8.3	2.7	4.0
SS (mg/l)	110	10	5
Total coliform (10^2) (MPN/100ml)	7.9	4.6	1.3
T-N (mg/l)	1.30	0.39	0.30
T-P (mg/l)	0.34	0.096	0.041
Turbidity	31	5.4	2.8

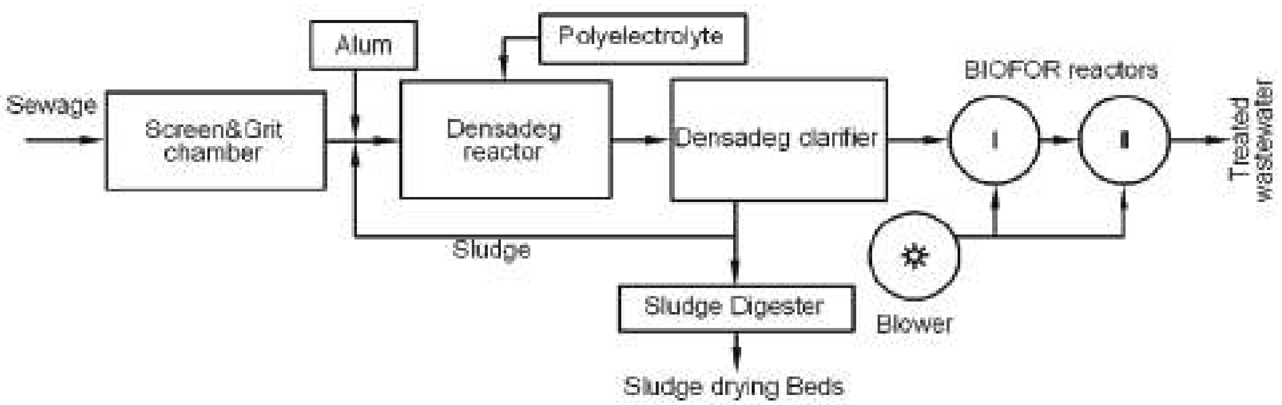
Source: http://www.ariakeebb.com

5.18.16 BIOFOR Technology (Biological Filtration and Oxygenated Reactor)

5.18.16.1 Key Features of the Technology

- Enhanced primary treatment with addition of coagulants and flocculants
- High rate primary tube settlers and integrated thickening offering space economy
- Two stage high rate filtration through a biologically active media and with enhanced external aeration
- Co-current up flow movement of sewage and enable higher retention and contact
- Treatment scheme excluding secondary sedimentation but recycling of primary sludge
- Deep reactors enabling low land requirements
- A compact and robust system

The process flow diagram of BIOFOR technology is presented in Figure 5.91.



Source: MoUD, 2012

Figure 5.91 Process flow diagram of BIOFOR Technology

5.18.16.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Compact layout because of high rate processes.
ii. Higher aeration efficiency through co-current diffused aeration system
iii. Space saving as secondary sedimentation tank is dispensed
iv. Ability to withstand fluctuations in flow rate and organic loads
v. Compliance with stricter discharge standards
vi. High quality effluent for reuse without separate nutrient removal and fine filtration
vii. Effluent suitable for UV disinfection without filtration
viii. Absence of aerosol and odour nuisance in the working area
ix. Absence of corrosive gases in the area
x. Lower operation supervision enables lesser manpower requirement

b. Disadvantages

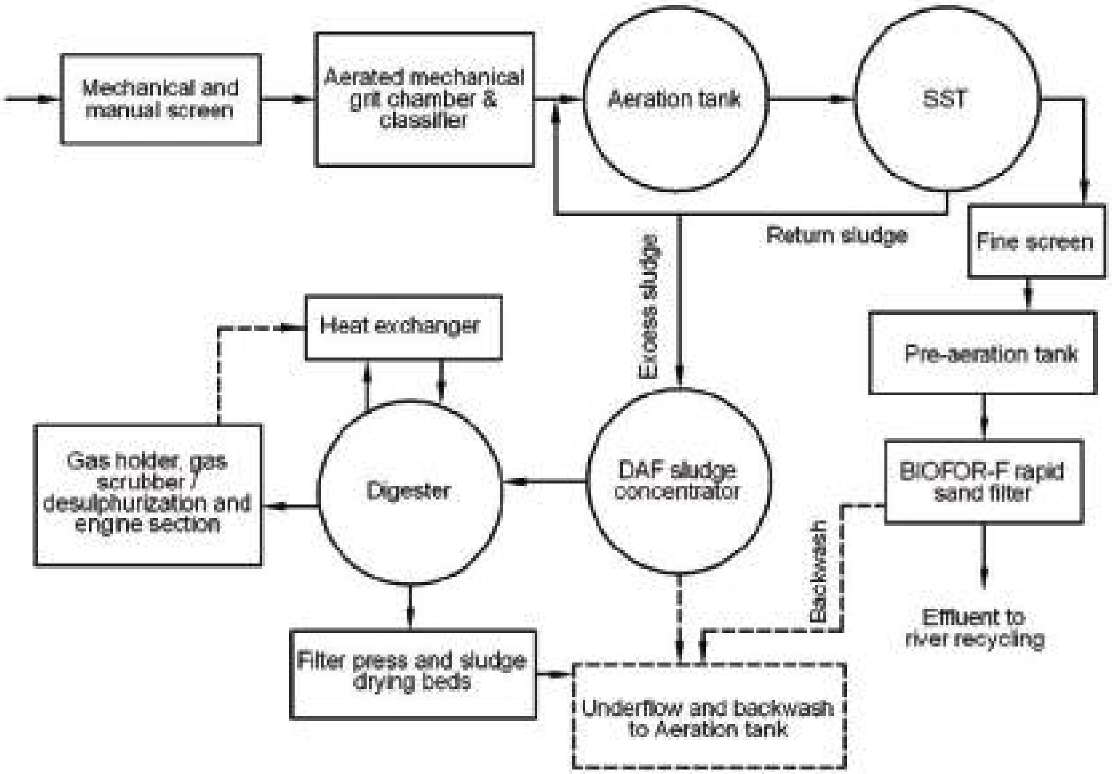
i. Continuous and high chemical dosing in primary clarification
ii. Large sludge generation due to the addition of chemicals
iii. Undigested sludge from primary clarification requiring post treatment
iv. Yet to be validated on reasonable number and sizes of STPs in India.

5.18.17 High Rate Activated Sludge BIOFOR-F Technology

5.18.17.1 Key Features of this Technology

- In general, high level of mechanization and sophistication
- The flow scheme excludes primary sedimentation tank
- Superior aerated grit chamber and classifier
- Circular aeration tank with tapered air diffusion system
- Second stage aeration and rapid sand filtration through a biologically active filter media
- Dissolved air floatation for sludge thickening
- Digester heating and temperature controlled anaerobic sludge digestion
- Mixing of digester contents through biogas
- Dynamic cogeneration of electrical and thermal energy through gas and dual fuel engines

The process flow diagram of High Rate Activated Sludge BIOFOR-F Technology is presented in Figure 5.92 overleaf.



Source: MoUD, 2012

Figure 5.92 Process flow diagram of High Rate Activated Sludge BIOFOR-F technology

5.18.17.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Compact layout as a result of high rate processes

ii. Higher aeration efficiency through diffused and tapered aeration system

iii. Space saving as primary sedimentation is dispensed

iv. Compliance with stricter discharge standards

v. Effluent suitable for high end industrial applications

vi. Stable digester performance and consistent gas production

vii. Almost self-sufficient in energy requirement due to gas engine based cogeneration system

viii. Absence of aerosol and odour nuisance in the working area

b. Disadvantages

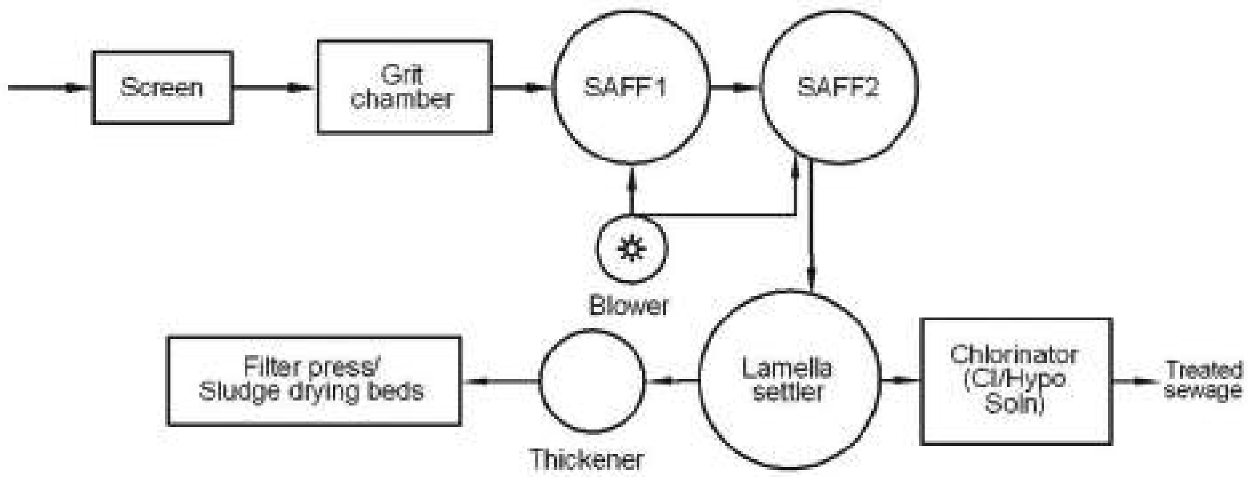
i. None, except high cost

5.18.18 Submerged Aeration Fixed Film (SAFF) Technology

5.18.18.1 Key Features of the Technology

- Essentially a fixed film media with enhanced oxygen supply through submerged aeration
- Unconventional plastic media offering high void ratio and specific area compared to stone and aggregates
- Large biomass and long solid retention time in the reactor leading to low 'food to micro-organism ratio' and higher organic removal
- Two stage biological oxidation
- Treatment scheme excluding primary sedimentation and sludge digestion
- Reactors up to 6 m deep enabling low land requirements
- Many plants based on such technology are functioning in industrial wastewater applications. Pilot study is required for municipal sewage applications.

Process flow diagram of Submerged Aeration Fixed Film (SAFF) Technology is presented in Figure 5.93.



Source: MoUD, 2012

Figure 5.93 Process flow diagram of Submerged Aeration Fixed Film (SAFF) Technology

5.18.18.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

i. Deep reactors enabling small space requirements

ii. Ability to effectively treat dilute domestic sewage

iii. Low and stabilised sludge production eliminating the need for sludge digestion

iv. Absence of odour and improved aesthetics

v. Absence of emission of corrosive gases.

b. Disadvantages

i. Clogging of reactor due to absence of primary sedimentation

ii. Reliance on proprietary filter media.

iii. Strict quality control on media.

iv. High reliance on external energy input.

v. Requires skilled manpower.

vi. Yet to be validated on reasonable number and sizes of STPs in India

5.18.18.3 Applicability

The SAFF technology based system is particularly applicable for:

- Small to medium flows in congested locations
- Sensitive locations
- Decentralised approach
- Relieving existing overloaded trickling filters.

5.18.19 Rim Flow Sludge Suction Clarifiers

These are clarifiers with inlet along the rim and sludge is sucked out at the floor through suction boxed arms instead of scrappers and is reported to save on foot print and denser sludge and quicker return to aeration tank without lysis of the live sludge.

5.18.19.1 Advantages

- It is claimed that given the same clarifier volume as conventional centre feed clarifiers, these types of clarifiers can handle much higher throughputs and the rising sludge phenomenon is minimized.
- The need for a buried central feed pipe in large central feed clarifiers is avoided.
- The sludge is sucked out as soon as it settles on the floor and transferred to aeration tank and thus avoiding cell lysis.

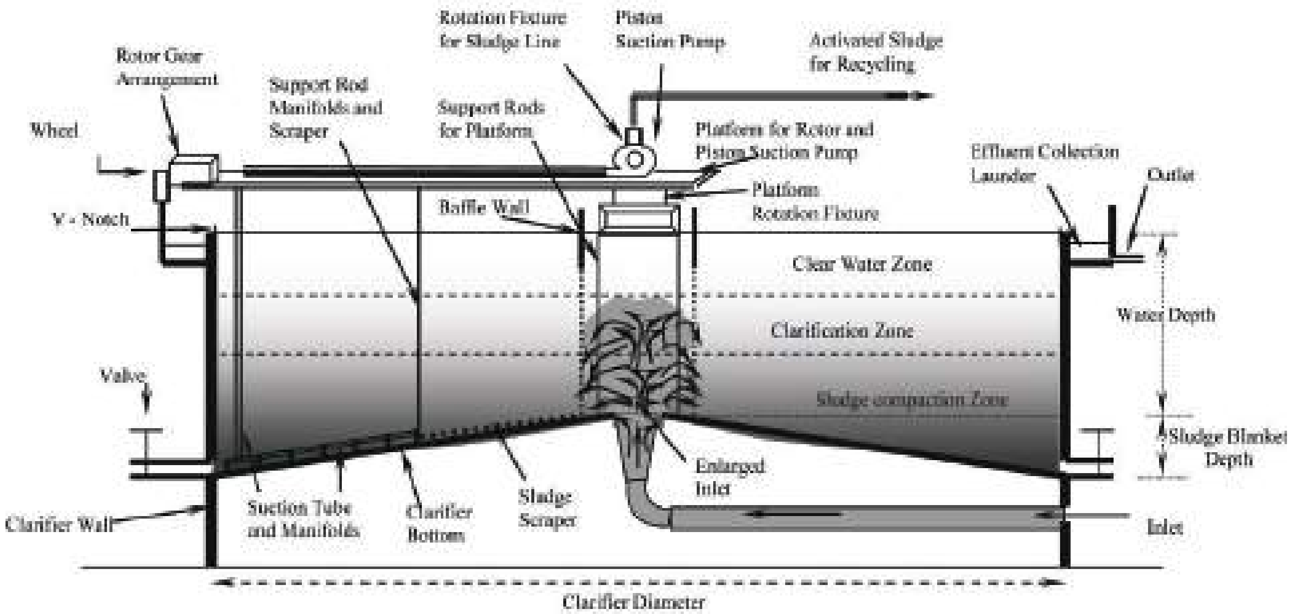
5.18.19.2 Disadvantages

- Here again, each vendor advocates their own criteria for the equipment and their types which makes it difficult to bring about a common and validated design criteria.
- The sludge suction arrangement if it gets into repair necessitates the emptying of the clarifier for repairs.

5.18.20 Improved Circular Secondary Clarifier (HYDROPLUME®) CSIR-NEERI

The conventional secondary clarifiers do not take hydraulic energy dissipation into account. They are either too large or often fail in giving the efficient solids-liquid separation. In this endeavour, CSIR - NEERI has developed a clarifier design radically different from the conventional circular clarifiers. It is called as HYDROPLUME®, which is an effective hydraulic energy dissipating, solids contact and sludge recirculation type high rate secondary clarifier that provides natural flocculation through plume formation.

It produces excellent effluent quality and helps in attaining the treated effluent quality conforming to discharge standards and the settled sludge is removed through a specially designed suction mechanism. The sludge removal mechanism is designed and fabricated to remove sludge from all around the clarifier and discharging it from a stationary outlet as depicted in Figure 5.94.



Source: Pophali et al., 2009

Figure 5.94 Sectional Drawing of HYDROPLUME® Clarifier showing the principle of functioning of plume formation, solids-liquid separation and sludge withdrawal (US Patent No. 7637379 B2)

5.18.20.1 Salient Features of HYDROPLUME®

- Hydrodynamics- Optimization of velocity gradient and hydraulic energy dissipation to ensure natural flocculation
- Geometry- Provision of an improved inlet design and bottom to enhance the solids-liquid separation and facilitate sludge removal
- Sludge Removal Mechanism- Development of an improved sludge removal suction mechanism to remove the settled sludge

5.18.20.2 Advantages

- Improved solids-liquid separation ensures minimum SS in the treated effluent
- High underflow solids concentration minimizes pumping rate, and maintains desired active biomass concentration in aeration tank
- Requires less surface area and operates at low hydraulic retention time (1.5 – 2.0 hrs HRT), thereby facilitates savings in capital cost
- It does not require a separate sump cum pump house for sludge recycling / removal and thereby saves capital and recurring costs
- It provides natural flocculation and does not require separate flocculation facility and thereby reduces capital and recurring cost
- The design has been validated using computational fluid dynamics studies for its plume behaviour and hydraulic energy dissipation besides other parameters.

5.18.20.3 Applicability

The next step is to take it up in field scale trials for firming up the design criteria for use in STPs.

5.19 ADDRESSING THE RECENT TECHNOLOGIES IN CHOICE OF STP

A reference is made to the advisory on recent trends in technologies in sewerage system issued by the MoUD in March 2012, to encourage the implementers in the field to innovate and explore new technologies as well as Public Private Participation (PPP) models without compromising on the basic safeguards both technical and financial. It states that, “There are various technology options available for treating sewage. The technology option as well as the project cost would be outlined in the detailed project report prepared for implementing the project. Irrespective of the technology chosen, STP projects could be developed on a long term commitment from the private sector partner either on PPP / build own operate transfer (BOOT) basis or on engineering procurement construction (EPC) plus O&M for 15 years where a part of the EPC cost is payable over a long-term O&M period. However, it is suggested that no new technologies will be considered under EPC contract”.

Box No. 5.1 Honouring Edward Ardern, MSc and William T. Lockett, MSc

It was on April-3-1914 that Edward Ardern, MSc and William T. Lockett, MSc, presented their paper titled “Experiments on the Oxidation of Sewage without the Aid of Filters” (at the Society of Chemical Industry in Manchester, England), in which they made the first reported use of the term “activated sludge” to refer to biological solids that they settled out of aerated sewage and recycled these back into the treatment process. Almost all the aerobic biological treatment processes to date, trace their lineage to this epoch making invention of the Activated Sludge Process. They continued to lead distinguished research and are reported to have published over 25 papers.

This manual honours them in the centenary year of 2014.

CHAPTER 6: DESIGN AND CONSTRUCTION OF SLUDGE TREATMENT FACILITIES

6.1 THE APPROACH

In STP, sludge means the following.

- Primary sludge – When raw sewage is settled in a primary clarifier, the suspended solids settle down by gravity. These are drawn out from the conical floor of the clarifier. This is called primary sludge (PS). It will have mostly organic substances and also inorganic substances. If it is stored, the organic substances will undergo anaerobic reaction as in Figure 5.2. This will result in production of Methane and Hydrogen Sulphide gases.

- Secondary sludge – When the sewage is aerated in aeration tanks, biological microorganisms grow and multiply. The aerated liquid is called the mixed liquor. It is settled in secondary clarifiers to separate the microorganisms by gravity. These are drawn out from the conical floor of the clarifier. This is called secondary sludge.

- Return sludge – A major portion of the secondary sludge is returned to the aeration tank for seeding the microorganisms. This is called return sludge (RS).

- Excess sludge – A small portion of secondary sludge is wasted. This is equal to secondary sludge minus return sludge. This is called excess sludge (ES) on waste sludge (WS).

- Chemical sludge – When raw sewage or secondary treated sewage is subjected to chemical precipitation, the resulting sludge is called chemical sludge (CS).

In treatment units such as MBR, there are two optional arrangements for separating the treated sewage from the aerated mixed liquor.

In one type, the filtration membranes are submerged into the mixed liquor and the treated sewage is sucked out as the filtrate. In this case, there is no secondary sludge or return sludge. The mixed liquor itself is separately wasted as excess sludge.

In another type of MBR, the membranes are outside the aeration tank and the mixed liquor is filtered into treated sewage and secondary sludge.

The primary and excess sludge are to be further treated to produce fully inert matter which will not decay any further. Normally this is achieved by the treatment process in Figure 5.2. Here the organisms themselves are food source for new organisms till almost all organisms are reduced. This process also produces methane and hydrogen sulphide gases. The hydrogen sulphide is removed and the methane is sent to gas engines to generate electricity.

Alternatively, it can also be achieved by the treatment process in Figure 5.1, but this will need aeration and hence electrical energy is to be spent.

The typical sludge generation values are shown in Table 6-1 overleaf.

The illustrative computation of sludge generation values from ASP is in Appendix A.6.1.

Table 6.1 Typical sludge generation values

Process	kg Sludge/kg BOD removed	
	as VSS	as TSS
Primary alone	1.92	3.20
Primary and secondary as conventional ASP	0.86	1.31
Extended aeration ASP	0.26	0.93

The capacity of each sludge treatment unit is determined by considering the operating hours, sludge moisture content, retention time, etc., and is based on the solids balance of the entire sludge treatment facility. The solids balance considers the reduction due to gasification, return load from each facility and the increase due to addition of chemicals, etc. This is important for sizing the sludge treatment units.

In general, the primary and excess sludge will need a blending tank before further treatment so that the properties are made almost uniform when feeding the units. After this, if the thickened sludge is put through anaerobic digestion as in Figure 5.2 for producing Methane, it is called anaerobic digester. If it is oxidized as in Figure 5.1, it is called aerobic digester. In both cases, the digested sludge will have to be dewatered. There are many types of equipments like centrifuge or filter press or natural solar drying beds for this purpose. The solids concentrations by different treatment processes are listed in Table 6.2.

Table 6.2 Solids concentration by treatment process

Treatment process	Solids concentration (%)	
	Excess sludge	Mixed sludge
Conventional activated sludge process	0.5-1.0	1.0
Oxidation ditch process	0.5-1.0	-
Extended aeration process	0.5-1.0	-
Sequencing batch reactor process (low load)	0.5-1.0	-
Biological aerobic filtration process	-	1.0
Contact oxidation process	0.8	1.0

Source: Guideline and Manual for Planning and Design in Japan, JSWA, 2009

An illustrative solids balance in two different types of sludge treatment processes is shown in Figure 6.1 overleaf. In this figure, a pertains to direct dewatering and incineration and b pertains to the case of digestion, dewatering and incineration.

Appendix A.6.2 illustrates the calculations further.

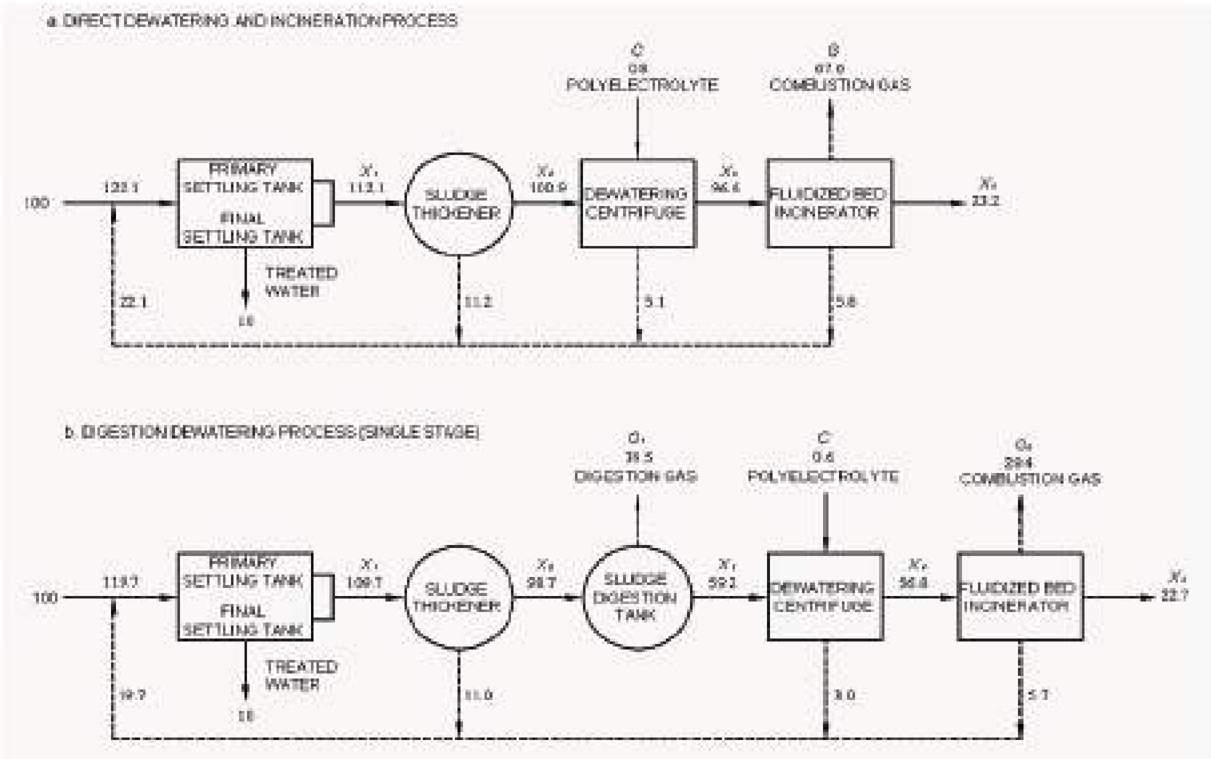


Figure 6.1 Example of solids balance (Conventional Activated Sludge Process)

In case of option a (dewatering and incineration) the solids balance occurs as follows.

- The solids load from the STP to the sludge treatment section is taken as 100.
- The solids load from the recirculation from the various units is taken as 22.1.
- Thus, the solids load entering the sludge treatment units is 122.1
- The solids load escaping in the liquid portion from the blending tank is taken as 10
- Thus, the solids load entering the sludge thickener becomes 112.1
- The solids load escaping in the liquid portion from the thickener is taken as 11.2
- The solids load leaving the thickener becomes 100.9.
- In the dewatering centrifuge, solids load from polyelectrolyte is 0.8
- Thus solids load entering the dewatering centrifuge is 101.7
- The solids load in the dewatered cake is 96.6
- The solids load of 5.1 is in the filtrate and is recirculated
- The solids load of 96.6 in the dewatered cake is sent to fluidized bed and incinerator
- Here, the solids load of 67.6 goes into the formation of combustion gases
- The solids load of 5.8 escapes in the liquid portion of the fluidized bed
- The solids load of 23.2 remains in the final product as ash.

Similarly, in the case of process b, the solids load in the final product becomes 22.7 instead of 23.2 in process a. Moreover, a digestion gas equivalent of 39.5 is gained, which in turn is a source of energy.

Thus, anaerobic digestion has its importance. The solids recovery rate varies at each stage of sludge treatment and is shown in Table 6.3.

Table 6.3 Example of solids recovery rate in each treatment stage

Process	Solids recovery rate	
Sludge thickening	Gravity thickening Centrifugal thickening Air floatation thickening (dispersed air) Gravity belt thickening	80 to 90% 85 to 95% More than 95% More than 95%
Sludge digestion	Sludge reduction ratio due to formation of gas, etc.	30 to 40%
Sludge dewatering	Pressure-type screw press dewatering Rotary pressure dewatering Belt press dewatering Centrifugal dewatering	More than 95% More than 95% 90 to 95% More than 95%
Sludge incineration	Sludge reduction ratio due to formation of gas, etc. Recovery rate	40 to 80% 80 to 90%

Source: Guideline and Manual for Planning and Design in Japan, *JSWA*, 2009

6.2 HYDRAULICS OF SLUDGE PIPELINES

6.2.1 Sludge Piping

Sludge piping can be by gravity or by pumping. For example, when primary sludge is drawn from clarifiers, it is sometimes by gravity and sometimes by direct suction using pumps. The friction loss in gravity pipelines and pumped pipelines are calculated as follows.

6.2.1.1 Friction losses in Gravity Sludge Pipelines

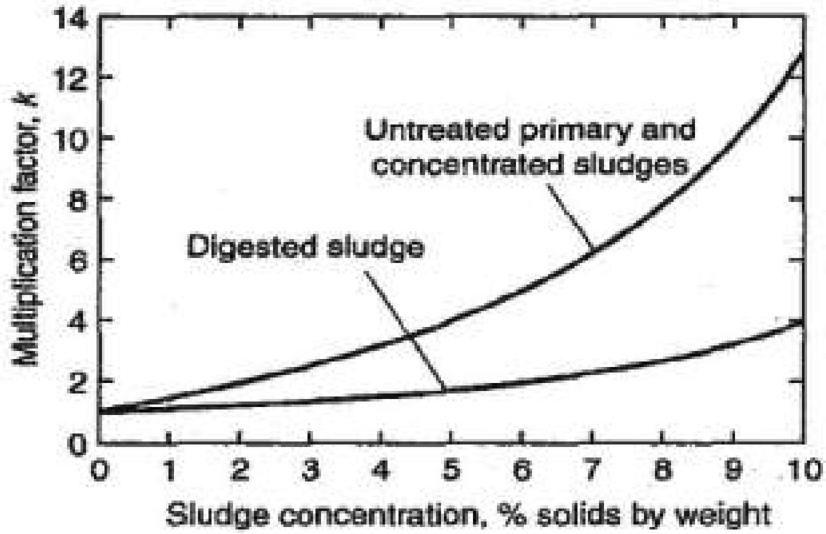
1. Calculate friction loss using Manning’s formula as though it is flowing with water.
2. This friction loss is multiplied by a friction compounding factor (F) as follows
3. Estimate the solids content in the sludge (P).
4. F for undigested sludge is = 2.88 + (0.176×P×P) – (0.866×P)
5. F for digested sludge is = 1.52 + (0.041×P×P) – (0.227×P)
6. Allow for an additional factor of safety of 10%.

6.2.1.2 Friction Losses in Pumped Sludge Pipelines

First calculate the head loss as though it is pumping water by using the Hazen Williams formula with the value of C taken from Table 6.4 overleaf. Then, multiply the head loss by the factor k from Figure 6.2 for the given solids content (P).

Table 6.4 Hazen Williams value of C for sludge flows

Sludge solids (%)	Raw Sludge	Digested Sludge	Sludge solids (%)	Raw Sludge	Digested Sludge	Sludge solids (%)	Raw Sludge	Digested Sludge
0	100	100	4	53	78	8	33	60
1	83	100	5	47	73	9	29	55
2	71	91	6	42	69	10	25	48
3	60	83	7	37	65			



Source: Metcalf & Eddy

Figure 6.2 Head loss Multiplication Factor for Different Sludge Types and Concentrations

The equations for these curves are simplified as follows where P is the % of sludge solids.

k for undigested sludge = $0.125P^2 - 0.1656P + 1.5733$
k for digested sludge = $0.0354P^2 - 0.0699P + 1.0858$

The calculations for these friction losses are illustrated in Appendix A 6.3.

6.2.2 Sludge Pump Types and Applications

There are specific considerations to be borne in mind in the use of different types of pumps for handling sludge. The relative applicability of these is shown in Table 6.5.

The illustrations of the internal arrangements of these are compiled in Figure 6.3 (overleaf) a to d and indexed to serial numbers in Table 6.5 for an easier visual understanding of these. In respect of impellers in centrifugal pumpsets, the rotary speed is advised not to exceed 960 rpm especially when pumping return sludge.

Table 6.5 Types of sludge pumps and their application

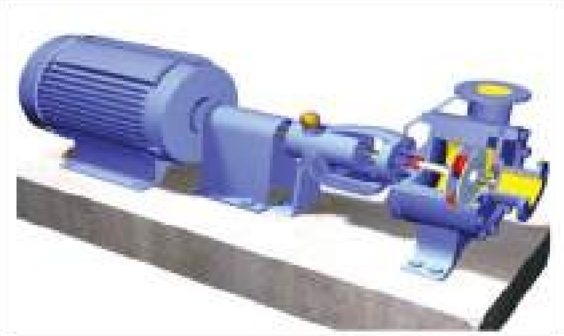
No	Type of Pump	Primary sludge	Return sludge	Excess sludge	Chemical sludge	Thickened sludge	Digested sludge
1	Air lift		✓	✓			
2	Archimedean screw		✓	✓			
3	Centrifugal	✓	✓	✓	✓		
4	Double diaphragm	✓	✓	✓	✓	✓	✓
5	Plunger	✓			✓	✓	✓
6	Progressive cavity	✓	✓	✓	✓	✓	✓
7	Reciprocating piston					✓	✓
8	Rotary-lobe				✓	✓	✓
9	Single diaphragm	✓					
10	Screw centrifugal	✓	✓	✓			
11	Torque flow	✓	✓	✓	✓		✓

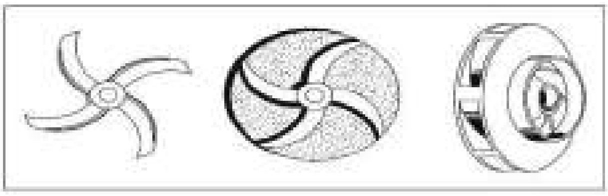
http://www.axflow.com

http://www.axflow.com

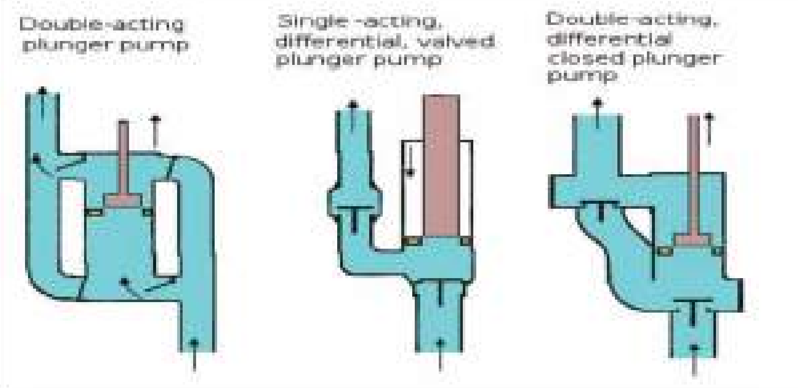
www.weirpowerindustrial.com

Torque Flow Pumps-The energy is stated to be transmitted to the liquid by an induced vortex motion (11)





Horizontal Foot Mounted Centrifugal Pump.(3)
Open, semi open and closed impellers.
Source: Photo is from Wikipedia.

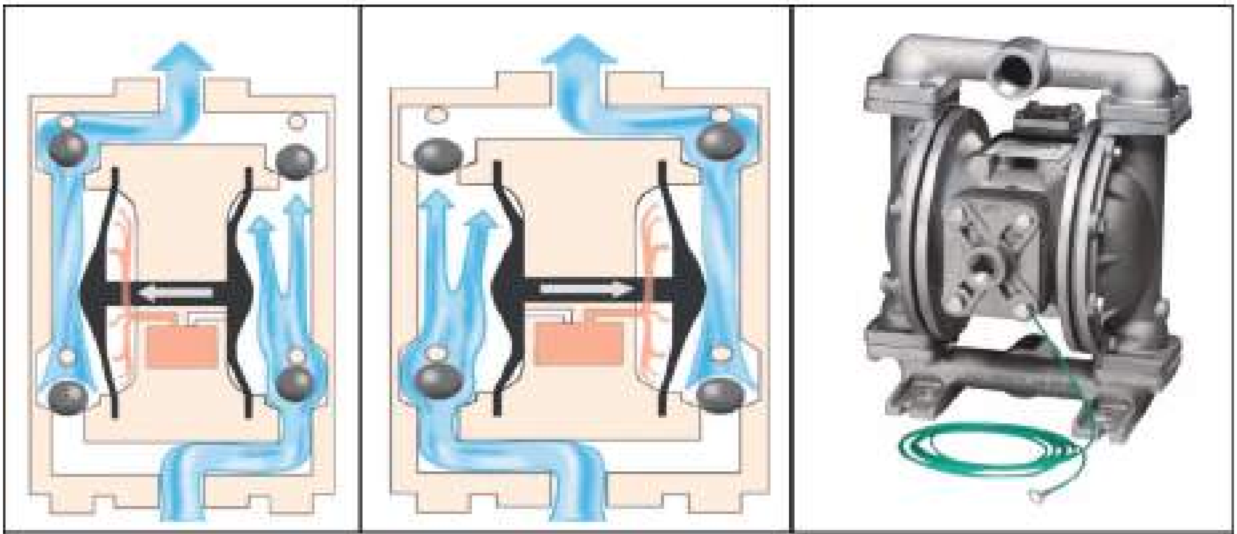


Plunger Pump- Source: Photo from a public website & drawing from http://www.cd3wd.com.(5)

Stator-Rotor pump or Progressive Cavity pump.(6)
Source: Drawing from http://news.thomasnet.com. Photo from http://www.globalpumps.com.au

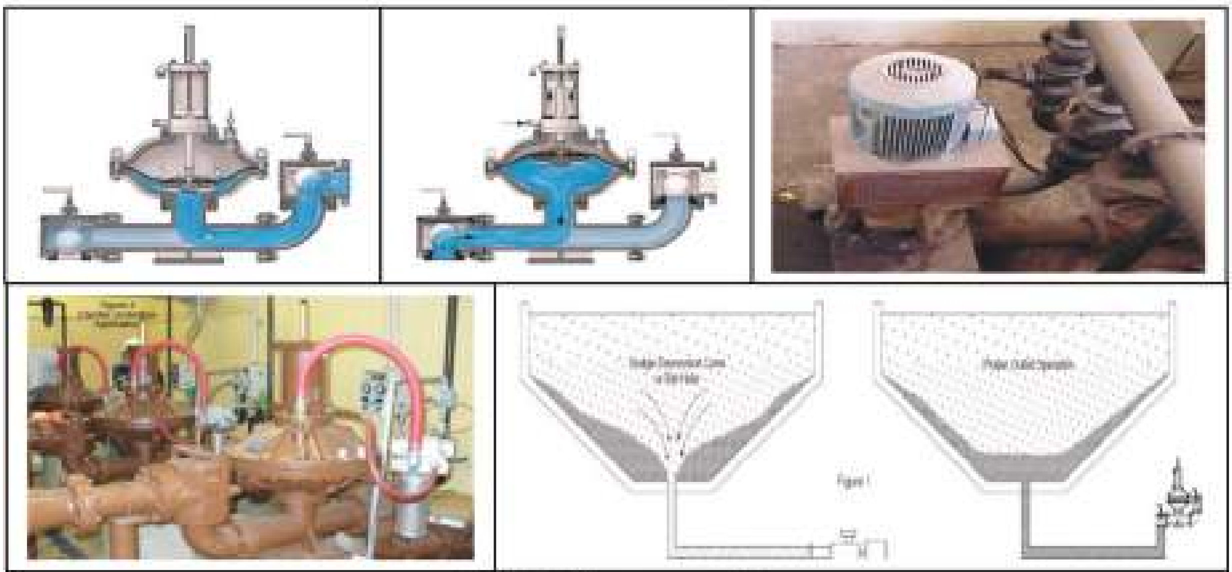
Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 6.3 a Types of sludge pumps



Double acting Diaphragm pumps (4).

When the diaphragm set is pushed to the right, it displaces the sludge upwards on that side as the ball valve below prevents the sludge from going downwards. After this, when the diaphragm set is pushed to the left, it displaces the sludge already in the left compartment upwards as the corresponding ball valve prevents the sludge going downwards. At this time, sludge is drawn into the right side compartment by pushing the ball valve upwards. Thus, a near continuous sludge displacement is got. The diaphragm to and fro movement is by pneumatic. Source: Drawing from http://www.tapflo.com. Photo from http://www.coleparmer.com



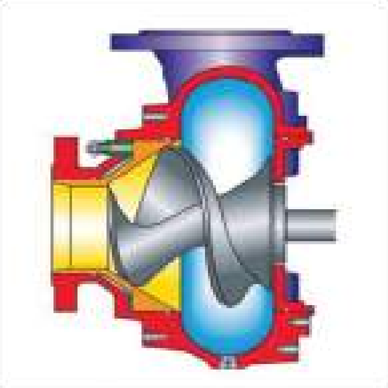
Single diaphragm pump (9).

Only one diaphragm is used. Thus, sludge drawal can be intermittent. A photo of an installation during mid 1990's at the Nesapakkam STP of CMWSSB for primary clarifier sludge is shown in upper row. These suit primary clarifier sludge drawal from small sized clarifiers as the rate of sludge accumulation is much lesser than the commercially available sludge pump sets. When such pumps are started, initially the settled sludge is drawn but soon after the clarifier liquid starts flowing out. This is called as the cone of depression. These can be operated intermittently to allow sufficient accumulation of sludge each time the pump is activated. This can be done by electrical or pneumatic control. Source: From public websites & courtesy CMWSSB.

Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 6.3 b Types of sludge pumps-continued

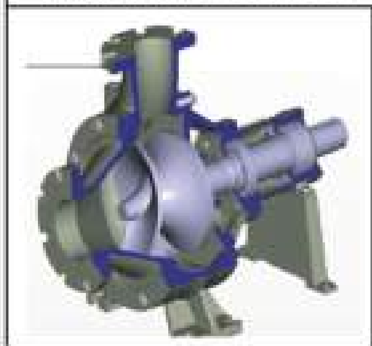
http://www. lawrencepumps.com



http://www.hidrostal.co.uk

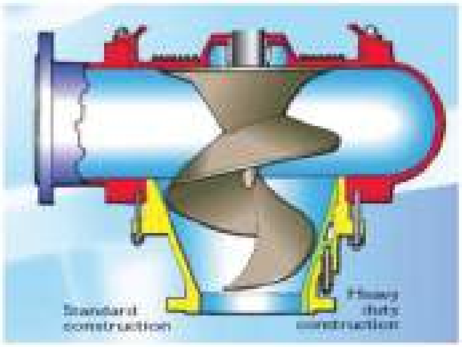
http://www.ib-pompen.nl/EN

http://pumps.kmspecialty.com



http://www.gobizkorea.com

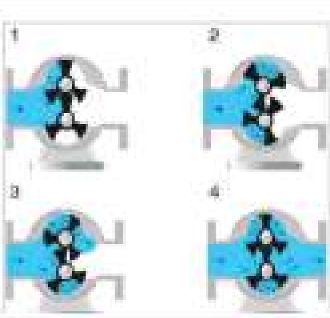
http://www.chopperpumps.com



Vertical screw impeller mounting http://www.hidrostal.co.uk

Vertical screw centrifugal pump for primary clarifier sludge in CMWSSB Koyambedu STP in the 1990's

The screw centrifugal impellers with non clog properties of the impellers (10)

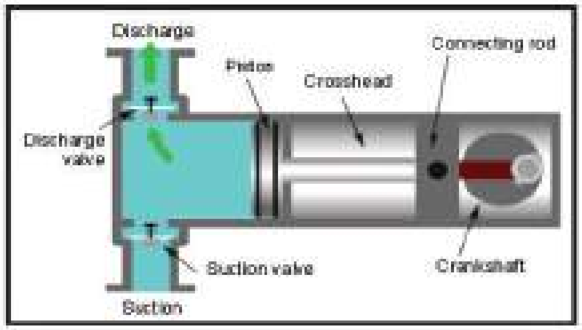


The liquid flows into the cavity and is forced through the outlet as the lobes engage. The rotary lobe pump. (8)

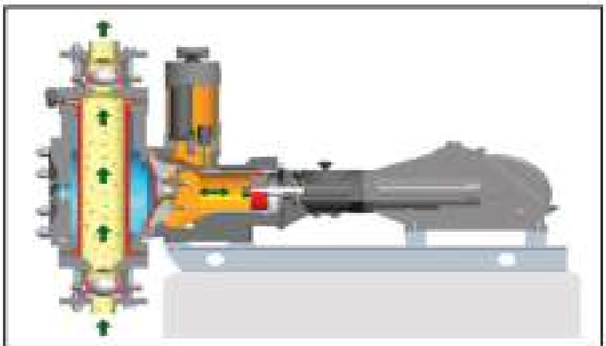
Archimedean screw pumps for (RS) at Koyambedu STP as in mid 1990's (2)

Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 6.3 c Types of sludge pumps-continued

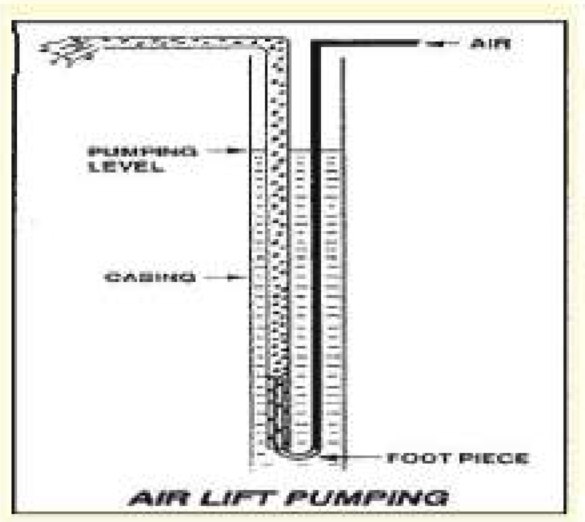


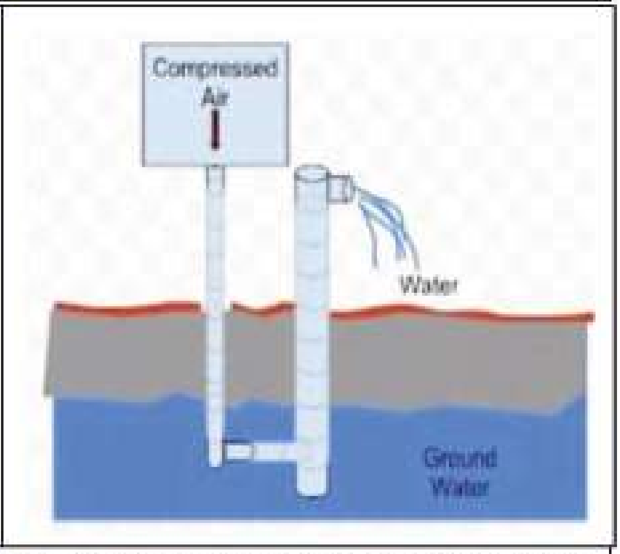
http://toolboxes.flexiblelearning.net.au/demosites/series2/204v2/PROC201/PROC201-010100-RecognisePumps.htm



(8) Reciprocating piston pump, http://www.feluwa.de/cn/prozesstechnik/sonderanlagen/

Reciprocating piston pump (7)





http://ecosustainablevillage.com.ip01-web23.net/air_lift_water_pumps.htm and www.meritnation.com
Airlift pump (1)

Illustrations are only for familiarity of explanations & not standalone endorsements

Figure 6.3 d Types of sludge pumps-continued

6.2.2.1 Centrifugal Pumps

Centrifugal pumps (as in notation 3 in Table 6.5 and Figure 6.3 a) for handling sludge must be of the non-clog type. They should be robust and should have easily accessible hand-holes for cleaning. Pumps of the macerator type impeller or additional cutters with a cutting ring whereby stringy rags and other fibrous material can get shredded are preferable. When the specific speed of the pump is low, non-clog impellers are designed with less number of blades than in impellers for handling clear liquids. In pumps of high specific speed, the mixed flow impeller should generally have wide passages. Centrifugal pumps with non-clog impellers have less efficiency than those of normal design for handling clear liquids. The rating for the drive motor has to be selected keeping this in mind. The specific speed of the pump also affects the suction-lift capability of the pump. This can be overcome by selecting a vertical centrifugal pump to be so installed that the impeller would be adequately submerged always. The impellers are best chosen as open impellers or semi open impellers or centrifugal screw impellers.

6.2.2.2 Air-Lift Pumps

Air lift pumps (as in notation 1 in Table 6.5 and Figure 6.3 d). These are used in small extended aeration plants to return the sludge and scum to the aeration tank. Small air bubbles are formed in the liquid, which makes the air-water mixing less dense to get lifted to the discharge point.

A compressor / blower supplies the air. Air-lift pumps and ejectors are pumping systems, which are though inherently inefficient, there are no moving parts inside and hence, their operation is fairly trouble-free.

6.2.2.3 Screw Pumps

These are three major variations of these as under

1. Archimedean Screw (as in notation 2 in Table 6.5 and Figure 6.3 c)
2. Stator-Rotor or Progressive Cavity (as in notation 6 in Table 6.5 and Figure 6.3 a)
3. Centrifugal Screw Impeller (as in notation 10 in Table 6.5 and Figure 6.3 c)

In the Archimedean screw pumps, the sludge enters the screw pump by a screw conveyor, which moves solids to an open impeller and lifts them to the point of discharge. The submerged lower bearing is of the enclosed and sealed type and the upper bearing is usually grease-lubricated with anti-friction bearing. The discharge rate plotted on x axis and head plotted on y axis will be a horizontal line in these pumps. In general, these are ideal for return sludge because it permits incidental additional aeration over the screws and rotates at gentle speeds of just about 20 or 30 rpm. They also permit visual inspection of the sludge.

A variation of the screw pump is the stator rotor or progressive cavity pump. The pumping element is a helical rotor of steel. It has a compressible stator or lining inside a cast iron body and is contoured to mesh with the helical rotor. Although the pump has some self-priming capability, the rotor must never run dry against the rubber stator. The pump can pump forward or reverse depending upon the direction of rotation. They are not advised for return sludge as the live organisms will be squeezed in the stator and rotor.

The centrifugal screw impeller has a shape of an Archimedean screw with widening diameter of each successive spiral. They can be mounted both vertically and horizontally. The impeller weight is usually heavier as compared to other screw pumps. They however, have the advantage of a truly non-clog design and are especially suited for sludge drawal from primary clarifiers because the possible fibrous materials or rags etc which might get into the clarifier sludge will be gliding over the impeller screw and are pushed out without choking the impeller. They are also useful in return sludge pumping as the live organisms do not get hit at the impeller or casing surfaces.

6.2.2.4 Reciprocating Plunger

Plunger type pumps (as in notation 5 in Table 6.5 and Figure 6.3 a) have a plunger reciprocating in a cylinder. A pump can have one or more plungers connected to common crankshaft, thereby obtaining arrangements called simplex, duplex, triplex, etc. Their capacities are of the order of 150 to 250 rpm per plunger. The pump speeds should be between 40 to 50 rpm.

They are self-priming and can usually work well with suction-lifts up to 3m. The suction-lift capability depends on the design of the pump, especially the suction valve. The pumps can develop high heads and are hence, suitable where accumulation of grease in piping can cause progressive increase in head.

However, if the delivery piping is likely to get choked, the pumps may develop very high pressures and this can cause a burst. A relief valve is provided to protect the pump in case of a clogged delivery piping after each use. The pump should be flushed so that no solids settle in the cylinder which could damage the pump during the next start. The suction and delivery valves are the main source of trouble. The valves should be easily accessible for quick cleaning, in case the valves fail to seat properly.

6.2.2.5 Diaphragm Pumps

The diaphragm pumps (as in notation 4 & 9 in Table 6.5 and Figure 6.3 b) have a flexible diaphragm, usually of rubber and actuated by a reciprocating movement. They can be either a single diaphragm or double diaphragm type.

The diaphragm is fastened peripherally to the casing, which also houses the suction and delivery valves. The interesting feature of the diaphragm pumps is that the components of the reciprocating mechanism, which are most prone to wear, are isolated from the path of the sludge. Pneumatic or hydraulic drives can also be employed for the reciprocating movement. These are suited for intermittent pumping of primary clarifier sludge.

6.2.2.6 Torque Flow Pumps

The torque flow pumps (as in notation 11 in Table 6.5 and Figure 6.3 a) can handle solids up to the full delivery bore size on the discharge side. The energy imparted to the liquid is by the principle of hydro-dynamic liquid coupling The shape of the impeller helps to generate the necessary swirl inside the casing and this acts as a pumping impeller component.

It is stated as a non-clog concept in pumping. In essence, the sludge that is drawn on the suction side is made to pick up the energy and glide around the inside of the volute before going out with the delivery head.

6.2.2.7 Rotary Lobe pumps

Rotary Lobe Pumps (as in notation 8 in Table 6.5 and Figure 6.3 c). Liquid flows around the interior of the casing, but without making contact. This is prevented by external timing gears located in the gearbox. Pump shaft support bearings are located in the gearbox, and since the bearings are out of the pumped liquid, pressure is limited by bearing location and shaft deflection.

As the lobes come out of mesh, they create expanding volume on the inlet side of the pump. Liquid flows into the cavity and is trapped by the lobes as they rotate. Liquid travels around the interior of the casing in the pockets between the lobes and the casing. The liquid does not pass between the lobes. The meshing of the lobes forces the liquid through the outlet port under pressure.

6.2.2.8 Operational Problems

The gases like Hydrogen Sulphide often get liberated when the sludge, particularly the digested sludge, is subjected to suction. This hampers the proper operation of the pump. The pumps should be installed, as far as possible, with positive suction. If the suction arrangements are improperly designed, a vortex-cone or sink developing in the sludge blanket will cause the watery sludge or supernatant to be drawn instead of the sludge.

The suction pipe should not be too long, nor should the pumping be too long or too fast. It is better to pump more often than at reduced speed. When a pump is equipped with variable speed drive, it can be started at a relatively high speed and the speed can then be reduced.

Sludge from two settling tanks should not be connected to the suction of a common pump. The settling tank with the thinner sludge will get pumped and the thickened sludge in the other tank will not get pumped. Similar problem will happen, if the suction lines from the two tanks will have differential frictional losses.

The tank with higher frictional loss in its suction piping, which may be because of more length or because of choking, will not get pumped. The capacity of sludge pumps is required to be regulated according to the sewage load. Further, variable speed drives are more appropriate for regulation because delivery valves present in the sludge pumping system makes the system inefficient and prone to trouble.

6.2.2.9 Requirement of Standby Units

The number of pumping units required including the standby is determined by several factors like the particular function involved, the size of the plant and the arrangement of the units, especially having combination of more than one function. A standby capacity of 16 hours in 2 shifts, 7 days working 100% standby is recommended wherever mechanical thickening and mechanical dewatering is practised. However, these standby units are not required for gravity thickening with picket fence. Since sludge pumping is an important function, standby pumps are provided in equal numbers or by such arrangement that permits dual duty. The scum is usually mixed with primary sludge and pumped.

6.2.2.10 Pump Appurtenances

The performance of the sludge pumps can be more efficient and their control can be better if various appurtenances such as air chambers, sampling devices, measuring devices, valves, gauges are incorporated in the system and facilities such as revolution counters, gland seals, time clocks, etc., are kept available at the plant.

6.2.2.10.1 Air Chamber

An air chamber of adequate size is necessary for all plunger type sludge pumps on the discharge side of the pump as well as the suction side of the pumps, particularly where positive suction head exists. Such chambers absorb the shock of plunger pump pulsations.

6.2.2.10.2 Revolution Counter

Plunger-type sludge pumps should be equipped with revolution counters or integrating recorders to help the operator to determine the quantity of sludge pumped in duplicate pump installations. These pumps aid in equalizing the service and wear of each pump.

6.2.2.10.3 Gland Seals

In the case of centrifugal pumps, external sealing is provided in the stuffing box to prevent the ingress of air into the pump. The external sealing may be grease seal or water seal. The water seals are preferable, as it helps the grit and dirt to be washed away. The water to the water-seal has to be potable water. However, the connection of potable water should not be taken directly from supply lines.

6.2.2.10.4 Valves

When a dry pit pump has positive suction head in the wet well, there should be an isolating valve. Usually a gate-valve or a knife edge valve on the suction line, is used to facilitate isolating the pump for maintenance. On the delivery side of centrifugal pumps, a non-return valve is necessary, so that the pump would not experience the back-pressure from the delivery head when the pump has to be switched off. To minimize the pressure-drop across the valve during the running of the pump, the non-return valve should be of the swing-check or the ball-check type.

To avoid water-hammer, which is likely to be caused by the closure of the valve, the valve may be provided with an anti-slam device, either of the lever and dead-weight type or of the spring-loading type or of the dash pot type. Dual check valves are sometimes used, which gives more consistent operation and facilitates for the use of the pump as metering device. All the valves may be provided with drain plugs. In larger size plants, where pumps may run in parallel operation with different permutation of the standbys, isolation valves are required to isolate those pumps that may be idle. All valves should preferably be of the rising stem type, since they offer the advantage of visual indication of the valve-position.

For exterior underground locations, gate valves are generally used. Underground sludge valves should be avoided as far as possible by taking advantage of the hydrostatic pressure for sludge withdrawal through a slant pipe and valve.

6.2.2.10.5 Gauges

Pressure gauges should be provided on both the suction and delivery sides. For pumps having suction lift, the gauge on the suction side should be a composite vacuum-pressure gauge. The gauges should be with a cast iron bowl and an oil-resistant rubber diaphragm to keep the sludge away from the finer working parts of the gauges.

6.2.2.10.6 Sampling Devices

All sludge pumps shall be provided with sampling taps either within themselves or in the piping adjacent to the pump. These are usually plug valves, normally of size NS 40. Plug valves are simple and easy to operate for taking samples.

6.2.2.10.7 Washouts and Drains

Washout or flushing arrangements are provided for sludge pumps to facilitate easy and rapid cleansing. The drains on the pump body should be of ample size to ensure release of pressure and drainage of the liquid. The outlet of the drain should be connected to an adjacent floor drain to keep the floor clean.

6.2.2.10.8 Time Clocks

Time clocks, wired across the magnetic starters or motor leads of sludge pumps can be a valuable help to the operators. They help to keep an accurate record of the time taken to run the pump for observing the preventive maintenance schedules in respect to attending to the lubrication, equalization of wear and tear, etc.

6.2.2.10.9 Measuring Devices

While time clocks and counters are adequate for small plants, supplementary flow-metering arrangements, such as flow tubes with flushing provisions are used in large plants for measuring and recording the quantities of sludge handling. Magnetic meters are more suitable for sludge metering. Sludge density meters to be installed in the return sludge line of plants for more than 1 mld capacity and an advisory for small plants so that they can install if they want in STPs of less than 1 mld capacity.

6.2.2.11 Pump Drive Equipment

The prime movers for the pumps are usually the electric motors, which have been discussed in detail in Section 5.12.3 of Chapter 5. It is desirable to use flame-proof motors. I.C. engines can be used for standby services in the case of failure of electric power. Again, the I.C. engine is better used as prime mover for a standby generator than as a prime mover for the pump, because the standby generator can then provide the power for lighting and ventilation facilities. Gas engines using sludge gas as fuel would help not only as a standby power supply facility, but also as an effective energy conservation in the operation of the plants.

6.2.3 Physical Features of Sludge Piping for Pumps

After selecting the type of pump, the next important thing is to design the suction and delivery pipelines. The design is based on the sludge pumping rate, the velocity in the pipeline, a layout with minimum bends and a material that is corrosion and abrasion resistant.

Sewage sludge flows like a thin plastic material and hence, the formulae for the flow of water are not applicable. The velocity of flow should be in the critical range above the upper limit of the laminar flow and below the lower limit of the turbulent flow, in order to avoid clogging and deposition of grease, so that the application of the hydraulic formulae for flow of water becomes permissible. In general, velocities between 1.5 and 2.5 m/s are to be considered.

The diameter of sludge pipes is important to permit cleaning. Where sludge is drawn intermittently as in primary clarifiers, it is advisable that it should be at least 150 mm for suction drawal and 200 mm for gravity drawal.

Provision shall be mandatorily provided for periodical flushing of the pipeline. This can be made by inserting a “Y” branch double flanged special and closing the free side of the flange by a knife edge valve and then another double flange short pipe with blank flange. For flushing the pipeline, first the sludge pump shall be stopped or if it is by gravity, the delivery side valve should be closed. Thereafter, the delivery side of the service line of the plant air compressor or a branch line from the final treated sewage pump shall be suitably connected to the free flange of the short pipe after removing the blank flange. Thereafter, the knife gate valve shall be slowly opened and the air or treated sewage gradually allowed to dislodge any choked sludge back into the clarifier.

For smaller STPs the single diaphragm sludge pump arrangement can be used to advantage. In order to take care of thin sludge to flow by gravity for short distances within the STP, a 3% or greater slope should be adopted.

The suction and discharge piping shall be arranged in such a way that their lengths are as short as possible, straight and with minimum bends. Adequate provision shall be made to facilitate cleaning. Large radius elbows and sweep tees are usually adopted for change in direction. High points should be avoided, as far as possible, to prevent gas pockets. Suitable recess and sleeves are usually provided for all pipes passing through masonry. Double-flanged pipes are usually adopted for sludge lines with at least one 45 degrees double flanged joint in the line for easy dismantling and reassembling. Valves shall be provided at selected locations to clean the lines.

6.2.4 Adverse Effects of Heavy Metals and Sludge Components on Unit Processes

Heavy metals may have adverse effects upon the sludge digestion which is a biological process. If concentrations of certain materials (e.g., ammonia, heavy metals, light metal cations and sulphide) increase significantly, they can create unstable conditions in the anaerobic digester. A shock load of such materials in the plant influent or a sudden change in digester operation (e.g., overfeeding solids or adding excessive chemicals) can create toxic conditions in the digester.

Typically, excess concentrations of such toxicants inhibit methane formation, which leads to volatile acid accumulation, pH depression and digester upset. Depending on the concentration and type of toxics, the effect can be acute (e.g., instant process failure) or chronic (e.g., depressed performance). Chemicals can control the concentrations of dissolved toxics (e.g., using iron salts to control sulphide). A sound monitoring and control programme, and an understanding of toxic agents, can greatly improve the design of mitigation systems.

6.2.5 Sludge Digestion or Stabilization Requirements including Appropriate Pathogen and Vector Attraction Reduction

Processes that significantly reduce pathogen levels in sludge include aerobic and anaerobic digestion, air drying, alkaline stabilization and composting. Processes that further reduce pathogens include Beta or Gamma ray irradiation, composting, heat drying, heat treatment, pasteurization and thermophilic anaerobic digestion.

The STPs typically use the following four processes to reduce pathogen level in sludge.

1) Heat drying
2) Aerobic and anaerobic digestion
3) Composting
4) Alkaline stabilization

The pathogens in sludge pose risk only if there are routes by which these come into contact with humans or animals. The route for transport of pathogens is transmission by vectors such as insects, rodents and birds. These are capable of transmitting a pathogen from one organism to another either mechanically (by simply transporting the pathogen) or biologically by playing a specific role in the life cycle of the pathogen. Suitable methods for measuring vector attraction directly are not available.

Vector attraction reduction is accomplished by employing one of the following:

i. Biological processes which breakdown volatile solids and thereby reducing the available food nutrients for microbial activities and odour potential

ii. Chemical or physical conditions that stop microbial activity

iii. Physical barriers between vectors and volatile solids in the sewage sludge

6.2.6 Return Flow Treatment Requirements

The thickened centrate, digested supernatant liquor, dewatered filtrate, etc., generated in each sludge treatment process are known by the general name “return flow” and this return flow is generally returned to the STP and treated. In this case, the water quality that requires to be checked in the return flow generated in each treatment stage is as follows:

1) Thickening: SS, nitrogen, phosphorous

2) Anaerobic digestion: Nitrogen, phosphorous, COD

3) Dewatering: although the items vary depending on the treatment process up to dewatering, the digestion process exits nitrogen and phosphorus

In a STP that treats only sludge generated from individual treatment plants, the return flow is generally assumed to have no adverse effect when designing the STP considering the return flow loads generated from the sludge treatment. However, when temporal changes in quantity or quality of the return flow are large, measures should be adopted such as installing a return flow storage tank, temporarily storing the return flow and returning the averaged return flow to the sewage treatment facilities. The return destination is taken as the grit chamber or the primary settling tank on the influent side, but in case of the former, considerations are necessary to sample influent sewage that does not include the return flow.

Sometimes, the return flow may be independently treated as a method of reducing the return flow load circulating between the STP and the sludge treatment facilities. When sludge is received from another treatment plant and anaerobic digestion is being performed, the BOD, SS, COD, nitrogen and phosphorus loads of the return flow will increase.

Therefore, independent return flow treatment or some other form of pre-treatment and returning to the STP may be considered. There are two methods for independently treating return flows so that the treated water quality is approximately the same as that of the influent and returning it to the secondary treatment facility; and the method of treating it as far as possible by direct discharge. However, the method should be decided after making an overall judgment considering economics including treatment cost and the stability of treated water.

6.2.7 Sludge Storage Requirements

Sludge storage tanks are installed when sludge is to be stored or transferred between various facilities when the sludge withdrawn intermittently from primary settling tank, secondary settling tank and gravity thickener is to be continuously loaded to the post treatment stages.

These tanks may sometimes be installed when sludge has to be stored for a comparatively long period such as when sludge dewatering equipment needs to be operated only during day time and sludge generated in a small-scale facility needs to be stored, or sludge has to be temporarily transported. A sludge storage tank should be decided as described below.

6.2.7.1 Capacity of Sludge Storage Tank

Tank capacity should be decided considering the amount of sludge loaded into and drawn off from the tank, the sludge transfer process (continuous, intermittent), and the storage time required for O&M.

6.2.7.2 Structure

Sludge storage tanks are generally located underground; therefore, reinforced concrete structure should be adopted as watertight structure with no permeation of groundwater.

Corrosive gases such as hydrogen sulphide may be emitted and hence, anti-corrosive coating should be applied on the internal surface of the tank to protect from corrosion.

6.2.7.3 Number of Tanks

A minimum of two tanks of each 50 percent capacity may be provided.

6.2.7.4 Agitator

When sludge is stored for a long period, it will decompose in accordance with Figure 5-2 in chapter 5. In this process, scum will form and sediments will form at the bottom. Hence, air should be blown as necessary or an agitator or scum skimmer should be installed.

6.2.7.5 Odours in Tank and Deodorization

In principle, odours should be captured and deodorized to prevent leakages.

When air agitation is used, the suction capacity should be greater than the blowing capacity.

6.2.8 Methods of Ultimate Disposal

The methods of ultimate disposal are as follows:

1. Dewatered sludge reused as it is in agricultural land or in landfills.
2. Agricultural land applications have the advantages of simplicity and low costs, but there are safety issues such as bacteria; therefore, introduction of a digestion stage and stabilization of quality are recommended. There are also issues related to generation of odours and transportation issues in this treatment process; thus, considerations for the surrounding environment are necessary. Special measures are necessary when sun-dried beds are adopted.
3. Method in which dewatered sludge is processed and used in agricultural land as fertilizer; sludge is dried in granular form by mechanical drying or solar drying or by composting to improve safety and handling ability. Hence, this form is recommended for agricultural land applications.
4. Dewatered sludge is incinerated or fused, and ash or slag is effectively used as building material, etc., or used as landfills.
5. Thermal or solar dried sludge can be used as low-grade fuel with the concurrence of PCB

6.2.9 Back-up Techniques of Sludge Handling and Disposal

It is necessary to prepare the back-up techniques of sludge handling and disposal for the case where sludge treatment and disposal cannot be performed due to failure of sludge treatment facilities. The following back-up techniques may be considered.

1. Storage of liquid sludge in vacant or unoccupied settling tanks, etc. and storage of solid sludge in open space at STPs if available.
2. Transportation of liquid sludge using a tanker lorry to other STPs.
3. Preparation of the mobile dewatering machine.

6.3 SLUDGE THICKENING

This is to thicken the concentration of sludge solids generated in the clarifier to make sludge digestion and sludge dewatering more effective. Sludge to be thickened may be primary sludge or combined sludge from primary and excess sludge. Thickening may be broadly classified into three types namely, gravity, centrifugal and floatation. The floatation can further be dissolved-air floatation or dispersed-air floatation. When the thickening of sludge is inadequate, the filtrate from dewatering will have large amounts of suspended solids returning to the STP and affect the water quality. Hence, excess sludge is increasingly being mechanically thickened using centrifugal thickening machines or floatation thickeners. Moreover, when performing sludge treatment for sludge collected from various STPs, sludge with varying properties is likely to be treated; therefore, forced sludge thickening process such as by using mechanical thickening equipment is indispensable.

Degritting and debris removal equipment preferably be installed as the pre-treatment process before thickening unless the STP itself has such facilities in the raw sewage stage.

6.3.1 Gravity Thickening

Gravity thickening is the most common practice for concentrating the sludge. It is adopted for primary sludge or combined primary and activated sludge, but is not successful in dealing with excess sludge independently. Gravity thickening of combined sludge is not effective when excess activated sludge exceeds 40% of the total sludge weight. In such cases, other methods of thickening of the excess activated sludge have to be considered. Gravity thickeners are either continuous flow or fill and draw type, with or without addition of chemicals. Use of slowly revolving stirrers improves the efficiency. Continuous flow tanks are deep circular tanks with central feed and overflow at the periphery. They are designed for a hydraulic loading of 20,000 to 25,000 lpd/m^2. Loading rates less than 12.000 lpd/m^2 are likely to give too much solids to permit this loading hence, it is necessary to dilute the sludge with plant effluent and it is referred to as dilution water. Better efficiencies can be obtained for gassy sludge by slow revolving stirrers.

The surface loadings for various types of sludge are given in Table 6.6 along with solid concentration of various types of thickened sludge.

Table 6.6 Surface loadings and solids concentration

Type of Sludge	Solids Surface Loading (kg/day/sqm)	Thickened Sludge Solids Concentration (%)
Separate Sludge		
Primary	90 - 140	5 – 10
Activated	25 - 30	2.5 - 3.0
Trickling filter	40 - 45	7 – 9
Combined Sludge		
Primary + activated	30 - 50	5 – 8
Primary + trickling filter	50 - 60	7 – 9

Continuous thickeners are mostly circular with a side water depth of about 3 m. Concentration of the underflow solids is governed by the depth of sludge blanket up to 1m beyond which, there is very little influence of the blanket. If underflow solids concentration is increased with increased sludge detention time, 24 hours is required to achieve maximum compaction. Sludge blanket depths may vary with fluctuation in solids production to achieve good compaction. During peak conditions, lesser detention times will have to be adopted to keep the sludge blanket depth sufficiently below the overflow weirs to prevent excessive solids carryover. It is necessary to ensure provisions for (a) regulating the quantity of dilution water needed; (b) adequate sludge pumping capacity to maintain any desired solids concentration, continuous feed and underflow pumping; (c) protection against torque overload and (d) sludge blanket detection.

6.3.1.1 Capacity

The tank capacity is decided considering the following:

1) Consider solids load as 60 to 90 kg.dry solids / (m^2.d) approximately.
2) Consider effective water depth as approximately 4m.

6.3.1.2 Shape and Number

The shape and number of tanks are decided considering the following:

1. In principle, the tank should be of circular shape.
2. Consider the slope of the tank bottom as follows:
    i. A slope as 5/100 or greater if a sludge scraper is installed.
    ii. If no sludge scraper is installed, assume hopper system and take a slope of 60 degrees or greater with respect to the horizontal.
3. In principle, the number of tanks should be two or more tanks.

6.3.1.3 Structure

The structure of the tank is decided after considering the following:

1. In principle, the tank should be of RCC with consideration given to anti-corrosion.
2. Provide sludge inlet pipes, sludge draw-off pipes and overflow weirs.

6.3.1.4 Appurtenances

Decide the appurtenances after considering the following:

1. The speed of the sludge scraper should not agitate the deposited sludge
2. In principle, draw off the sludge using a pump.
3. Use ductile or cast iron pipe as far as possible.
4. Provide backwash pipes at appropriate locations considering that the sludge draw-off pipe may be blocked.
5. If multiple tanks are present, install the distribution tank at the front end.
6. Install a scum skimmer on the liquid surface of tank. Take steps to ensure that the overflow weir can be cleaned.
7. If necessary, install de-gritting and debris removal equipment before thickening.
8. If necessary, cover the tank and install ventilation and deodorization equipment as dealt with in Chapter 5.

6.3.2 Centrifugal Thickening

Thickening by centrifugation is applied only when there is space limitation or sludge characteristics will not permit the adoption of the other two methods. This method involves high maintenance and power costs. Centrifuges employed are of either disc or solid bowl type. Disc centrifuges are prone to clogging while the latter gives a lower quality of effluent.

6.3.2.1 Centrifugal Thickener

1. In principle, two or more thickeners should be installed.
2. Take the water content of thickened sludge as 96% approximately, and the standard solids recovery rate as 85% to 95%.
3. Use durable material.

6.3.2.2 Sludge Feed Pump

Decide the sludge feed pump after considering the following:

1. Select a pump with adequate capacity.
2. Install separate pumps for each centrifugal thickener.

6.3.2.3 Appurtenances

Decide the appurtenances after considering the following:

1. If necessary, install de-gritting and debris removal equipment before thickening.
2. Install sludge feed tank.
3. Install thickened sludge storage tank.
4. Install water supply system for internal cleaning of the centrifugal thickener and for cooling the bearing.
5. Formulate measures against vibration and noise. Install ventilating equipment and deodorization equipment, if necessary.
6. Install equipment for controlling the water content of thickened sludge.
7. If necessary, install chemical dosing equipment.

6.3.3 Air Floatation Thickening

Air floatation units employ floatation of sludge by air under pressure or vacuum and are normally used for thickening the waste activated sludge. These units involve additional equipment, higher operating costs, higher power requirements, and more skilled maintenance and operation. However, the removal of oil and grease, solids, grit and other material as also odour control are distinct advantages.

In the pressure type floatation units, a portion of the subnatant is pressurized from 3 to 5 kg/cm^2 and then saturated with air in a pressurization chamber. The effluent from this is mixed with influent sludge immediately before it is released into the flotation tank. Excess dissolved air then rises up in the form of bubbles at atmospheric pressure attaching themselves to particles which form the sludge blanket. Thickened blanket is skimmed off while the un-recycled subnatant is returned to the plant.

The vacuum type employs the addition of air to saturation and applying vacuum to the unit to release the air bubbles which float the solids to the surface.

The efficiency of air floatation units is increased by the addition of chemicals like alum and polyelectrolytes. The addition of polyelectrolytes does not increase the solids concentration, but improves the solids recovery rate from 90% to 98%.

6.3.3.1 Dissolved-air Floatation Thickening

Dissolved-air floatation thickening refers to the process of making fine air bubbles stick to sludge particles, to reduce the apparent specific gravity of sludge with respect to water, and make the particles buoyant so as to separate solids and liquids. Systems include partial-flow pressurization, full-flow pressurization and return flow pressurization.

It is important that the appropriate size of fine air bubbles is generated and these attach effectively to sludge particles. The attachment of bubbles may be easy or difficult depending on the physical and chemical characteristics of the surface of particle; sometimes, addition of coagulant may be necessary depending on the particle. Dissolved-air floatation thickening equipment consists of dissolved-air floatation tank, pressurization pump and air dissolution tank.

Partial-flow pressurization is not used much for sludge treatment since the air dissolution level is low compared to other methods because a part of the loaded sludge is directly conveyed to the air dissolution tank by the pressurization pump.

Full-flow pressurization system is simpler than the return flow pressurization system, since the complete volume of loaded sludge is sent to the air dissolution tank by the pressurization pump. However, since there is a limit to air solubility, it is mostly used when the concentration of loaded sludge is comparatively small.

Return flow pressurization is one in which loaded sludge and pressurized water are mixed by an ejector. The power of the sludge pump can be reduced, but its suitability depends on the properties of sludge.

In recent years, the system of mixing and pressurizing loaded sludge in pressurized sewage (mixing under pressure) and the system of mixing loaded sludge immediately after reducing the pressure of pressurized sewage (mixing under reduced pressure) are being used. The return flow pressurization system uses centrate or treated water.

6.3.3.1.1 Capacity

The floatation tank capacity is decided considering the following:

1. Consider the solids load as 100 to 120 kg dry solids / (m^2.d) and the standard solids recovery rate as 85% to 95%.

2. Consider the standard effective depth of tank as 4.0 m to 5.0 m.

6.3.3.1.2 Shape and Number

The shape and number of floatation tanks are determined considering the following:

1. The shape of the tank is to be circular, square or rectangular.
2. In principle, the number of tanks should be two or more tanks.

6.3.3.1.3 Structure

The structure of floatation tank is decided considering the following:

1. The material should be watertight reinforced concrete or equivalent.
2. To adjust the water surface, install weir and other equipments.

6.3.3.1.4 Sludge Remover and Sludge Scraper

A froth remover that removes the froth and a sludge scraper that scrapes and collects the settled sludge are installed in the floatation tank.

6.3.3.1.5 Pressurizing Pump

Decide the pressurizing pump after considering the following:

1. Select discharge pressure in the range of 0.2 to 0.4 MPa (2 to 4 kgf/cm^2).
2. The capacity is decided as follows:

i) In case of the partial-flow pressurization, consider the concentration of sludge and pressure so that the desired air-solid ratio is obtained.

ii) In case of the full-flow pressurization, take the loaded sludge amount.

iii) In case of the return flow pressurization, take the loaded sludge amount and decide the capacity after considering the concentration of sludge and pressure.

6.3.3.1.6 Air Dissolution Tank

The structure of air dissolution tank is decided considering the following:

1. The structure should comply with pressure vessel construction standards, and should be capable of giving good air dissolution efficiency.
2. The capacity should be decided based on a retention time of about 2 minutes.
3. Dispersion equipment, pressure gauge, safety valve, manhole for internal inspection, etc., should be provided.

6.3.3.1.7 Appurtenances

Decide the appurtenances after considering the following:

1. Install a sludge storage tank.
2. Install floatation sludge degassing tank.
3. Install thickened sludge storage tank.
4. Install sludge pump.
5. Install return flow tank (in case of return flow pressurization system).
6. If necessary, cover it and install ventilation and deodorization equipment.
7. If necessary, install chemical dosing equipment.

6.3.3.2 Dispersed-air Floatation Thickening

In this process, air bubbles generated by adding a foaming agent are attached to solids in the sludge, mixing equipment by adding a polymer coagulant and sludge is floated. The main equipment consists of floatation equipment, foaming equipment, mixing equipment, water level adjusting equipment and auxiliary equipment such as sludge pump and floatation sludge de-aeration tank.

Foaming agent and air mixed in water are agitated mechanically in foaming equipment and fine bubbles are generated under atmospheric pressure.

These bubbles are made to attach to the solids to which polymer coagulant is added and solids of bubbles and floc with strong bonding strength are formed.

The solids made of floc and bubbles are transferred to the floatation equipment and are floated, separated and removed, and the fine bubbles in sludge are mechanically agitated and removed. The centrate is drained out and it overflows from the water level adjusting equipment.

6.3.3.2.1 Capacity

The floatation tank capacity is determined considering the following:

1. The solids load is 25 kg dry solids/ (m^2h) approximately, and the standard solids recovery rate is 95% and above.
2. Consider the effective water depth as approximately 4 m.

6.3.3.2.2 Shape and Number

The shape and number of floatation tanks are decided considering the following:

1. The standard shape of the tank is circular.
2. In principle, the number of tanks should be two or more tanks.

6.3.3.2.3 Structure

The structure of floatation tank is determined considering the following:

1. The standard material is durable and corrosion resistant steel.
2. To adjust the water surface, install weir and other equipments.

6.3.3.2.4 Sludge Remover and Sludge Scraper

A sludge remover that removes the froth and a sludge collector that scrapes and collects the settled sludge are installed in the floatation tank.

6.3.3.2.5 Control of Foaming

Sometimes sludge digesters are found to be foaming inside the digester. Technically this does not interfere with the treatment process and as long as it does not escape from the digesters into the atmosphere, this can be ignored. It tends to get controlled by itself. If the problem persists and foam is found in the sludge gas, commercially available anti-foaming chemicals can be briefly used as per the guidelines of respective manufacturers with proven track record in such applications.

6.3.4 Belt Type Thickening

When coagulant is loaded, the belt type thickener performs gravity filtering and thickening on a travelling belt, which may be a stainless steel belt or a plastic belt. While being transported to the discharge side, the sludge is filtered by gravity and thickened; it is separated by a scraper at the concentrated sludge discharge unit. The belt is subsequently washed with filtrate. Chemical conditioning equipment is used to mix sludge and polymer coagulant. In addition to the stand alone equipment, line mixing type equipment are also available.

6.3.4.1 Capacity

The capacity of belt filter press thickener is decided after considering the following:

1. The sludge volume is approximately 10 m^3/hour to 100m^3/hour.
2. The standard thickening performance is as given below.
   i) Thickened concentration: 4-5% approx
   ii) Chemical addition rate: 0.3% approximately (percent solid)
   iii) SS recovery rate: 95% or greater

6.3.4.2 Shape and Number

The shape and number are decided after considering the following:

1. The standard shape is rectangular with the longer side being horizontal
2. In principle, the number should be two or more units

6.3.4.3 Chemical Dosing Equipment

Decide the chemical dosing equipment after considering the following:

1. Install coagulant dissolution tank
2. Install coagulant feeder
3. Install coagulant dosing pump

6.3.4.4 Appurtenances

Decide the appurtenances after considering the following:

1. Install sludge feed tank
2. Install thickened sludge storage tank
3. Install sludge pump
4. If necessary, cover it and install ventilation and deodorization equipment

6.3.5 Comparison of Different Types of Sludge Thickening

A comparison of different sludge thickening processes is presented in Table 6-7 overleaf.

6.4 ANAEROBIC SLUDGE DIGESTION

6.4.1 General

This is the biological degradation of organic matter in the absence of oxygen. In this process, much of the organic matter is converted to methane, carbon-dioxide and water and therefore, it is a net energy producer. Since, little carbon and energy alone is available to sustain further biological activity, the remaining solids are rendered stable.

6.4.1.1 Microbiology of the Process

Anaerobic digestion involves several successive biochemical reactions earned by a mixed culture of microorganisms. There are three degradation stages namely, hydrolysis, acid formation and methane formation. The reactions involved in anaerobic digestion are shown in Figure 6.4 overleaf.

In the first stage of digestion, complex organic matter like proteins, cellulose, lipids are converted by extra cellular enzymes into simple soluble organic matter. In the second stage, soluble organic matter is converted by acetogenic bacteria into acetic acid, hydrogen, carbon dioxide and other low molecular weight organic acids. In the third stage, two groups of strictly anaerobic methanogenic bacteria, are active. While one group converts acetate into methane and bicarbonate, the other group converts hydrogen and carbon-dioxide into methane. For satisfactory performance of an anaerobic digester, the second and third stages of degradation should be in dynamic equilibrium, that is, the volatile organic acids should be converted into methane at the same rate as they are produced.

Table 6.7 Comparative Evaluation of Different Sludge Thickening Processes

Evaluation Criteria	Gravity	Dissolved Air Flotation	Centrifugation	Gravity Belt	Rotating Drum
Space requirement	High	Medium	Low	Medium	Medium
Operation and maintenance	Simple	Medium	High	Medium	Medium
Typical Application	Primary and Combined	Waste activated sludge	Waste activated sludge	Waste activated sludge	Waste activated sludge
Conditioning Chemicals	None	High	High	Medium	Medium
Power requirement	Low	High	High	Medium	Medium
Capital Cost	Low	High	High	Medium	Medium
Operation Cost	Low	High	High	Medium	Medium
Thickened Sludge solids concentration	Medium	Low	High	Medium to high	Medium to high
Building corrosion if enclosed	High	Medium	None	Medium	Medium
Odour problem	Serious	Moderate	Low	Moderate	Moderate

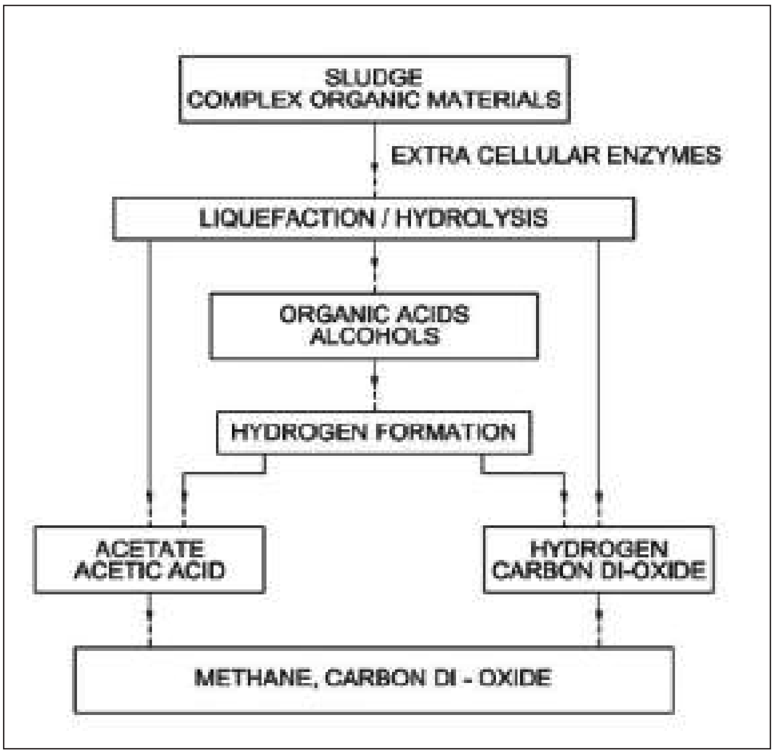


Figure 6.4 Anaerobic digestion mechanisms

However, methanogenic microorganisms are inherently slow growing compared with the volatile acid formers and they are adversely affected by fluctuations in pH, concentration of substrates and temperature. Hence, the anaerobic process is essentially controlled by the methanogenic microorganisms.

6.4.2 Digestion Types

Two different types in anaerobic sludge digestion processes are namely, low rate and high rate and are used in practice. The basic features are in Figure 6.5

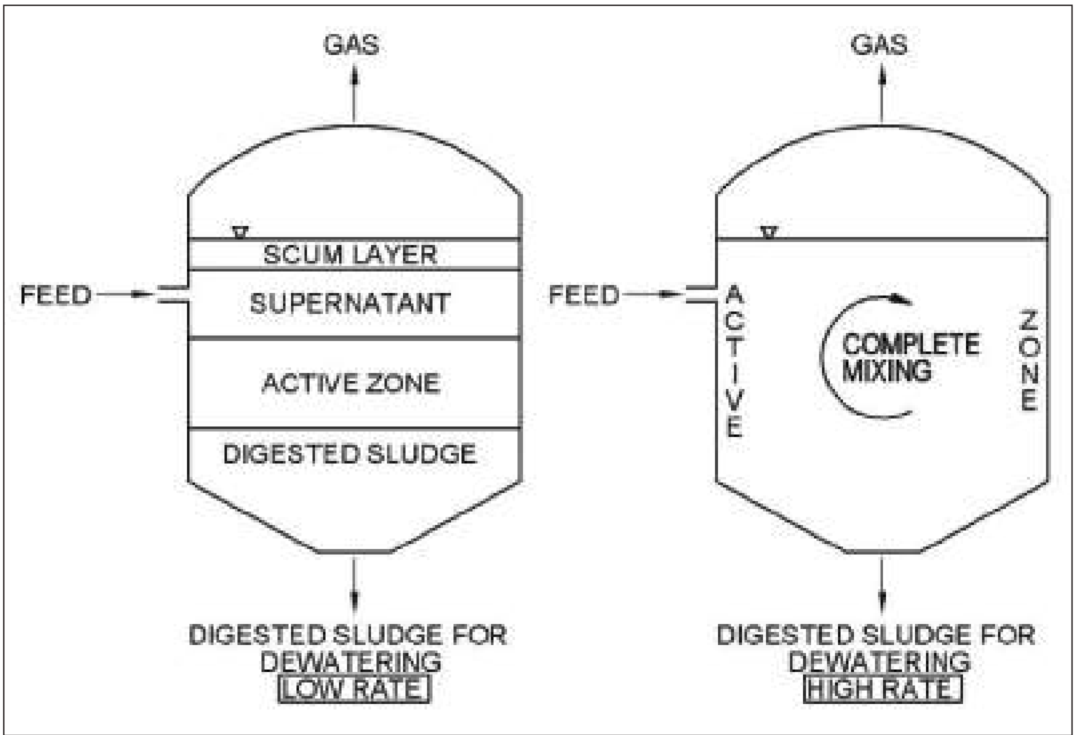


Figure 6.5 Sludge digestion system

6.4.2.1 Low Rate Digestion

Raw sludge is fed into the digester intermittently. Bubbles of sewage gas are generated and their rise to the surface provides some mixing. In the case of few old digesters, screw pumps have been installed to provide additional intermittent mixing of the contents, say once in 8 hours for about an hour. As a result, the digester contents are allowed to stratify, thereby, forming four distinct layers: a floating layer of scum, layer of supernatant, layer of actively digesting sludge and a bottom layer of digested sludge; essentially the decomposition is restricted to the middle and bottom layers. Stabilized sludge that accumulates and thickens at the bottom of the tank is periodically drawn off from the centre of the floor. Supernatant is removed from the side of the digester and returned to the treatment plant.

6.4.2.2 High Rate Digestion

The essential elements of high rate digestion are complete mixing and more or less uniform feeding of raw sludge.

Pre-thickening of raw sludge and heating of the digester contents are optional features of a high rate digestion system. All these four features provide the best environmental conditions for the biological process and the net results are reduced digester volume requirement and increased process stability.

Complete mixing of sludge in high rate digesters creates a homogeneous environment throughout the digester. It also quickly brings the raw sludge into contact with microorganisms and evenly distributes toxic substances, if any, present in the raw sludge.

Furthermore, when stratification is prevented because of mixing, the entire digester is available for active decomposition, thereby increasing the effective solids retention time.

Pre-thickening of raw sludge before digestion results in the following benefits:

1. Large reduction in digester volume requirements
2. The thickener supernatant is of far better quality than digester supernatant; thereby, it has less adverse impact when returned to the STP
3. Less heating energy requirements
4. Less mixing energy requirements

There is however, a point beyond which further thickening of raw sludge has the following effects on digestion:

a) Solid concentration higher than 6% in the digester affects the viscosity, which in turn affects mixing, hence deserves special consideration.

b) In case of highly thickened raw sludge, the concentration of salts and heavy metals present in the raw sludge and end products of digestion, such as volatile acids and ammonium salts, may exceed the toxic levels.

6.4.2.3 Sludge Temperature

Sludge temperature is one of the important environmental factors. Where the digester sludge temperatures are low, digester heating is beneficial because of the rate of microbial growth and therefore, the rate of digestion increases with temperature.

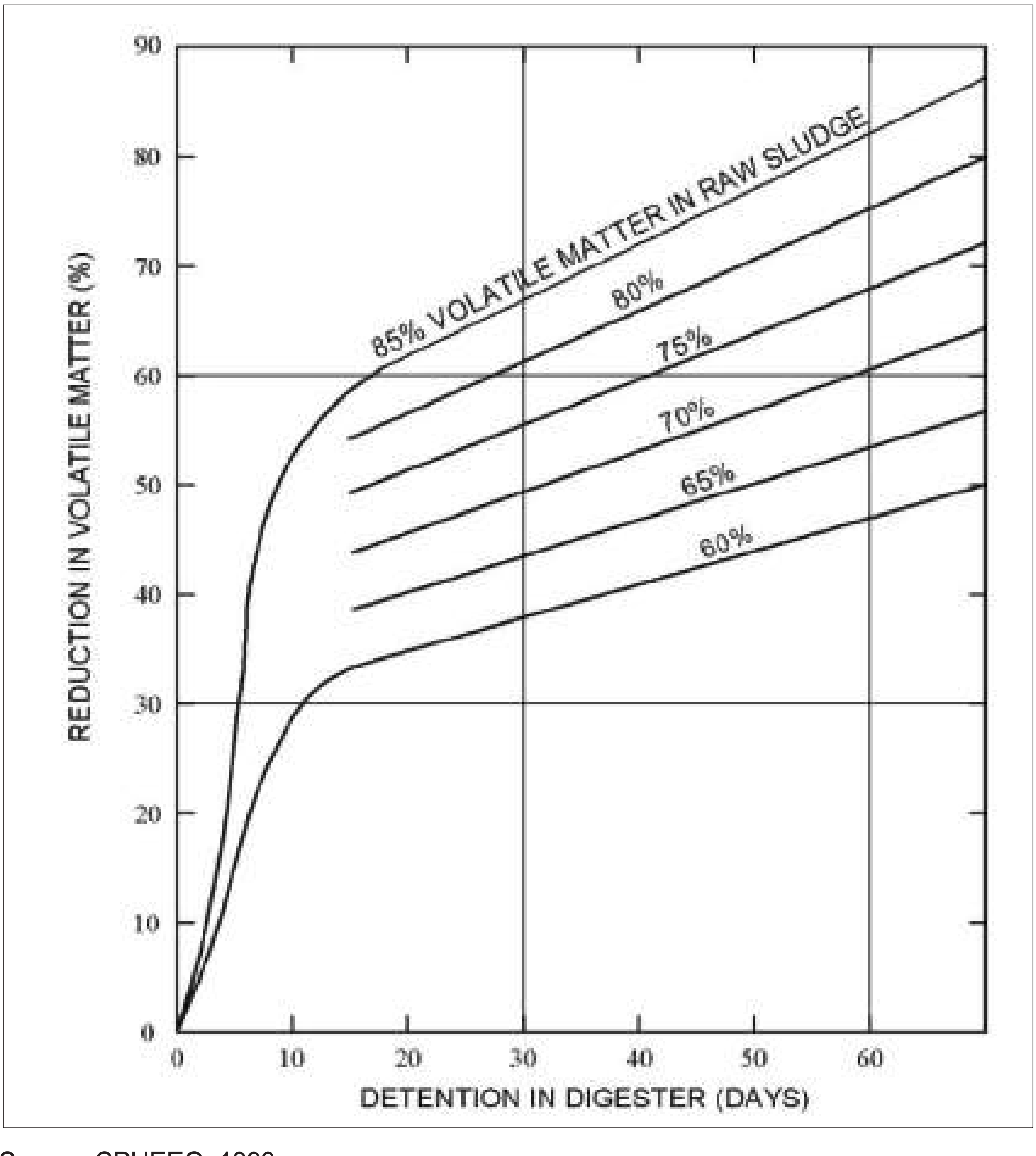
Depending upon the temperature, different kinds of microorganisms are active in the digester. For an operating temperature of 20° to 40°C, the range is known as mesophilic and for 40° to 60°C, the range is known as thermophilic.

The ambient temperature in our country is generally favourable for operation under mesophilic condition throughout the year. However, in special conditions such as hilly regions, where extremely low temperatures are likely to be encountered, it may be necessary to heat the digesters during specific periods of the year.

6.4.3 Digester Tank Volume

The determination of digester tank volume is a critical step in the design of anaerobic system. The digester volume must be sufficient to prevent the process from failing under all accepted conditions. Process failure is defined as accumulation of volatile acids that results in decrease in pH, when volatile acids/alkalinity ratio becomes greater than 0.5 and methane production stops. Once the digester turns sour, it usually takes several days to return to normal operation after the corrective actions are taken.

The digester capacity must also be large enough to ensure that raw sludge is adequately stabilized as discussed below in the section on solids retention time. The relationship between percentage volatile matter in the raw sludge, its reduction and detention time is shown in Figure 6.6.



Source: CPHEEO, 1993

Figure 6.6 Reduction of volatile matter as related to digester detention time

6.4.4 Sludge Loading Rate

6.4.4.1 Loading Criteria

Traditionally, volume requirements for anaerobic digestion have been determined from empirical loading criteria. Volatile solids loading rate (kg VSS/day/m^3) criteria have been commonly used to size anaerobic digesters. Table 6.8 lists the typical loading rates used for design purpose. However, it is now recognized that process performance is better correlated to solids retention time (SRT), as shown in the table and is discussed subsequently.

Table 6.8 Typical design criteria for sizing mesophilic anaerobic sludge digesters

Parameters	Low Rate Digestion	High Rate Digestion
Volatile Solids Loading Rate, kgVSS/day/m^3	0.6 - 1.6	1.6 - 6.4
Solids Retention Time, days	(*)	10 - 20
Hydraulic Retention Time, days	30 - 40	10 - 20

Note: (*) Computation of actual SRT is difficult as it depends on the capacity utilization.

6.4.4.2 Solids Retention Time and Temperature

The most important consideration in sizing anaerobic digester is that the microorganisms must be given sufficient time to reproduce so that they can (a) replace the cells lost with the withdrawn sludge and (b) adjust the microbial mass to the organic loading and its fluctuation.

The key design parameter for anaerobic biological treatment is the biological solids retention time (SRT), which is the average time a unit of microbial mass is retained in the anaerobic digester without recycling. The SRT is equivalent to the hydraulic retention time, that is, volume of digester/volume of sludge withdrawn per day.

Experiments have proved that percentage of destruction of volatile solids and formation of methane decreases as the SRT is reduced. The SRT can be lowered to a critical point (SRTc) beyond which the process will fail completely.

The temperature has an important effect on bacterial growth rates and accordingly changes the relationship between SRT and digester performance. The effect of temperature on volatile solids destruction is presented in Figure 6.7 overleaf.

The inset in Figure 6.7 shows that at SRT values greater than 30 days, fluctuations in temperature do not affect the digester stability, that is, no significant change in percentage volatile solids reduction.The size of anaerobic digester should be adequate enough to ensure that the solids retention time in the system is always well above the SRTc. Typical solids retention time design criteria followed for high rate digestion design are given in Table 6.9 overleaf.

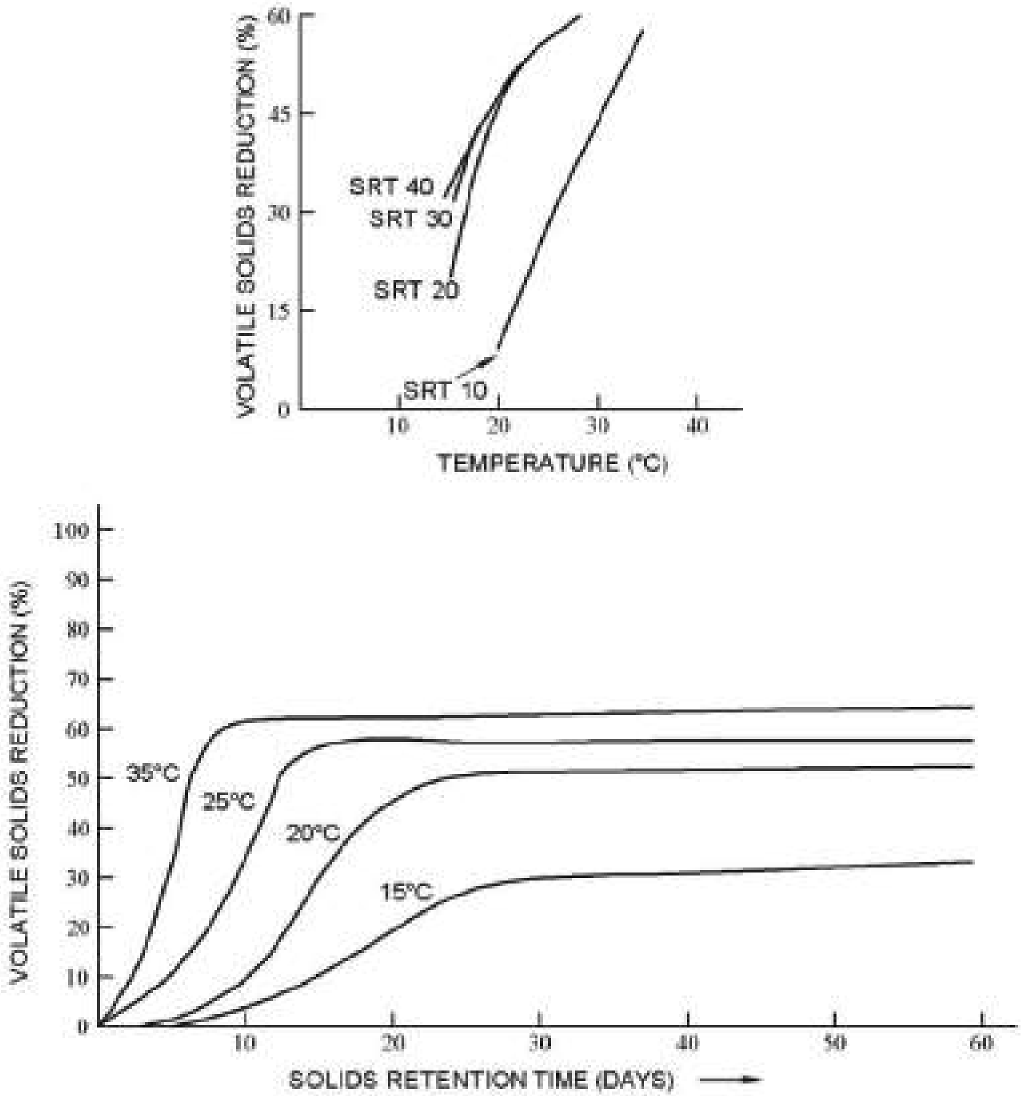


Figure 6.7 Effect of solids retention time and temperature on volatile solids reduction in a laboratory scale completely mixed aerobic digester

Table 6.9 Solids retention time at different temperatures (high rate digestion)

Operating Temperature, °C	Solids Retention Time, days		
	SRTc	Suggested for Design (SRTd)	Ratio
18	11	28	28/11 = 2.54
24	8	20	20/8 = 2.5
30	6	14	14/6 = 2.3
35	4	10	10/4 = 2.5
40	4	10	10/4 = 2.5

The SRT design criteria must be met under all anticipated conditions including:

1. Maximum grit and scum accumulations: Considerable amount of grit and scum may accumulate before a digester is cleaned. This reduces the active volume of the tank. Hence, about 0.6m to 1.0m additional depth for grit and scum accumulation must be provided.
2. Free Board: About 0.6 to 0.8m free board (from rim of the digester wall to the highest liquid level) must be allowed for differences in the rate of feeding and withdrawing and to provide reasonable operational flexibility.

6.4.4.3 Storage for Digested Sludge

Storage capacity for digested sludge is required in places where digested sludge is applied to drying beds for dewatering, and use of sludge drying beds is interrupted during monsoon periods.

This additional capacity requirement can be met either by increasing the digester capacity or by providing a separate digested sludge holding tank. Normally, an additional 10-15 days digested sludge storage capacity can be sufficient. However, if local meteorological data is available, such data should be used to determine the capacity of storage.

6.4.5 Sizing

6.4.5.1 Sizing of Low Rate Digesters

Lack of proper mixing in the conventional digesters leads to stratification, and gives rise to distinct layers of scum, supernatant, actively digesting sludge and digested sludge. The supernatant is withdrawn periodically and returned to the influent of the treatment plant, while the sludge is added at mid depth and withdrawn from the bottom. Since the supernatant is removed during digestion resulting in a decrease in digested sludge volume, the capacity of the digester is given by the expression:

$$V=[V_f - 2/3(V_f - V_d)]T_1 \qquad (6.1)$$

Where,

V : Volume of digester, m^3

V_f : Volume of fresh sludge m^3 added per day

V_d : Volume of digested sludge m^3 withdrawn per day

T_1 : HRT, days

Sometimes sludge drying beds alone are used and left in the open under the sky. In such cases, the rainfall in monsoon times results in many environmental problems. Hence, sludge storage facilities for the monsoon season are to be provided. The volume is given by the expression:

$$\text{Additional monsoon storage volume} = V_d\, T_2 \qquad (6.2)$$

Where, T_2 is the storage in days, during monsoon

The digester can be a single unit or two units – the primary and the secondary, the former being provided with the needed time for digestion and the latter to meet the requirements of monsoon storage. As discussed in the above Subsection 6.4.3, further additional capacity to compensate for grit accumulation and free board should be provided.

6.4.5.2 Sizing of High Rate Digester

Due to good mixing, there is no stratification hence, no loss of capacity due to scum or supernatant layers. By adopting more or less continuous addition of raw sludge and resorting to pre-thickening of the raw sludge to a solid content of about 6%, the digester volume can be designed for 10-15 days retention time.

When the digested sludge is to be dewatered on sludge drying bed, a second stage digester is normally provided where separation of supernatant and reduction in volume of sludge due to gravity thickening take place and digestion is completed. Additional storage capacity needed for the monsoon period can also be provided in the second stand digester. Capacities for high rate digestion may be determined by:

$$V' = V_f\, T_h \qquad (6.3)$$

$$V'' = [V_f - 2/3(V_f - V_d)]T - V_d\, T_2 \qquad (6.4)$$

Where,

V' : Volume of first stage digester, m^3

V'' : Volume of second stage digester, m^3

V_f : Volume of fresh sludge m^3 added per day

V_d : Volume of digested sludge m^3 withdrawn per day

T_h : Detention time in the high rate digester, days

T : Detention time in the second stage digester which is of the order of 10 days and

T_2 : Storage in days, during monsoon

As discussed in the above subsection 6.4.3, while computing the digester volume, additional volume to compensate for grit accumulation and free board should be provided.

The mass balance calculation in the 1993 edition in word has since been converted to M S Excel format and is presented as Appendix A.6-1. An example of digester sizing is presented in Appendix A.6.4.

6.4.6 Structure

6.4.6.1 Number of Units

Conventional digesters are designed as single units for plants treating up to 4 MLD. For larger plants, units are provided in multiples of two, the individual capacity not exceeding 3 MLD. High rate digesters are designed comprising primary and secondary digestion tanks, each unit generally capable of handling sludge from treatment plants up to 20 MLD.

6.4.6.2 Digester Shape and Size

The most common digester shape is a low, vertical cylinder with diameter ranging from 6 to 38 m and with height ranging from 6 to12 m. Digester mixing is effective when the ratio of digester diameter to sludge depth is between 1.5 and 4. Computational Fluid Dynamics (CFD) to decide on structural design is used. Figure 6.8 shows typical Low height cylindrical, Egg shaped and tall form cylindrical digesters.

		
Low Height Cylindrical Source: Perungudi STP-Chennai Metropolitan Water Supply & Sewerage Board	Egg Shaped Location: Kanazawa STP, Yokohama City, Japan, Source: Kajima Corporation	Tall form Cylindrical Source: Article on Biomethanation by A K Dhussa and V K Jain, Newsletter, National Bioenergy Board, MNRE, March 2002.

Figure 6.8 Typical low height cylindrical, egg shaped and tall form cylindrical digesters

6.4.6.3 Free Board and Depth

The free board is dependent upon the type of cover and the maximum gas pressure. For fixed dome or conical roofs, free board between the liquid level and the rim of the digester wall should not be less than 0.6m. For flat covers, the free board between water level and the top of the tank wall should preferably be not below 0.6 m. For fixed slab roofs, a free board of 0.8m is recommended. Sludge depth in a digester has to be carefully worked out. Too deep a digester causes excessive foaming, which may result in choking of the gas pipes and building up high pressures in the digester. In case of conventional low rate digester, when gas production reaches, a figure of about 9 m^3/day/ m^2 of top surface of sludge foaming becomes noticeable. Therefore, before the tank depth and surface area of a digester are worked out, maximum gas production rate should be determined. An average of about 0.9 m^3 of gas is produced per kg of volatile solids destroyed. The optimum diameter of depth of digester is calculated such that at the average rate of daily gas production, the value of 9 m^3 per m^2 of tank area is not exceeded.

6.4.6.4 Floor Slope

The floor slope should be in the range of 1 in 6 to 1 in 10 to facilitate easy withdrawal of sludge The digester floor should be designed for uplift pressure due to the subsoil water or suitably protected by anchoring.

6.4.6.5 Roofing

Sludge digesters can have either fixed or floating roofs. Reinforced concrete domes, conical or flat slabs are used for fixed roof and steel domes are used for floating cover. Steel floating covers may either rest on the liquid or act as gas holders in the digesters themselves. If a floating cover is used for gas holder in a digestion tank, an effective vertical travel of 1.2 to 2m should be provided.

6.4.6.6 Digester Control Room

Normally a control room is provided near the digesters to house the piping and the process control equipment, which are principally the sludge heating units (if used), sludge transfer and recirculation pumps, sludge sampling sinks, thermometers, blowers for ventilation and electrical control equipment. Where heating of sludge digesters is practiced, the operation could be managed by locating conveniently the necessary valves for supernatant and sludge withdrawal in the digester wall itself. However, in sewage treatment plants having more than four digesters, it is advisable to have a separate operation control room to house the necessary control equipment for convenient operation.

6.4.7 Mixing System

A certain amount of natural mixing occurs in anaerobic digester caused by both the rise of sludge gas bubbles and the thermal convection currents created by the addition of fresh or heated sludge. This effect of natural mixing is significant, particularly in case of high rate digesters fed continuously. However, this natural mixing is not sufficient to ensure stable performance of the digestion process. Therefore, methods used for mixing include external pumped circulation, internal mechanical mixing and internal gas mixing.

External pumped circulation while relatively simple is limited in application because of large flow rates involved. However, this method can achieve substantial mixing, provided sufficient energy in the range of 5 to 8 watts/m^3 is dissipated in the digester. More energy will be required if piping losses are significant. Pumped circulation allows external heat exchanges to be used for heating the digester contents and uniform blending of raw sludge with heated circulating sludge prior to the raw sludge entry to the digester.

Internal mechanical mixing by means of propellers, flat-bladed turbines, or draft tube mixers are also often used. Mechanical mixers can be installed through the cover or walls of the digester.

Substantial mixing can be effected with about 5 to 8 watts/m^3 of digester content is dissipated in the digester.

Internal gas mixing types normally used for digesters are

1. The injection of a large sludge gas bubble at the bottom of a 30 cm diameter tube to create piston pumping action and periodic surface agitation
2. The injection of sludge gas sequentially through a series of lances suspended from the covers to as great a depth as possible, depending on cover movement

3. The free or unconfirmed release of gas from a ring of spargers mounted on the floor
4. The confined release of gas within a draft tube positioned inside the tank

The first method generally has a low power requirement and consequently, produces only a low level of mixing. As a result, the major benefit derived from its use is in scum control. Lance free gas lift and draft tube gas mixing, however, can be scaled to induce strong mixing of the digester contents.

The circulation patterns produced by these two mixing methods differ. In the free gas lift system, the gas bubble velocity at the bottom of the tank is zero, accelerating to a maximum as the bubble reaches the liquid surface. Since the pumping action of the gas is directly related to the velocity of the bubble, there is no pumping from the bottom of the tank with a free gas lift system.

In contrast, a draft tube acts as a gas lift pump which, by the law of continuity, causes the flow of sludge entering the bottom of the draft tube to be the same as that exiting at the top. Thus, the pumping rate is largely independent of height.

The significance of this difference is that draft tube mixers induce bottom currents to prevent or at least reduce accumulations of settling material. Another difference among internal gas mixing systems is that the gas injection devices in a free gas lift system are fixed on the bottom of the digester and thus, cannot be removed for cleaning without draining the tank.

To reduce clogging problems, provisions should be made for flushing the gas lines and diffusers with high pressure water.

With the lance and draft tube systems, the gas diffusers are inserted from the roof and, therefore, can lie withdrawn for cleaning without removing the contents of the tank.

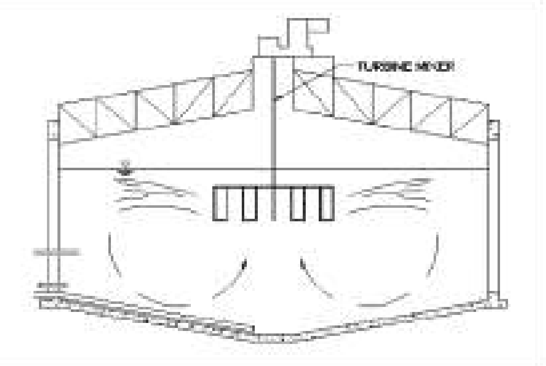
A drawback in these systems, is that the draft tube and gas lines inside the tank may foul with rags and debris in the digesting sludge. Some of these are compiled in Figure 6-9 and Figure 6-10.

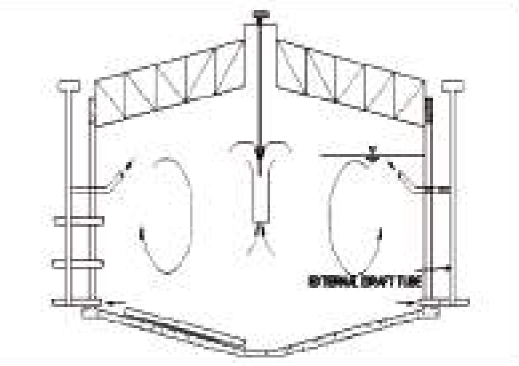
There are many types of sludge mixing arrangements in tandem with the shape of digesters. However, as far as India is concerned, the cylindrical digesters with either mechanical agitator mixers or externally sludge recirculated pump sets are more common.

The cylindrical digester with the upper and lower compartments and gas induced central draft tube mixing system is in use in cattle dung, abattoir waste and vegetable market waste biomethanation plants with subsidy from the Ministry of New and Renewable Energy (MNRE).

However, there is very limited information either in India or elsewhere on the effectiveness of their geometry Vs the efficiency of mixing system. Recently, this had engaged the attention and Computational Fluid Dynamics (CFD) modelling approach is being explored to optimize the degree of mixing for the energy put in.

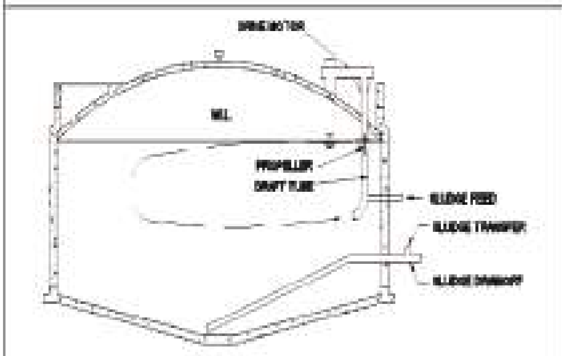
The usefulness of this type of modelling can be seen from the documentation of a reported study as reproduced in Figure 6.11.



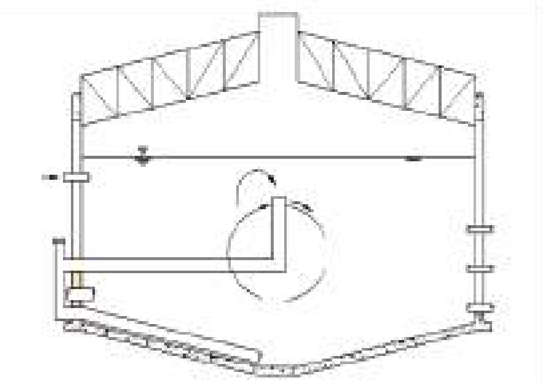


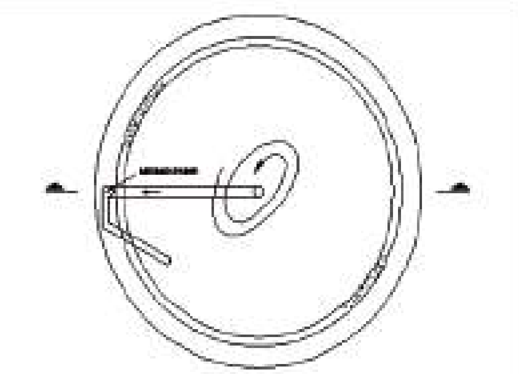
Mechanical mixing system: The entire digester contents are subjected to a central vortex based uplift and its all round throw of the liquor at the surface. This results in a hydraulic mixing in all the vertical planes.

Mechanical mixing system: A combination of a central draft tube with downward turbine & external draft tubes relying on downward hydraulic energy dissipated in the draft tube.



Mechanical mixing system: A combination of multiple downward draft tubes along inner rim with spouted ends to suck in the feed sludge and pump towards the centre and in turn the upward circulation to draft tubes.

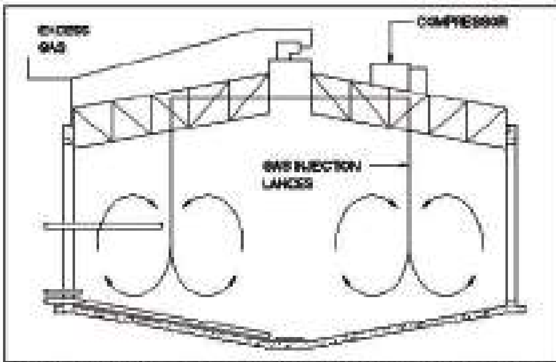
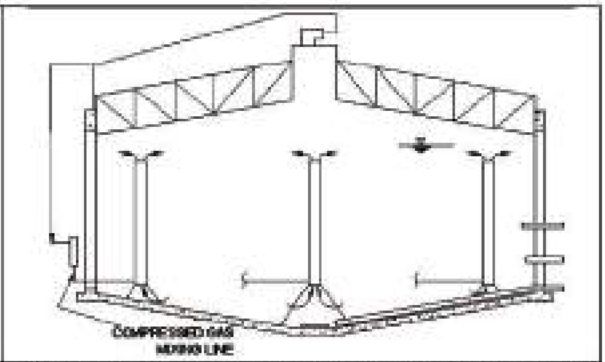
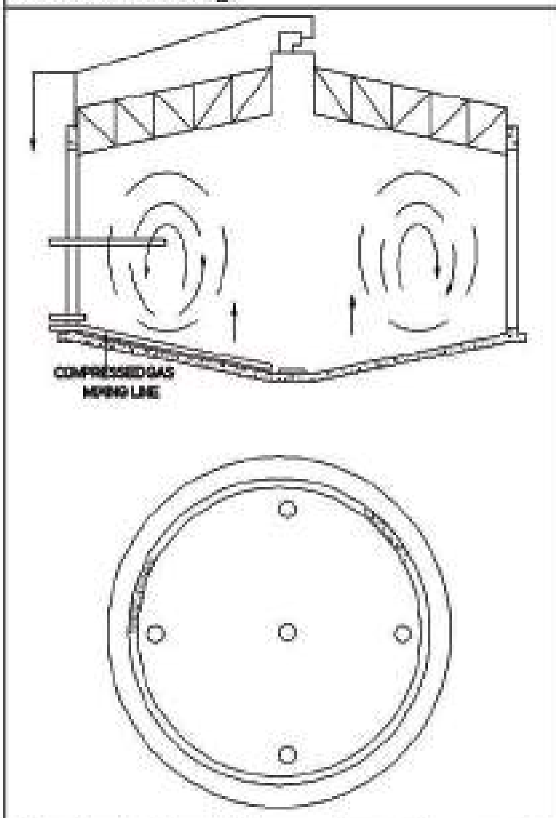
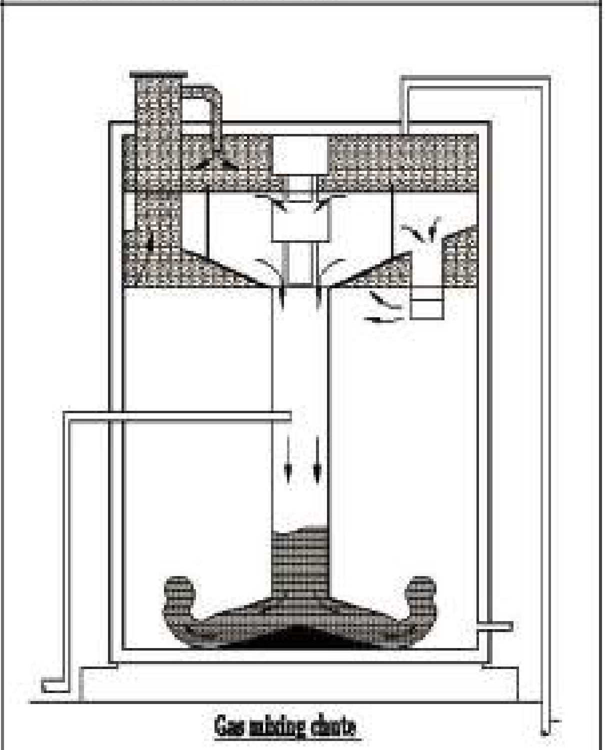




Pumped feed system: The liquor is withdrawn by pumped suction from the tip of the digester conical floor and/or near the upper zone and is pumped back in multiple tangential pumped ends to sustain mixing.

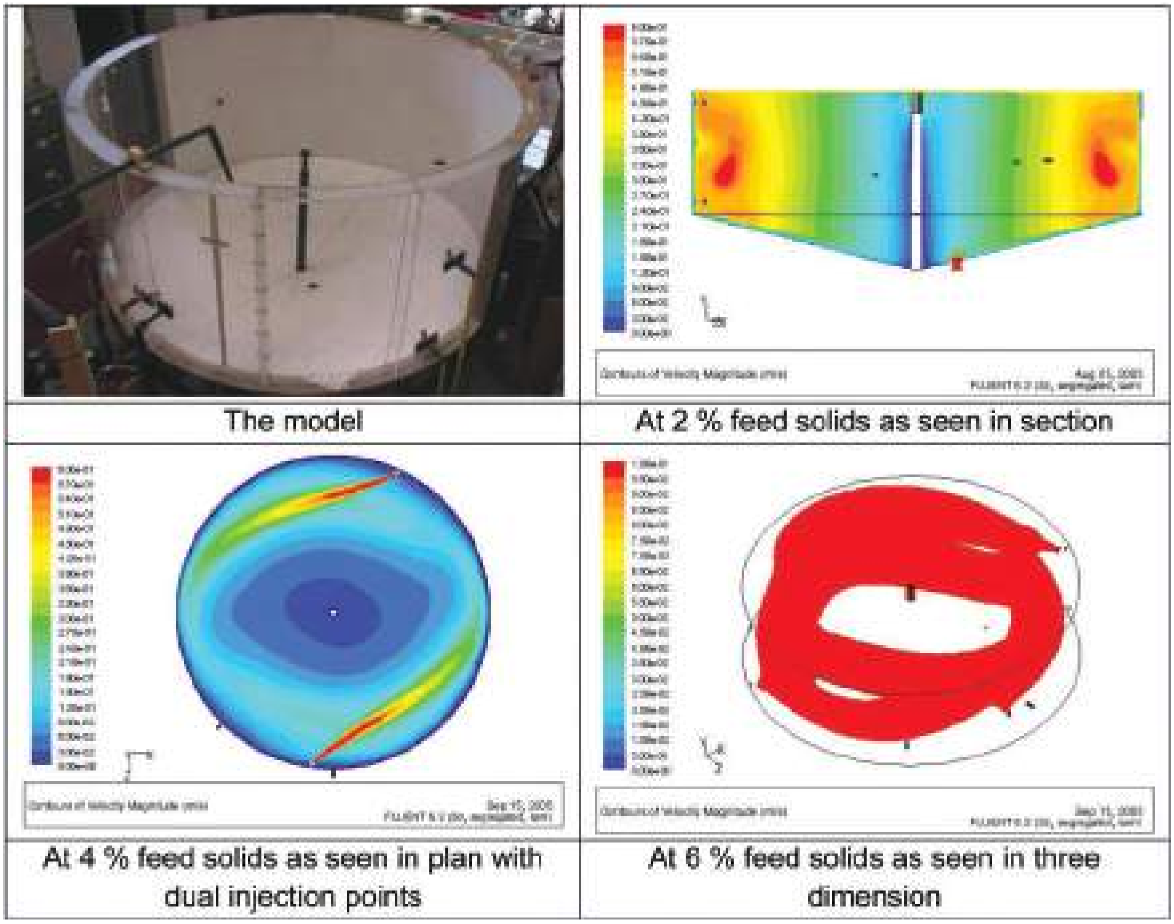
Source: WEF Manual of Practice 8

Figure 6.9 Some of the mechanical mixing systems in digesters

	
Gas injection mixing system: The gas is withdrawn from the roof, compressed and released at the bottom of the submerged lances. This induces upward swell and mixing.	Unconfined Gas mixing system: The gas is withdrawn from the crown, compressed and released at the conical base of inverted cone floor. This escape of gas induces upward swell and mixing.
	
Confined Gas lifting system. The gas is withdrawn from the crown of the digester, compressed and released into the bottom of multiple draft tubes. Its upward escape sets in the buoyancy induced mixing of liquor.	The induced gas mixing system. Once gas attains a pre-set pressure in the digester, it is released into the upper chamber thus inducing a syphon based mixing of the liquor through downward draft tube and floor level lances.

Sources: (1) WEF Manual of Practice-8,
(2) Article on Biomethanation by AK Dhussa and VK Jain, Newsletter, National Bioenergy Board, MNRE, March 2002.

Figure 6.10 Some of the gas based mixing systems in digesters



Source: D Harrison et. al., 2006

Figure 6-11 CFD Modelling results of digester mixing efficiency

CFD is a mathematical modelling technique that solves fundamental flow equations for each cell in a mesh, 1.6 million in this instance, to form a construct of the equipment being investigated. The flow field within the equipment is simulated and flow patterns, velocities, residence time distributions, additive blend times, pressure drops and other relevant parameters can be determined. Many single and some multi-phase flows can be modelled. Knatz. et. al. (2010).

To assess this effect a CFD model of the digester is stated to have been constructed at the Orange County Sanitation District STP in USA using both physical and CFD models. The mixing system is stated to have been simulated at 2 percent, 4 percent and 6 percent dry solids concentration. At 2 percent dry solids concentration, the mixing was shown to be very effective with over 90 percent of the digester volume being actively mixed and the feed blend time being calculated as 1.7 hours. At 4 percent digested solids concentrations the active volume was calculated as 45 percent with a blend time of 8.5 hours and at 6 percent dry solids concentration the active volume was greatly reduced to less than 28 percent and blend time was in excess of a day.

It was inferred that doubling the dry solids concentration within the digester will be more than halve the active volume provided by the digester mixing system.

The CFD model is not just a tool to evaluate the effect of increasing viscosity on digester mixing performance. Once constructed and calibrated, the CFD model can also be used to test modifications to the digester mixing system to improve performance at elevated dry solids concentrations.

Accordingly, it is to be recognized that investigating the integrated approach to size, shape and mixing efficiency is to be given its due importance in firming up future digester systems.

6.4.8 Heating System

Heating and boiler systems are decided after considering the following:

1. Heating method

   i. Direct heating method (steam blow-in system). This consists of blowing high-temperature steam directly into the sludge in the tank. As long as agitation is satisfactory, the degradation of biotic action due to steam is small, both equipment and operation are simple, so this method is widely used. This requires boiler make-up water equivalent to the steam blow-in quantity.

   ii. Indirect heating system (system using heat exchanger) - The indirect heating system consists of a heat exchanger installed outside the tank. The sludge that circulates around the tank and heat exchanger are heated by the hot water circulating around the heat supply source and the heat exchanger. The heat exchanger may be of two types – double tube and spiral type.

   The double tube heat exchanger consists of an inner tube in which the sludge flows and an outer tube around which hot water flows in the opposite direction.

   The spiral type heat exchanger consists of heat transfer plate wound in spiral form within the cylindrical pipes in which sludge and hot water flow in opposite directions.

   The indirect heating system has more equipment such as circulating pump and heat exchanger, compared to the steam blow-in system, but since digested sludge circulates, agitation of the sludge is facilitated.

2. Heat required for heating consists of the heat required for the loaded sludge, and the radiation heat loss to surrounding from the heating pipes and tank.

3. The boiler capacity is decided after considering the maximum heat of the sludge digestion tank, the operating hours and the number of boilers. The number of boilers should be two or more in principle.

4. The construction of the boiler should be based on relevant laws and standards, and should be such that boiler can be operated in a stable manner.

5. Steam pipes should be covered by insulating material; steam trap and vacuum breaker should be installed in the steam piping system. Liquid depth at blow-off and shape of blow-in part should be considered so as to prevent abnormal noise and vibration in the steam blow-in pipes.

6.4.9 Sludge Inlets and Draw-offs

Sludge inlets and draw-offs are decided after considering the following:

1. Thickened sludge inlet pipe should be installed at such a position that sludge is dispersed uniformly in the tank.
2. The sludge withdrawal pipe should be at least 150 mm in suction drawal and 200 mm in gravity drawal.
3. Consideration should be given to preventing clogging of sludge pipes.
4. Considerations should be given to power outages for sludge draw-off valve.

6.4.10 Supernatant Withdrawal and Treatment

For withdrawal and treatment of supernatant liquor, the following should be considered.

6.4.10.1 Supernatant Liquor Draw-off Pipe

The supernatant liquor draw-off pipes should be laid such that supernatant liquor can be drawn off at varying water depths. In a secondary tank for two-stage digestion, supernatant liquor with small SS concentration occurs between the concentrated digested sludge at the bottom and the layer with a major concentration of floating solids near the water surface. The water depth at which satisfactory supernatant liquor is generated differs depending on the digestion level; therefore, draw-off pipes should be installed at 3 to 4 different locations from half the depth of the tank, and the best location from these should be selected.

The draw-off of supernatant liquor in the secondary tank should be performed from the overflow pipe. Piping to bypass the overflow pipe should be kept ready, considering operation by controlling the liquid level during transfer to secondary tank, draw-off of supernatant liquor and withdrawal of digested sludge. The pipe diameter should be minimum 150 mm.

6.4.10.2 Supernatant Liquor Return Piping

After directly treating the supernatant liquor, piping should be laid such that it can be returned to the grit chamber or primary settling tank and so on.

The quantity of supernatant liquor is small compared with the influent sewage, but consideration should be given from the beginning so that high load does not act temporarily on sewage treatment. Although it is common to return the supernatant liquor to the grit chamber and primary settling tank and treated, it may be returned and treated independently if necessary. If discharge piping within the premises is used as return piping, it becomes difficult to understand the quality of influent sewage because of its mixing in the grit chamber.

In case of combined sewerage system, issues of effluent delivery during rainfall exist; therefore, it is preferable to use dedicated piping that is separate from the discharge piping within the premises.

Treatment of supernatant liquor containing highly concentrated COD, phosphorous, nitrogen, etc., should be studied in conjunction with the return flow from other sludge treatment facilities. Measures for return flow with the aim of removing phosphorous include the MAP method developed recently (crystallization of MAP granules). Although the generated MAP granules include potassium, it also includes components and has properties that are suitable for fertilizer, such as nitrogen and phosphorous.

6.4.11 Guidelines for Sludge Piping Architecture

The following guidelines taken from "Design Guidelines for Sewage Works, 2008, Ministry of the Environment, Ontario, PIBS 6879" are considered useful.

1. Digested sludge withdrawal piping should have a minimum diameter of 200 mm (NPS-8) for gravity and 150 mm (NPS-6) for pump suction and discharge lines.

2. Clearance between the end of the withdrawal line and the hopper walls should be sufficient to prevent bridging of the sludge.

3. Adequate provisions should be made for rodding or back-flushing pipe lengths.

4. Where withdrawal is by gravity, the available head on the discharge pipe should be at least 120 cm and more. The same is good for pumped deliveries also.

5. Gravity piping should be laid on uniform grade and alignment.

6. Slopes on gravity discharge piping should be at least 3 percent for primary sludge and at least 2 percent for aerobically digested sludge

7. Where gravity sludge transfer is proposed, provision should be made for a pumped transfer on a regular basis to remove deposits and clean out the lines.

8. Valves should be provided to allow for both gravity and pumped transfer. Cleanouts should be provided for all gravity sludge piping.

9. The section of piping between isolation valves should have drain and vent valves or other means to relieve built-up pressure, due to gas formation, prior to dismantling the piping for cleaning or repairs.

10. Special consideration should be given to the corrosion resistance and permanence of supporting systems for piping located inside the digestion tank.

11. Adequate provisions should be made for rodding or back-flushing individual pipe runs. Piping should be provided to remove sludge for further processing.

12. Air-lift pumps are not recommended for the removal of primary sludge.

13. The tank bottom should slope to drain toward the withdrawal pipe. For tanks equipped with a suction mechanism for sludge withdrawal, a bottom slope not less than 1 to 12 is required.

14. Where the sludge is to be removed by gravity a 1 to 4 slope is recommended.

15. Maximum flexibility should be provided in terms of sludge transfer (a) from primary and secondary units to the digesters, (b) between primary and secondary digesters and (c) from digesters to subsequent treated sludge or biosolids handling operations.

16. An unvalved vented overflow should be provided to prevent damage to the digestion tank and cover in case of accidental overfilling.

17. This emergency overflow should be piped to an suitable point and at an suitable rate to the STP liquid train or side stream treatment facilities to minimize the impact on process units.

18. Many clean outs and plugged tees or crosses should be provided. Elbows and sharp turns should be avoided.

19. If the sludge flow is small, large capacity pumping with a timer should be used to flush the line during the pumping cycle. The pump should have sufficient head to move the settled solids.

20. Provision for high-pressure water jet, pipe rodding and cleaning devices is needed.

21. Long sludge lines should have bypass lines for cleaning and maintenance.

6.4.12 Gas Collection and Storage

6.4.12.1 Gas Collection

Sludge gas is normally composed of about 60% to 70% methane and 25% to 35% carbon dioxide by volume, with smaller quantities of other gases like hydrogen sulphide, hydrogen, nitrogen and oxygen. The combustible constituent in the gas is primarily the methane.

Depending upon the sulphate content of the sewage and the sludge, the concentration of hydrogen sulphide in the gas varies. Hydrogen sulphide in addition to its corrosive properties imposes a limit on the usability or causes nuisance during the burning of the gas.

The gas can be used as heat energy in a gas engine to generate electricity for in plant consumption or piped to nearby institutions like the case of the Dadar STP at Mumbai or merely flared.

Minimum or maximum rates of gas production will however, depend upon the mode of feeding of raw sludge into the digester, when batch feeding is practiced, the minimum and maximum gas production rates may vary from 45% to more than 200%.

In the continuous feeding system, the difference between the maximum and the minimum is considerably reduced. Intermittent mixing of digester contents is also responsible for wide fluctuations in gas production rates.

It is, therefore, desirable to feed the high rate digesters with raw sludge and run the mixing device as continuously as possible to obtain not only a uniform rate of digestion, but also uniform production of gas.

Sludge gas should be collected under positive pressure to prevent its mixing with air and causing explosion. The explosive range of sludge gas is between 5% and 15% by volume of gas with air. The gas may be collected directly from under a floating cover on the digester or from the fixed cover by maintaining a constant water level. Where primary and secondary units are provided to operate in series with the primary having a fixed cover and the secondary with a gas holding or floating cover, the gas piping from each digester should be interconnected. A separate gas holder may be provided to collect the gas from the primary unit where the secondary units are kept open.

A fixed integrally built gas dome above the digester roof is advantageous for gas take off. The velocity in sludge gas piping should not exceed 3.5 m/s to prevent carryover of the condensate from the condensation traps and avoid high pressure loss and damage to meters or flame traps and other appurtenances of the system.

An integrating meter made of corrosion resistant material should be used to measure gas production from the digesters. Removal of condensate from the meter is also desirable. Pressure release valves are provided for controlling the gas in the digester by releasing gas pressure exceeding 200 to 300 mm of water and also preventing partial vacuum and possible cover collapse during rapid withdrawal of sludge or gas.

A mandatory distance of at least 30 m should be kept between a waste gas burner and a digestion tank or gas holder to avoid the possibility of igniting the gas mixture. Waste gas burners should be located in the open for easy observation. A pilot device should also be provided with the waste gas burners. Condensate traps, pressure release valves and flame traps should also be provided ahead of waste gas burners. Manometers indicating the gas pressure in cm of water may be used on the main gas line from the digester or ahead of the gas utilization device. A common open end U tube manometer should not be used for such purposes as it may be hazardous.

6.4.12.2 Storage (Gas Holder)

The primary purpose of a gas holder is to adjust the difference in the rate of gas production and consumption as well as to maintain uniform pressure at the burner. When gas holders are also used for storage of gas for utilization, a storage capacity of at least 25% of the total daily gas production should be provided. The gas holders may be of the following types:

1. A bell shaped cylindrical tank submerged in water installed either on the top of a digester or as a separate unit. The structure holding the water may be made of RCC. As the gas enters or leaves, the holder rises or falls.

2. A pontoon cover type that floats on the liquid content of the digester consisting of steel ceiling, skirt plates, a gas dome and steel trusses

3. Dry type gas holder consisting of a cylindrical steel tank in which a disc-shaped piston makes contact at its periphery with the inside of the tank. The gas enters the holder from beneath the piston which floats on the gas. Leakage of gas is prevented by either tar or a felt seal around the edge of the piston. A suitable roof should be provided if this type of dry gas holder is installed.

4. A high pressure holder either cylindrical or spherical in shape and made of either welded or netted steel, for storing the gas under high pressure. This type of gas holder is seldom used for sewage treatment plants unless the gas has to be utilized for special purposes.

5. A relatively trouble-free gas holder is the flexible inflatable fabric top, as it does not react with the H2S in the biogas and is integral to the digester. These types of covers are often used with plug-flow and complete-mix digesters. Flexible membrane materials commonly used for these gas holders include high-density polyethylene (HDPE), low-density polyethylene (LDPE), linear low density polyethylene (LLDPE), and chlorosulfonated polyethylene covered polyester.

The appurtenances for gas holders include ladders, condensate drains, pressure gauges and safety valves. Typical drawings and photographs are as in Figure 6.12 and Figure 6.13.

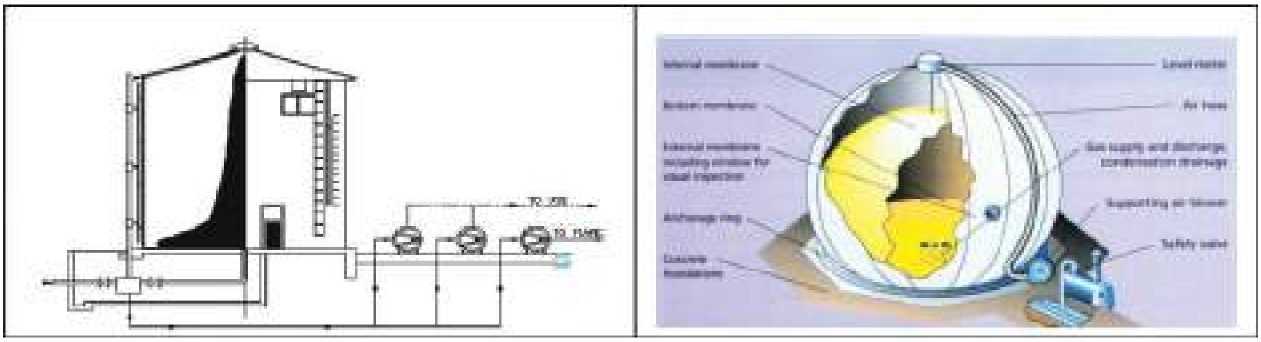


Figure 6.12 Illustrative flexible gas holder configurations.

At left in Figure 6.12 is a top held holder. It expands in the shape of a cone and is housed inside a structure. The left half shows the inside and the right half is the outer with doorway, vents and ladder. At right in Figure 6.12 is a bottom held holder. It expands as a sphere and is protected by another outer sphere. The filling and withdrawal of gas is at the bottom. Lightning arresters are mandatory. (There are many other configurations and the ones shown here are only illustrative).

Figure 6.13 A view of the gas holders.

6.4.13 Piping

Cast iron is commonly used for pipelines carrying sludge including fittings and joints. Pipes should be well supported and be capable of being drained. Vents should be provided at high points in order that the gas generated by the digesting sludge does not accumulate in these pipelines.

Adequate number of flanges and flexible couplings should be provided on exposed sludge lines to facilitate dismantling and insertion of cleaning equipment whenever necessary. In long pipe runs, tees with flanges equipped with 40 to 60 mm hose connectors should be provided for easy cleaning and flushing of the pipe. Flushing is an important requirement and arrangements should be provided for flushing with water or treated sewage.

A minimum diameter of 200 mm should be used for the sludge pipelines for both gravity withdrawal and suction to pumps. Velocities of 1.5 to 2.4 m/s should preferably be maintained to prevent solids deposition and accumulation of grease.

Primary and digested sludge have different hydraulic characteristics from those of water, though the secondary sludge is almost similar to water in its characteristics. The head loss in sludge pipes increase with the increase in percentage of solids and as such ‘C’ values of 40 to 50 in the Hazen William formula should be used for designing the pipelines.

For gas lines CI, GI or HDPE are commonly used. Galvanized steel may also be used for exposed gas piping. Flanged joints may be provided for exposed gas piping of sizes 100 mm and above in diameter while screw or welded type joints are recommended for pipe less than 100 mm. Mechanical joints should be used for underground piping. It is necessary that all gas piping be located at a level that will allow proper draining of the condensate. It is desirable to maintain a gas pipe slope of 1 in 50 with a minimum of slope of 1 in 100 for adequate drainage. Gas pipes should preferably be painted with bituminous coating. For a diameter of 100 mm and above, CI with flanged joints or flexible mechanical joints are used.

Adequate pipe supports should be provided to prevent breakage. It is desirable to provide a flanged pipe bypass before a gas meter, a firm foundation should also be laid below the pipe and caution must be exercised during back filling to prevent any disturbance of the alignment and grade. In highly acidic or alkaline soils, the pipe must be wrapped with either asbestos or some other protective material. Coal tar enamel may also be used in some cases. Cathodic protection is not generally needed on gas lines.

Adequate number of drip traps must be provided in gas pipelines, especially at the downward bends. Suitable number of tees should also be provided with removable screwed plugs or flanges for cleaning purposes. A drip trap of 1 litre capacity would be satisfactory. Trap outlets should run to floor drains wherever convenient. It is preferable to use positive type traps, which prevent gas from escaping while emptying the condensate.

6.4.14 Appurtenances

6.4.14.1 Sampling Sinks and Valves

A sink should be provided for each digester unit for drawing the supernatant liquor and sludge from various levels in the digester. Sinks should either be of white enamelled cast iron or of stainless steel. They should be made at least 30 cm deep. The supply of adequate water for flushing the sinks should also be provided. The sludge sampling pipes of SS should be short and 40 to 50 mm in diameter and arranged to draw samples from at least three levels at 0.6 m intervals. Sink valves should be brass plug type or CI flanged type.

6.4.14.2 Liquid Level Indicator

The digester may be designed for a fixed liquid level. Alternatively, a liquid level indicator with gauge board or any other device may be used for each digester.

6.4.15 Corrosion Prevention

The temperature and humidity of digester gas are high, and the gas contains hydrogen sulphide, hence, it is strongly corrosive. Therefore, preventive measures are necessary for parts in contact with sludge including digester gas. The following should be considered in relation to preventing corrosion:

6.4.15.1 Material of Equipment and Piping Parts in Contact with Sludge and Digester Gas before Desulphurization

Stainless steel and ductile cast iron coated with epoxy powder coating, etc., should be used as the material of equipment and piping parts in contact with sludge and digester gas before desulphurization. If the use of carbon steel is unavoidable, adequate measures such as preventing corrosion by providing a lining, etc., should be considered. Corrosion resistant synthetic rubber should be used in gaskets of flanges. Natural rubber should not be used.

6.4.15.2 Coating of Parts of Piping and Equipment in Contact with Digester Gas and Sludge

Epoxy resin-based paints and coating should be used considering resistance to heat and chemical action for coating on parts in contact with digester gas and sludge. To prevent rise in temperature of gas holder due to sunlight, the coating on the outside should preferably be in a colour that absorbs very little light from the sun.

6.4.15.3 Stripping of Hydrogen Sulphide in Digester Gas

Typically, the digester gas contains Methane (CH_4), Hydrogen Sulphide (H_2S) and Carbon Di-oxide (CO_2). Standard requirements for dealing with digester gas are either electricity generation by using appropriate engines or flaring in the atmosphere. In the case of flare, the auto ignition temperature of H_2S is 260°C and that for CH_4 is 595°C thus, well before ignition of CH_4 the H_2S is disintegrated and the foul odour aspect is not relevant.

In the case of feeding into engines, the H_2S in the presence of moisture gets converted to sulphurous acid, which is corrosive and damages the burner hence, the removal of this, is essential before using the gas in engines. It is not that it has to be eliminated fully. Engines can generally tolerate up to about 200 ppm of this gas volume by volume. The permissible concentration of sulphates in drinking water is 200 mg/l and the limit permissible when alternate sources are not available is 400 mg/l. Thus, wherever STPs are planned, the issue of sulphates in raw sewage would appropriately dictate the need for the removal of sulphide in case the biomethanation route and electricity generation from engines is contemplated.

The CMWSSB has installed and is operating three different types of stripping facilities for digester gas in three different STPs. The operating principle of each of these and a summary of observations is compiled in the following section.

6.4.15.3.1 Biochemical Process

The raw biogas containing H_2S is passed through a scrubbing tower from the bottom and the process liquid containing thiobacillus is sprayed from the top. The tower is filled with raschig rings for enhancing the contact between biogas and process liquid whereby, the H_2S is washed and carried away in the liquid and circulated via a reactor tank. The reactions are

$H_2S + OH \quad = HS + H_2O$
$HS + (O_2)/2 \quad = S + OH$

The regeneration of process liquid involves (a) Caustic solution, (b) Nutrient for thiobacillus, (c) Oxygen by way of air and (d) Demineralized water. The process liquid is transferred to a settling tank where the sulphur is stated to be settled and the supernatant flows back to the reactor tank. The settled sulphur is to be pumped to drying beds for recovering elemental sulphur. The controlling parameters are pH, Redox potential and TDS of process liquid. The process units are scrubber tower, reactor tank, settling tank and drying beds.

The process machineries are biogas blower, air blower, spray pump, caustic dosing pumps, measuring pumps and sludge recirculation pumps. An issue is the sludge, which if not used as recovered sulphur would become a hazardous sludge and need secure landfill.

6.4.15.3.2 Biological Process

The biogas containing H_2S is stated to be oxidized to Sulphate according to the equations by using the sulphur oxidizing thiobacillus organisms grown on fluidized media in the tower.

$2HS + O_2 \quad = 2S + 2\ OH$
$2HS + 4O_2 \quad = 2SO_4 + 2\ H$

The process units involved are packing media in the tower for growth of thiobacillus, air supplying blower, recirculation pump to provide moisture for the bacteria and nutrient feed pump for maintaining the nitrogen, potassium and phosphorous balance. The removed H_2S ends up as the dissolved sulphate in the product residue stream which is returned to the inlet of the STP.

The issue of importance is the Stoichiometric increment in SO_4 hence, the TDS in the final treated sewage may not be significant as the H_2S itself is derived from the SO_4 present in the raw sewage. The system is provided with its SCADA.

6.4.15.3.3 Chemical Process

The biogas containing the H_2S is first passed through a venturi scrubber sprayed with caustic solution at two or three stages under pressure followed by a second in a counter current packed bed tower filled with porous raschig rings and the H_2S gets dissolved into the caustic and water.

The water coming out of the scrubber will be acidic and is treated with caustic solution which is re-circulated again.

The scrubbed gases are drawn through induced draft fan and forced into the downstream engines. The venturi as well as the packed towers is mounted on the same reservoir and the two liquids are segregated by compartments and individual recirculation pumps are provided, which circulate the liquids till they reach saturation.

The units are venturi scrubber, packed bed tower, water circulation tank, water circulation pump, chemical dosing system comprising dosing pumps and pH controller, dosing pumps for chemical storage tanks, water circulation pipe with valves, fittings and controls. The system has a custom built instrumentation unit for control.

6.4.15.3.4 Observations of the Systems

The results of performance of the three systems as functioning at the STPs of CMWSSB are compiled in Table 6.10. The installations are shown in Figure 6-14 overleaf.

Table 6.10 Observed performances of the three systems for removing H_2S gas

No.	Parameter	Inlet	Representative values in the processes
1	CH_4 %	60 to 63	64.00 to 70.20
2	CO_2 %	20 to 28	20.51 to 27.20
3	Hydrogen %	Nil to 2	Trace to 7.20
4	Nitrogen %	1.3 to 6.4	1.1 to 6.8
5	Oxygen %	0.5 to 1.2	0.40 to 2.25
6	H_2S ppm	200 to 500	10 - 20 ppm
7	Moisture %	varying	About 50 ppm
8	Calorific value	4,400 to 4,800	5,500 to 5,800 kcal/nm^3

6.4.15.3.5 Appraisal of the Systems in Terms of O&M

The chemical system is the oldest and the bio chemical system is the next generation with the biological system as the recent system. In terms of relative comparison, the biological system has least residues, but has to be maintained and managed carefully for the thiobacillus biomass to be maintained with respect to its nutrient requirements and the avoiding of the media getting too much growth thereon and getting clogged.

The bio-chemical system uses the porous raschig rings and to that extent, its excessive growth and clogging is a matter of concern and nutrient addition is also to be maintained.

The recovery of elemental sulphur, for example, if the raw sewage has a SO4 concentration of 300 mg/l, the elemental sulphur recovery even at 100% can be about 100 kg/MLD, but the aspects of its purity and consistent yield are to be critically rated.

Top Left: Bio-chemical scrubber in the foreground and dewatering beds at far end for Sulphur at Perungudi STP, Chennai, Right: Biological scrubber and sulphide gas disposed off as dissolved sulphate to inlet of the Perungudi STP, Chennai, Bottom Left: Chemical scrubber with first stage venturi and second stage packed bed at Koyambedu STP, Chennai, Caustic is used for naturalization is bled when saturation limit is met with

Figure 6.14 Digester gas stripping systems in use for stripping hydrogen sulphide.

Moreover, if the H_2S cannot be reduced fully the foul odour may persist. Upsets in biological sustainability is a common factor for both these processes. The chemical process has the advantage of not relying on the microbial population and its control, but the potential to achieve a 100% stripping of H_2S demands a continuous monitoring and day-to-day adjustments of chemical dosages, which is further compounded by variations in raw sewage quality and hence, variations in sludge gas produced.

Between the biological and chemical processes, relatively higher electrical consumption and feed of caustic are to be considered. Each system needs to be evaluated for the given project on hand.

6.5 AEROBIC SLUDGE DIGESTION

6.5.1 General

Aerobic digestion is also a useful method of stabilizing sewage sludge. It can be used for secondary tank humus or for a mixture of primary and secondary sludge, but not for primary sludge alone. The advantages of aerobic digestion over the anaerobic digestion are listed overleaf:

1) lower BOD concentration in digester supernatant
2) odourless and easily de-watered biologically stable digested sludge
3) recovery of more basic fertilizer value in the digested sludge
4) lower capital cost and
5) fewer operational problems

Due to these advantages, aerobic digesters are being increasingly used particularly in small STP. However, operating cost in terms of the power cost is much higher than for anaerobic digesters.

The factors that should be considered in designing an aerobic digester include detention time, loading criteria, oxygen requirement, mixing and process operation. The volatile solids destroyed in aerobic digestion in 10 to 12 days at a temperature of 20ºC would be 35 to 45%.

Higher temperature will result in reduction in the period of digestion. Oxygen requirements normally vary between 1.7 to 1.9 gm / gm of volatile solids destroyed. It is also desirable to maintain the dissolved oxygen between 1 to 2 mg/l in the system. Operational difficulties may be expected if compressed aeration is practiced. Extended aeration system including oxidation ditches are examples of aerobic digestion.Mechanical or jet aerators or sparger systems are preferable.

6.5.2 Tank Capacity

6.5.2.1 Volume Required

The following digestion tank capacities are based on a solids concentration of 2% with supernatant separation performed in a separate tank. If supernatant separation is performed in the digestion tank, a minimum of 25% additional volume is required. These capacities shall be provided unless sludge thickening facilities are utilized to thicken the feed solids concentration to greater than 2%. If it is provided, the digestion volumes may be decreased proportionally. The sludge source and volume/ population equivalent is in Table 6.11.

Table 6.11 Sludge source and volume/population equivalent

Sludge Source	(m^3/P.E.)
Waste activated sludge - no primary settling	0.13 (A)
Primary plus waste activated sludge	0.11 (A)
Waste activated sludge excluding primary sludge	0.06 (A)
Extended aeration activated sludge	0.09
Primary & attached growth biological reactor sludge	0.09

Note: (A) These volumes also apply to waste activated sludge from single stage nitrification with less than 24 hours detention time based on design average flow;

P.E.-population equivalent as design flow divided by per capita sewage flow.

6.5.2.2 Effect of Temperature on Volume

The volumes in Section 6.5.2.1 are based on digester temperatures of 15°C and a solids retention time of 27 days. Aerobic digesters should be covered to minimize heat loss for colder temperature applications. Additional volume or supplemental heat may be required if the land application method is used. Refer to Section 6.5.9 for necessary sludge storage.

6.5.3 Mixing

Aerobic digesters shall be provided with mixing equipment which can maintain solids in suspension and ensure complete mixing of the digester contents.

6.5.4 Air Requirements

Sufficient air shall be provided to keep the solids in suspension and maintain dissolved oxygen between 1 mg/l and 2 mg/l. For minimum mixing and oxygen requirements, an air supply of 0.5 l/(m3 s) of tank volume shall be provided with the largest blower out of service. If diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service.

If mechanical turbine aerators are utilized, at least two turbine aerators per tank shall be provided to permit continuity of service.

Mechanical aerators are not recommended for use in aerobic digesters where freezing conditions will cause ice build-up on the aerator and support structures.

6.5.5 Supernatant Separation

Facilities shall be provided for effective separation or decanting of supernatant. Separate facilities are recommended; however, supernatant separation may be accomplished in the digestion tank provided additional volume is provided as per Section 6.5.2. The supernatant draw-off unit shall be designed to prevent return of scum and grease back to plant process units. Provision should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

6.5.6 Scum and Grease Removal

Facilities shall be provided for the effective collection of scum and grease from the aerobic digester for final disposal and to prevent its return to the plant process and to prevent long term accumulation and potential discharge in the effluent.

6.5.7 High Level Emergency Overflow

A high level overflow without any valve and all necessary piping shall be provided to return the digester overflow to the head of the plant or to the aeration process in case of accidental overfilling.

Design considerations related to the digester overflow shall include waste sludge rate and duration during the period the plant is unattended, potential effects on plant process units, discharge location of the emergency overflow and potential discharge of suspended solids in the plant effluent.

6.5.8 Aerobic Digestion Sludge Production

For calculating sludge handling and disposal needs, sludge production values from aerobic digesters shall be based on a maximum solids concentration of 2 % without additional thickening.

The solids production on dry weight basis shall be as follows.

Primary plus waste activated sludge - at least 0.07 kg/(Population Equivalent per day).

Primary plus fixed film sludge - at least 0.05 kg/(Population Equivalent per day).

6.5.9 Digested Sludge Storage Volume

6.5.9.1 Sludge Storage Volume

Sludge storage must be provided in accordance with Section 6.2.7 to accommodate daily sludge production volumes and as an operational buffer for unit outage and adverse weather conditions. Designs utilizing increased sludge age in the activated sludge system as a means of storage are not acceptable.

6.5.9.2 Liquid Sludge Storage

Liquid sludge storage facilities shall be based on the following values unless digested sludge thickening facilities are utilized (refer to Section 6.3) to provide solids concentrations of greater than 2%. The sludge source and volume / population equivalent is given in Table 6.12.

Table 6.12 Sludge source and volume/population equivalent

Sludge Source	m^3/(P.E.)
Primary plus waste activated sludge (WAS), extended aeration WAS	0.004
Waste activated sludge exclusive of primary sludge	0.002
Primary plus attached growth biological reactor sludge	0.003

P.E.-population equivalent as design flow divided by per capita sewage flow.

6.6 SLUDGE DRYING BEDS

6.6.1 Applicability

This method can be used in all places where adequate land is available and dried sludge can be used for soil conditioning. Where digested sludge is deposited on well drained bed of sand and gravel, the dissolved gases tend to buoy up and float the solids leaving a clear liquid at the bottom, which drains through the sand rapidly.

The liquid drains off in a few hours after which drying commences by evaporation.

The sludge cake shrinks producing cracks, which accelerates evaporation from the sludge surface. The areas having greater sunshine, lower rainfall and lesser relative humidity, the drying time may be about two weeks while in other areas, it could be four weeks or more. Covered beds are not generally necessary.

6.6.2 Unit Sizing

The sludge drying process is affected by weather, sludge characteristics, system design (including depth of bed) and length of time between scraping and lifting of sludge material. High temperature and high wind velocity improve drying, while high relative humidity and precipitation retard drying.

6.6.2.1 Area of Beds

The area needed for dewatering and drying the sludge is dependent on the volume of the sludge, cycle time required to retain sludge for dewatering, drying and removal of sludge and making the sand bed ready for next cycle of application and depth of application of sludge on drying bed. The cycle time between two drying cycles of sludge on drying beds primarily depends on the characteristics of sludge including factors affecting its ability to allow drainage and evaporation of water, the climatic parameters that influence evaporation of water from sludge and the moisture content allowed in dried sludge. The cycle time may vary widely, lesser time required for aerobically stabilized sludge than for anaerobically digested sludge and for hot and dry weather conditions than for cold and/or wet weather conditions.

The area of land required for sludge can be quite substantial with values of 0.1 to 0.25 m^3/capita being reported for anaerobically digested sludge under conditions that are unfavourable for dewatering and drying. The average cycle time for drying may range from a few days to 2 weeks in warmer climates to 3 to 6 weeks or even more in unfavourable ones. The worked out example is presented at Appendix A.6.5.

6.6.3 Percolation Type Bed Components

A sludge drying bed usually consists of a bottom layer of gravel of uniform size over which, a bed of clean sand is laid. Open jointed tile under drains are laid in the gravel layer to provide positive drainage as the liquid passes through the sand and gravel.

6.6.3.1 Gravel

Graded gravel is placed around the under drains in layers up to 30cm with a minimum of 15 cm above the top of under drains. At least 3 cm of the top layer shall be gravel of 3 mm to 6 mm.

6.6.3.2 Sand

Clean sand of effective size of 0.5 mm to 0.75 mm and uniformity coefficient not greater than 4.0 is used. The depth of sand may vary from 20 cm to 30 cm. The finished sand shall be levelled.

6.6.3.3 Under Drains

Under drains are made of vitrified clay pipes or tiles or other suitable materials of at least 10 cm diameter laid with open joints. Under drains shall not be more than 6 m apart.

6.6.3.4 Walls

Walls shall preferably be of masonry and extend at least 40 cm above the sand surface. Outer walls should be kerbed to prevent washing outside soil on to beds.

6.6.3.5 Dimensions

Drying beds are commonly 6 m to 8 m wide and 30 to 45 m long. A length of 30 m away from the inlet should not be exceeded with a single point of wet sludge discharge, when the bed slope is about 0.5 %. Multiple discharge points may be used with large sludge beds to reduce the length of wet sludge travel.

6.6.3.6 Sludge Inlet

All sludge pipes and sludge inlets are so arranged to easily drain and have a minimum of 200 mm diameter terminating at least 30 cm above the sand surface. Splash plates should be provided at discharge points to spread the sludge uniformly over the bed and to prevent erosion of the sand.

6.6.3.7 Cover

Sludge drying bed in high rainfall areas in the country needs cover with FRP etc., in accordance with requirement.

6.6.3.8 Drainage

Drainage from beds should be returned to the primary settling units if it cannot be satisfactorily disposed of otherwise.

6.6.4 Sludge Removal

6.6.4.1 Preparation of Bed

Sludge drying beds should be prepared well in advance of the time of application of a fresh batch of sludge. All dewatered sludge, which has formed a cake, should be removed by rakes and shovels or scrapers, care being taken not to pick up sand with the sludge.

After the complete removal of sludge cake, the surface of the bed is cleaned, weeds and vegetation removed, the sand levelled and finally the surface properly raked before adding the sludge. The raking reduces the compaction of the sand and improves the filterability of the bed.

Only properly digested sludge should be applied to the drying beds.

Poorly digested sludge will take a much longer time for dewatering. Sludge containing oils, grease and floating matter clog the sand and interfere with percolation.

Sludge samples from the digester should be examined for the physical and chemical characteristics to ensure that it is ready for withdrawal.

6.6.4.2 Withdrawal of Sludge

Sludge should be withdrawn from the digester at a sufficiently high rate to clear the pipeline. Rodding and back-flushing of the inlet pipe may sometimes become necessary to make the material flow easily. Valves must be opened fully to start with and later adjusted to maintain regular flow. The flow may be regulated to keep the pipe inlet from being submerged. Naked flames should be prohibited while opening sludge valves and exposed discharge channels.

6.6.4.3 Removal of Sludge Cake

Dried sludge cake can be removed by shovel or forks when the moisture content is less than 70 %. When the moisture content reaches 40 % the cake becomes lighter and suitable for grinding. Some sand always clings to the bottom of the sludge cake and results in loss of sand thus reducing the depth of the bed. When the depth of the bed is reduced to 10 cm, clean coarse sand that matches the original sand already in the bed should be used for replenishment to the original depth of the bed.

6.6.4.4 Hauling and Storage of Sludge Cakes

Wheel barrows or pickup trucks are used for hauling of sludge cakes. In large plants mechanical loaders and conveyors may be required to handle large quantities of dried sludge. Sludge removed from the bed may be disposal of directly or stored to make it friable, thereby improving its suitability for application to soil.

6.7 SLUDGE DEWATERING

6.7.1 General

Most of the digested primary or mixed sludge can be compacted to a water content of about 90% in the digester itself by gravity, but mechanical dewatering with or without coagulant aids or prolonged drying on open sludge drying beds (SDB) may be required to reduce the water content further. The dewatering of digested sludge is usually accomplished on SDB, which can reduce the moisture content to below 70% or by mechanical equipment. However, excess oil or grease in the sludge will interfere with the process. Where the required space for SDB is not available, sludge conditioning, followed by mechanical dewatering in vacuum filters, filter presses or centrifugation followed by heat drying or incineration can be adopted. In most parts of the country, the climate is favourable for open SDB, which is economical and easy to manage.

Dewatering methods include filtration and mechanical separation. Filtration may be performed by pressure filtration or filtration by belt press filter, pressure filter, vacuum filter, screw press dewatering equipment and multi disc dehydrator. Of these, the dewatering performance and operability and maintainability (especially increase in dewatered sludge) of pressure filter and vacuum filter are inferior compared to those of other systems; therefore, instances of using these together have reduced, and in recent years, there are practically no instances of adoption of new pressure and vacuum filters.

In recent years, modifications have been made to screw press dewatering equipment, and these are being used in small-scale facilities.

6.7.2 Features

Vacuum filtration is the most common mechanical method of dewatering, with filter presses and centrifugation being the other methods. Chemical conditioning is normally required prior to mechanical methods of dewatering. Mechanical methods may be used to dewater raw or digested sludge preparatory to heat treatment or before burial or landfill. Raw sludge is more amenable to dewatering by vacuum filtration because the coarse solids are rendered fine during digestion. Hence, filtration of raw primary mixture of primary and secondary sludge permits slightly better yields, lower chemical requirement and lower cake moisture contents than filtration of digested sludge. When the ratio of secondary to primary sludge increases, it becomes more and more difficult to dewater in the filter. The feed solids concentration has a great influence, the optimum being 8% to 10%. Beyond 10% sludge becomes too difficult to pump and lower solids concentration would demand unduly large filter surface. In this method, conditioned sludge is spread out in a thin layer in the filtering medium, the water portion being separated due to the vacuum and the moisture content is reduced quickly.

6.7.3 Dewatering Principle

Belt press filter comprises two mechanisms; filtering and dewatering by compression. The gravity dewatering part in a belt press filter corresponds to the filter, while the compression part corresponds to dewatering by compressing the sludge. The centrifugal dewatering machine performs dewatering by solid-liquid separation by centrifugal force.

The filtration rate can be expressed by the flow rate of the filtrate. The filtration rate per unit filter area can be expressed by pressure and filtration area.

Dewatering by compression between rolls consists of compressing air gaps in the dewatered sludge and squeezing out the pore water.

The centrifugal separation characteristics vary according to the diameter of particles in sludge, density and mixing action. Centrifugal dewatering machines may be classified according to the solids separating function and the dewatering function that reduces the water content of solids that are settled and separated.

6.7.4 Sludge Conditioning

Thickened sludge and digested sludge includes large amount of organic matter with high affinity to water; moreover, the sludge has particles of various sizes and shapes, so the sludge is difficult to compress and dewatering the sludge as-is, is difficult.

To improve dewatering of the sludge, the quality of sludge should be improved physically and chemically before dewatering it and sludge should be conditioned with the aim of stabilizing its properties and coagulating it. In recent years, treatment plants have been adopting the so-called separation and thickening method in which, the primary sludge is thickened by gravity with mechanical thickening method used for excess sludge.

This method is being increasingly used because of the difficulty in thickening sludge generated from treatment systems. In such cases also, sludge conditioning is necessary for stabilizing sludge properties by uniformly mixing two kinds of sludge with varying properties. Sludge conditioning includes mixing of sludge, elutriation of sludge and adding of coagulant. Sludge mixing refers to the quantitative mixing of two or more kinds of sludge with varying properties based on the generated solids volume ratio to achieve consistency.

For elutriation of sludge, the digested sludge is elutriated with secondary treated water and the alkalinity of sludge is reduced so that the usage of coagulants can be reduced. It is often omitted when organic coagulants are used in the dewatering process. Chemical addition combines fine particles in sludge and makes solid-liquid separation easy, generates floc, and improves dewatering of sludge. Coagulants used may be organic or inorganic.

Coagulants are determined by the kind of dewatering machine used. That is, inorganic coagulants are used in vacuum filters and pressure filters, while organic coagulants are used in belt press filters, centrifugal dewatering machines and screw press dewatering machines. In recent years, belt press filters and centrifugal dewatering machines are mostly adopted and organic coagulants are widely used.

Prior conditioning of sludge before application of dewatering methods renders it more amenable to dewatering. Chemical conditioning and heat treatment are the two processes normally employed.

6.7.4.1 Chemical Conditioning

Chemical conditioning is the process of adding certain chemicals to enable coalescence of sludge particles facilitating easy extraction of moisture. The chemicals used are ferric and aluminium salts and lime, the more common being ferric chloride with or without lime. Digested sludge, because of its high alkalinity exerts a huge chemical demand and therefore, the alkalinity has to be reduced to effect a saving on the chemicals. This can be accomplished by elutriation.

Polyelectrolyte is useful for sludge with finely dispersed solids. The choice of chemical depends on pH, ash content of sludge, temperature and other factors. Optimum pH values and chemical dosage for different kinds of sludge has to be based on standard laboratory tests.

The dosage of ferric chloride and alum for elutriated digested sludge is 1.0 kg/m^3 of sludge.

Alum when vigorously mixed with the sludge reacts with the carbonate salts and release CO_2, which causes the sludge to separate and water drains out more easily. Hence, for effective results, alum must be mixed quickly and thoroughly. The alum floc, however, is very fragile and its usefulness has to be evaluated vis-a-vis ferric chloride before resorting to its application.

Feeding devices are necessary for applying chemicals. Mixing of chemicals with sludge should be gentle, but thorough, taking not more than 20 to 30 seconds. Mixing tanks are generally of the vertical type for the small plants and of the horizontal type for large plants. They are provided with mechanical agitators rotated at 20 rpm to 80 rpm.

6.7.4.2 Use of Polyelectrolytes

In the use of mechanical dewatering of digested sludge, equipment such as filter press and centrifuge are relatively popular in usage. The performance of these would however be enhanced if the feed sludge is conditioned by use of polyelectrolyte. It is difficult to specify any formula suitable for a polyelectrolyte. Even though there are quite a few such polyelectrolytes in the market, it is best to carry out an actual laboratory scale testing before launching into the procurement. In general, polyelectrolytes are available in both powder form and viscous liquid form. In essence both are same as far as usage is concerned, because the powder immediately on being added to water will also become viscous. Usually the dosage needed is expressed as kg of polyelectrolyte needed for ton of dry solids in the sludge to be conditioned for subsequent mechanical dewatering.

The solution for dosing is mostly a 0.1% solution and is injected into the feed line to dewatering equipment with adequate length and velocity ahead of the dewatering equipment. The type of pump which can be generally suitable is the diaphragm pump powered by a reciprocating shaft of a push-pull motion, which injects the calculated dosages as intermittent jets. Peristaltic pump sets are also used sometimes when the viscosity of the feed sludge solution is not very high. In all cases, actual on-site treatability evaluation is called for in respect of choice of polyelectrolytes.

6.7.4.3 Elutriation

The purpose of elutriation of sludge is to reduce the coagulant demand exerted by the alkalinity of the digested sludge, by dilution with water of lower alkalinity followed by sedimentation and decantation. Some end products of digestion such as ammonium bicarbonate, which exert increased demand of chemicals in conditioning, are removed in the process.

There are three methods of elutriation: single stage, multi-stage and counter-current washing, the water requirement being dependent upon the method used. For a given alkalinity reduction, single stage elutriation requires 2.5 times as much water as the two stage and 5 times as much water as counter-current washing. Hence, single stage washing is used only in small plants.

Counter-current washing, although higher in initial cost, is adopted in all large plants. Water requirement also depends on alkalinity of dilution water, alkalinity of sludge and desired alkalinity of elutriated sludge. Sludge and water are mixed in a chamber with mechanical mixing arrangement, the detention period being about 20 seconds. The sludge is then settled in settling tanks and excess water decanted. A maximum surface loading on settling tank of about $40m^3/m^3$/day and a detention period of about 4 hours are adopted.

Counter-current elutriation is generally carried out in twin tanks similar to sedimentation tanks, in which sludge and wash water enter at opposite ends. Piping and channels are so arranged that wash water entering the second stage tank comes first in contact with sludge already washed in the first stage tank. The volume of wash water required is roughly 2 to 3 times the volume of sludge elutriated.

The dosage of chemicals detention period and flow of conditioned sludge to mechanical dewatering units are automatically controlled by float switches, so that these variables are adjusted on the basis of performance and the quality of sludge cake coming out.

6.7.4.4 Heat Treatment

In this process, sludge is heated for short periods of time under pressure. Sludge is preheated in a heat exchanger before it enters a reactor vessel where steam is injected to bring the temperature to 145°C to 200°C at a pressure of 10 to 15kg/cm^2. After a 30-minute contact time, the sludge is discharged through the heat exchanger to a sludge separation tank. The sludge can be filtered through a vacuum filter to a solid content of 40% to 50% with filter yields of 100 kg/m^3/hr.

6.7.5 Screw Press

There are presently two major types of screw presses used in municipal dewatering applications: horizontal and inclined. Inclined screw presses are at angles 15 to 20 degrees from the horizontal. Other areas of difference pertain to sludge inlet configuration, screen basket design (wedge wire), basket cleaning from the inside and outside (brushes and rotating wash system) and filtrate water collection.

The major elements of a screw press dewatering system are the sludge feed pump, polymer makeup and feed system, polymer injection and mixing device (injection ring and mixing valve), flocculation vessel with mixer, sludge inlet headbox or pipe, screw driver mechanism, shafted screw enclosed within a screen, a rectangular or circular cross-section enclosure compartment, and an outlet for dewatered cake. Some horizontal screw press systems (e.g., the combined dewatering and pasteurization process) include a rotary screen thickener before the screw press, which may be desirable for reducing the hydraulic load to the screw press given certain feed sludge characteristics in conventional applications.

A screw press is a simple, slow moving device that achieves continuous dewatering. Polymer is combined with sludge in flocculation vessels upstream of the screw press to enhance the sludge's dewatering characteristics. Screw press dewater sludge first by gravity drainage at the inlet section of the screw and then by squeezing free water out of the sludge as they are conveyed to the discharge end of the screw under gradually increasing pressure and friction. The increased pressure to compress the sludge is generated by progressively reducing the available cross-sectional area for the sludge. The released water is allowed to escape through perforated screens surrounding the screw, while the sludge is retained inside the press. The liquid forced out through the screens is collected and conveyed from the press and the dewatered sludge is dropped through the screw's discharge outlet at the end of the press. The screw speed, configuration, screen size and orientation can be tailored for each application. The machine is shown in Figure 6.15 overleaf.

Advantages

1) Low rotational speed results in low maintenance and noise.
2) Low operating energy consumption
3) Containment of odours and aerosol, low building corrosion potential
4) Simple operation with low operator attention
5) Wash water demand and pressure requirements lower than belt presses.

Source: ISHIGAKI COMPANY, LTD

Figure 6.15 Screw press dewatering machine

Disadvantages

1) Cake concentration may be relatively low when there are no primary clarifiers.
2) Larger footprint
3) Only few manufacturers are available and equipment cannot be specified “as-equal.” It must be sole-sourced or pre-purchased.
4) Requires wash water
5) Lower solids capture than other dewatering processes in some cases.

6.7.6 Rotary Press

Rotary press dewatering technology relies on gravity, friction, and pressure differential to dewater sludge. Sludge is dosed with polymer and fed into a channel bound by screens on each side. The channel curves with the circumference of the unit, making a 180° turn from inlet to outlet. Free water passes through the screens, which move in continuous, slow, concentric motion. The motion of the screens creates a “gripping” effect toward the end of the channel, where cake accumulates against the outlet gate, and the motion of the screens squeezes out more water. The cake is continuously released in pressure-controlled outlet.

The major elements of a rotary press are the polymer feed and mixing system, parallel filtering screens, a circular channel between the screens, the rotation shaft and a pressure-controlled outlet. The screens consist of two layers of perforated stainless steel, with each layer having different sieve size. The rotary press drive configuration allows up to six rotary press channels to be operated on a single drive. Each channel has bearings and the combined unit has an outboard bearing cantilevered on one end. The rotary press dewatering machine is shown in Figure 6.16 overleaf.

Source: Sanki Engineering Co., Ltd.

Figure 6.16 Rotary press dewatering machine (1,200φ, 4 channels)

Advantages

1) Uses less energy than centrifuges or belt filter presses
2) Small footprint
3) Odours contained
4) Low shear
5) Minimal moving parts
6) Minimal building requirements
7) Minimal start-up and shutdown time
8) Uses less wash water than belt filter presses
9) Low vibration, low noise
10) Modular design

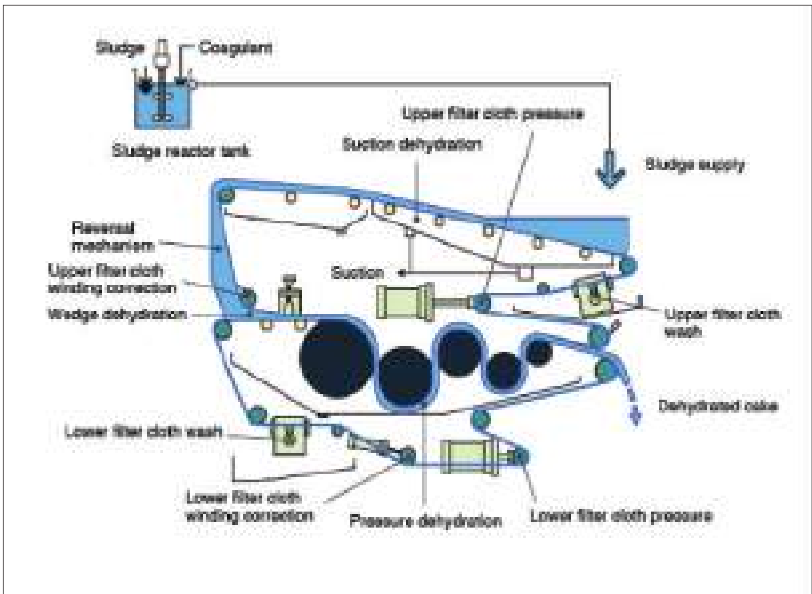
Disadvantages

1) May be more dependent on polymer performance than centrifuges or belt filter presses
2) Low throughput compared to other mechanical dewatering processes
3) Screen clogging potential
4) Need for heavy rated overhead crane to lift and maintain channels
5) High capital cost

6.7.7 Belt Press

Belt filter presses continuously dewater sludge using two or three moving belts and a series of rollers. The filter belt separates water from sludge via gravity drainage and compression. Sludge sandwiched between two tensioned porous belts is passed over and under rollers of various diameters. Increased pressure is created as the belt passes over rollers which decrease in diameter. Many designs of belt filtration processes are available, but all incorporate the following basic features: polymer conditioning zone, gravity drainage zones, low pressure squeezing zone and high pressure squeezing zones.

Each belt press manufacturer produces machines with slightly different mechanical features and operating characteristics. Presses are available in widths ranging from about 0.5 m to 3.5 m. Most belt filter press in municipal STP use belts of 1 m to 2 m width. The main components of a belt filter press include feed equipment and piping frame, belts, belt-tracking and tensioning systems, belt wash system, rollers and bearings, cake-discharge blades, chutes, cake conveyance, drive system, belt-speed control and chemical conditioning and flocculation as in Figure 6.17.



Source: SEKISUI AQUA SYSTEMS Co., Ltd.

Figure 6.17 Belt press dewatering machine

Personnel safety must be fully considered and incorporated into the design. The design must provide for and facilitate maintenance, provide safety stops, for instance with ultraviolet sensor and trip wires around the belt press and any cake conveyors, convenient and safe equipment access, drainage and spill containment, non-slip walkways and floors, sufficient lighting, noise reduction, ventilation and odour control. System interlocks should be provided to stop the solids and polymer feed pumps when the press is shut down.

Advantages

1) Staffing requirements are low, especially if the equipment processes the solids in one shift

2) Maintenance is relatively simple and can usually be completed by a sewage treatment plant maintenance crew.

3) Energy requirements are relatively low compared to some other types of dewatering equipment

4) Belt presses can be started and shut down quickly compared to centrifuges, which require up to an hour to build up speed

5) There is less noise associated with belt presses compared to centrifuges

Disadvantages

1) Because of the open nature of a belt press, there is a significant potential for odours and sprays. Workers in the belt press areas can be exposed to aerosols from the belt-wash spray nozzles, as well as pathogens and hazardous gases (e.g., hydrogen sulphide).

2) Belt presses require more operator attention if the feed sludge varies in the solids concentration or organic matter. This should not be a problem if the belt presses are fed from well-mixed digesters.

3) Sludge with higher concentrations of oil and grease can result in blinding the belt filter and lower solids content cake.

4) Sludge must be screened and/or ground to minimize the risk of sharp objects damaging the belt.

5) Belt washing at the end of each shift, or more frequently, can be time consuming and require large amounts of water. An automatic belt washing system and the use of effluent can minimize these costs.

6) Replacing the belt is the major maintenance cost

6.7.8 Filter Press

Pressure filtration uses a positive pressure differential to separate suspended solids from liquid slurry. Recessed-chamber filter presses are operated as a batch process. Solids pumped to the filter press under pressure ranging from 100 to 300 psi force the liquid through a filter medium, leaving a concentrated solids cake trapped between the filter cloths that cover the recessed plates. The filtrate drains into internal conduits and collects at the end of the press for discharge. Then the plates separate and the cake falls into a conveyor to the collection truck. Two types of filter presses typically are used to dewater sludge.

The most common is the fixed-volume, recessed-chamber filter press. The other is the variable volume, recessed-chamber filter press (also called the diaphragm filter press).

The main mechanical components of filter press include the structural frame, filter press plates, diaphragms,filter cloths and plate sifters. Various options are available for each component.

Advantages

1) The main advantage of a pressure filter press system is that it typically produces cakes that are drier than those produced by other dewatering equipments.

2) If the cake solids content are more than 35%, then filter presses can be a cost-effective dewatering option.

3) Filter press can adapt to a wide range of sludge characteristics.

4) Filter press produce a high-quality filtrate that lowers recycle stream treatment requirements.

5) Pressure filter press are cost-effective when the dewatered cake are incinerated.

Disadvantages

1) The main disadvantages of filter press are their high capital cost, relatively high O&M costs and substantial quantities of treatment chemicals.

2) The periodic adherence of cake to the filter medium must be manually removed. This problem may indicate the need to wash the filter media or increase conditioner dosages.

3) It requires significant amounts of energy to pressurize the units. Typical energy requirements are in order of 0.04 to 0.07 kWh per kilogram of dry sludge processed.

6.7.9 Centrifugal Dewatering Machine

The process of high speed centrifuging has been found useful to reduce the moisture in sludge to around 60 %. Usually the liquor from the centrifuge has higher solids content than filtrate from sand drying beds. Return of this liquor to the treatment plant may result in a larger recalculated load of these fine solids to the primary settling and sludge system and also in reduced effluent quality.

Advantages

1) Centrifuge may offer lower overall operation and maintenance costs and can outperform conventional belt filter presses.

2) Centrifuge require a small amount of floor space relative to their capacity.

3) Centrifuge require minimal operator attention when operations are stable.

4) Low exposure to pathogens, aerosols, hydrogen sulphide or other odours.

5) Centrifuge are easy to clean.

6) Centrifuge can handle higher than design loadings and the percent solids recovery can usually be maintained with the addition of a higher polymer dosage.

7) Major maintenance items can be easily removed and replaced. Repair work is usually performed by the manufacturer.

Disadvantages

1) Centrifuge have high power consumption and are fairly noisy.
2) Experience of operating the equipment is required to optimize performance.
3) Performance is difficult to monitor because the operator's view of centrate and feed is obstructed.
4) Special structural considerations must be taken into account. As with any piece of high speed rotary equipment, the base must be stationary and level due to dynamic loading.
5) Spare parts are expensive and internal parts are subject to abrasive wear.
6) Start-up and shut-down may take an hour to gradually bring the centrifuge up to speed and slow it down for clean out prior to shut-down.

6.7.10 Vacuum Filter

The vacuum filter consists of a cylindrical drum over which a filtering medium of wool, cloth or felt, synthetic fibre or plastic, or stainless steel mesh or coil springs is fixed. The drum is suspended horizontally so that one quarter of its diameter is submerged in a tank containing sludge.

Valves and piping are arranged to apply a vacuum on the inner side of the filter medium as the drum rotates slowly in the sludge. The vacuum holds the sludge against the drum as it continues to be applied as the drum rotates out of the sludge tank. This pulls water away from the sludge leaving a moist cake mat on the outer surface.

The sludge cake on the filter medium is scraped from the drum, just before it enters the sludge tank again. Vacuum pumps, moisture traps, filtrate pumps, filtrate receivers, conveyors and pipes and valves are necessary adjuncts to the filter.

Operating costs of vacuum filters are usually higher than for sludge drying beds. However, they require less area since dewatering is rapid. The operation is independent of weather conditions and it can be used for dewatering raw or partially digested sludge requiring drying or incineration.

The capacity of the filter varies with the type of sludge being filtered and in calculating the size of filter the desired moisture content of the filter cake is a factor.

If wet cake is acceptable, higher filtration rates and lower coagulant dosage can be used. The filtration rate is expressed in kg of dry solids per square meter of medium per hour. It varies from 10 kg/m^2/hr for activated sludge alone to 50 kg/m^2/hr for primary sludge.

A design rate of 15 kg/m^2/hr is a conservative figure that can be used when the quality of the sludge and the type of the filter to be used are not known.

Filter drums are rotated at a speed of 7 rpm to 40 rpm with a vacuum range of 500 mm to 650 mm of mercury. The filter run does not exceed 30 hours per week in small plants to allow time for conditioning, clean up and delays. At larger plants, it may work for 20 hours a day. The moisture of the filtered cake varies normally from 80% in case of raw activated sludge to 70% for digested primary sludge.

Filters should be operated to produce a cake of 60% to 70% moisture if it is to be heated, dried or incinerated. At the end of each filter run, the filter fabric is cleaned to remove sticking sludge.

A high pressure stream of water is used to clean the filter cloth. The filters are usually located in a separate room or building with adequate light and ventilation.

Advantages

1) Operation is easy to understand because formation and discharge of sludge cake are easily visible.
2) Does not require highly-skilled operator
3) Will continue to operate even if the chemical conditioning dosage is not optimized, although this may cause discharge problems
4) Has low maintenance requirement for a continuously operating piece of equipment, except in certain cases with lime conditioning

Disadvantages

1) Consumes a large amount of energy per unit of sludge dewatered
2) Vacuum pumps are noisy.
3) Lime and Ferric chloride conditioning can cause considerable maintenance cleaning problems.
4) The use of lime for conditioning can produce strong ammonia odours with digested sludge.
5) Best performance is usually achieved at feed solids of 3 to 4%. However, some well-conditioned sludge are filtered successfully at concentrations of <2%.
6) Ferric chloride and lime conditioning costs are higher than polymer conditioning costs. Polymer conditioning is not always effective on vacuum filters.

6.7.11 Comparison of Dewatering Systems

The advantages and disadvantages of dewatering systems are compared in Table 6.13 overleaf.

6.8 SLUDGE DISINFECTION

6.8.1 Heat Drying

The purpose of heat drying is to reduce the moisture content and volume of dewatered sludge, so that it can be used after drying without causing offensive odours or risk to public health.

Several methods such as sludge drying under controlled heat, flash drying, rotary kiln, multiple hearth furnaces, etc., have been used in combination with incineration devices. Drying is brought about by directing a stream of heated air or other gases at about 350°C.

Table 6.13 Comparison of advantages and disadvantages of dewatering systems

	Advantages	Disadvantages
Screw Press	1)Low rotational speed results in low maintenance and noise. 2)Low operating energy consumption 3)Containment of odours and aerosol, 4)Low building corrosion potential 5)Simple operation with low operator attention 6)Lower wash water and pressure than belt presses.	1) Cake concentration may be relatively low for primary sludges. 2) Larger footprint 3) Few indigenous manufacturers available as of this time 4) Requires wash water 5) Lower solids capture than other processes in some cases. 6) More dependency on supplier for repairs
Rotary Press	1)Uses less energy than centrifuges or belt filter presses 2)Small footprint, minimum building size, Odours contained, 3)Minimal moving parts, less vibration 4)Minimal start-up and shutdown time 5)Uses less wash water than belt filter presses	1) More dependent on polymers than centrifuges / belt filter presses 2) Low throughput than other mechanical dewatering processes 3) Screen clogging potential 4) Heavy rated overhead crane to lift and maintain channels 5) Higher building requirements in structurals
Belt Press	1) Staffing requirements are less. 2) Maintenance is relatively simple. Belt change is the major cost. 3) Energy is relatively low than some other dewatering equipment. 4) Can be started and shut down quickly compared to centrifuges. 5) Less noise compared to centrifuges. 6) The rotary speed is lesser as compared to centrifuges.	1) A significant potential for odours and sprays 2) More operator attention when feed sludge contents varies 3) High oil & grease causes blinding the belt and lesser cake solids 4) Sludge to be screened / ground as sharp objects damage to belt 5) Belt washing time consuming and requires more wash water. 6) Roller alignments requires skilled setting in repairs

	Advantages	Disadvantages
Filter Press	1)Produces drier cakes than other dewatering equipment 2)For solids of more than 35 % in cake, they can be cost-effective 3)Can adapt to a wide range of sludge characteristics 4)Produces a high-quality filtrate. Lowers recycle stream treatment 5)Cost-effective when the dewatered cake must be incinerated.	1) Relatively high O&M costs. 2) Treatment chemicals required 3) Needs periodic manual removal of adherence of cake to the filter 4) Increased washing of filter media or higher conditioner dosages. 5) Requires significant amounts of energy to pressurize the units.
Centrifuges	1)Lower O&M costs and can outperform conventional belt presses 2)Require a small amount of floor space relative to their capacity 3)Require minimal operator attention when operations are stable 4)Low exposure to pathogens, aerosols and odours for operators. 5)Relatively easier to clean in situ 6)Can handle higher than design loadings 7)Solids recovery can be sustained by higher polymer dosage 8)Major maintenance items can be easily removed and replaced.	1) High power consumption and fairly noisy 2) Experience of operating is required to optimize performance. 3) Difficult to monitor as view of centrate and feed is obstructed. 4) The base must be stationary and level due to dynamic loading. 5) Spare parts are supplier oriented 6) Internal parts are subject to abrasive wear. 7) Start-up and shut down may take an hour. 8) Heavy dependence on supplier for repairs
Vacuum Filter	1) Formation and discharge of sludge cake is easily visible. 2) Operation is easily understood visually 3) Does not require highly-skilled operator attention 4) Will operate even if the chemical dosage is not optimized, 5) Has a low maintenance requirement for a continuously operation 6) Certain cases with lime conditioning may cause O&M issues.	1) Higher energy per unit of sludge Vacuum pumps are noisy. 2) Lime / Ferric chloride conditioning causes cleaning problems. 3) Lime use can produce strong ammonia smell in digested sludge. 4) Polymer conditioning is not always effective on vacuum filters. 5) Heavy dependence on supplier for repairs to filter medium 6) Alignment of filter medium is crucial after repairs.

The hot gases, dust and ash released during combustion are to be removed by suitable control mechanisms to minimize air pollution. The dried sludge removed from the kilns is granular and clinker-like, which may be pulverized before use as soil conditioner.

6.8.2 Incineration

The purpose of incineration is to destroy the organic material, the residual ash being generally used as landfill. During the process all the gases released from the sludge are burnt off and all the organisms are destroyed. Dewatered or digested sludge is subjected to temperatures between 650°C to 750°C. Cyclone or multiple hearth and flash type furnaces are used with proper heating arrangements with temperature control and drying mechanisms. Dust, fly ash and soot are collected for use as landfill.

It has the advantages of freedom from odours and a great reduction in volume and weight of materials to be disposed of finally, but the process requires high capital and recurring costs, installation of machinery and skilled operation. Controlled drying and partial incineration have also been employed for dewatering of sludge, before being put on drying beds.

6.9 SLUDGE COMPOSTING

6.9.1 Outline

Sludge composting is a method, in which microorganisms decompose the degradable organic matter in sludge under the aerobic condition and create stable material that is easy to handle, store and use for farmland. Sludge compost is humus-like material without detectable levels of pathogens that can be applied as a soil conditioner and fertilizer to gardens, food and feed crops and farmland.

Sludge compost provides large quantities of organic matter and nutrients such as nitrogen and potassium to the soil. It improves the soil texture, elevates soil cation exchange capacity (an indication of the soil's ability to hold nutrients), all characteristics of a good organic fertilizer. Sludge compost is safe to use and generally has a high degree of acceptability by the public.

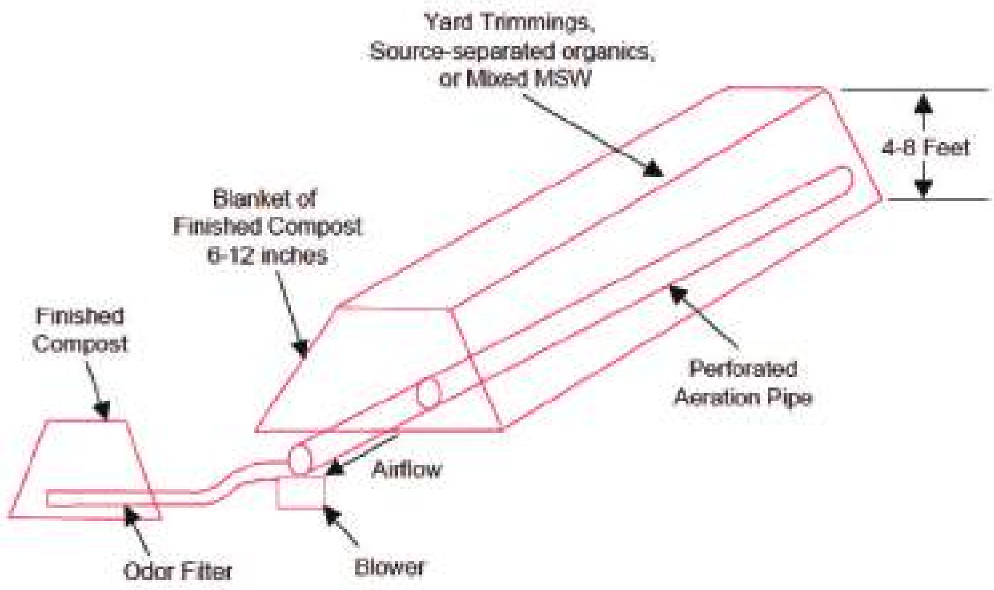
Sludge composting involve mixing dewatered sludge with a bulking agent to provide carbon and increase porosity. The resulting mixture is piled or placed in a vessel where microbial activity causes the temperature of the mixture to rise during the first phase active composting period. The specific temperatures that must be achieved and maintained for successful composting vary based on the method and use of the end product. After the first phase, active composting period of the material is cured and in the second phase, it becomes compost and is distributed.

6.9.2 Types of Composting Methods

Sludge composting methods are divided into, aerated static pile, windrow and in-vessel.

a. Aerated static pile

Figure 6.18 (overleaf) shows an aerated static pile composting method.



Source:Hickman Jr., H.Lanier, 1999

Figure 6.18 Configuration of an aerated static pile composting method

Dewatered sludge cake is mechanically mixed with a bulking agent and stacked into long piles over a bed of pipes, through which air is transferred to the composting material. After the first phase composting i.e., active composting, as the pile is starting to cool down, the material is moved into the second phase composting i.e., a curing pile. The bulking agent is often reused in this composting method and may be screened before or after curing so that it can be reused.

b. Windrow

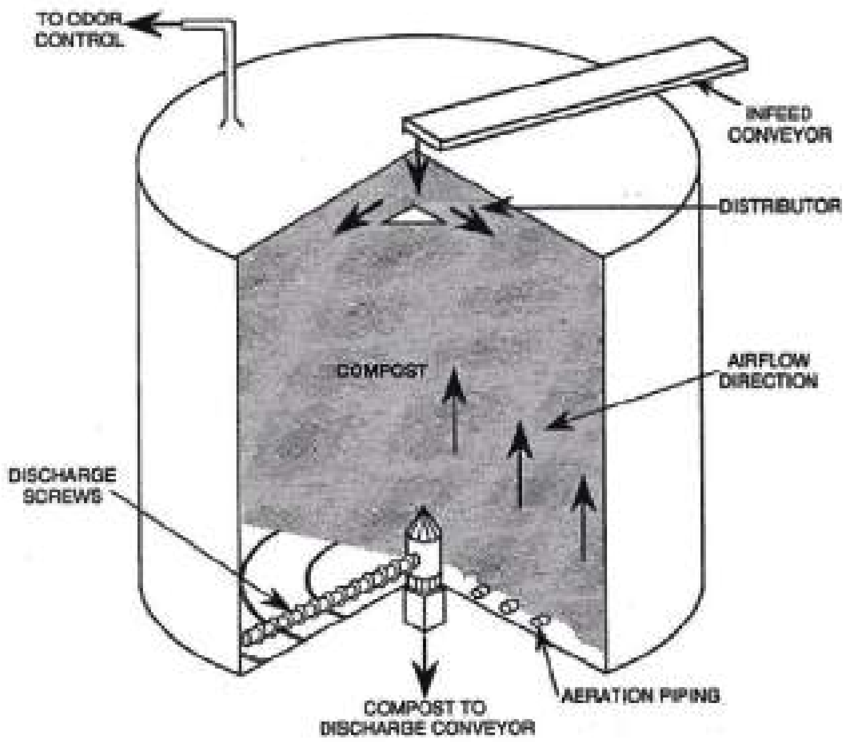
Dewatered sludge cake is mixed with bulking agent and piled in long rows because there is no piping to supply air to the piles; they are mechanically turned to increase the amount of oxygen. This periodic mixing is essential to move outer surfaces of material inward so they are subjected to the higher temperatures deeper in the pile. A number of turning devices are available. As with aerated static pile composting, the material is moved into the second phase composting i.e., curing piles after the first phase composting, it is active composting. Several rows may be placed into a larger pile for curing

c. In-Vessel

There are two types of In-Vessel composting reactor, a vertical type and a horizontal type.

Figure 6-19 (overleaf) shows an example of vertical type In-Vessel composting reactor.

A mixture of dewatered sludge cake and bulking agent is fed into a silo, tunnel, channel, or vessel. Augers, conveyors, rams, or other devices are used to aerate, mix and move the product through the vessel to the discharge point. Air is generally blown into the mixture. After the first phase composting i.e., active composting, the finished product is usually stored in a pile for the second phase composting i.e., curing prior to distribution.



Source: USEPA, 1989

Figure 6.19 In-Vessel composting reactor - Taulman composting system

All the three composting methods require the use of bulking agents; wood chips, saw dust, and shredded tires are commonly used, but many other materials are suitable.

6.9.3 Applicability

The physical characteristics of most sewage sludge allow for their successful composting. However, many characteristics (including moisture content, volatile solids content, carbon content, nitrogen content, and bulk density) will impact design decisions for the composting method. Both digested and raw solids can be composted, but some degree of digestion (or similar stabilization) is desirable to reduce the potential for generation of foul odours from the composting operation. This is particularly important for aerated static pile and windrow operations.

Carbon and nitrogen content of the sewage sludge must be balanced against that of the bulking agent to achieve a suitable carbon to nitrogen ratio of between 25 and 35 parts carbon to one part nitrogen.

Site characteristics make composting more suitable for some sewage treatment plants than others. An adequate buffer zone from neighbouring residents is desirable to reduce the potential for nuisance complaints. In urban and suburban settings, in-vessel technology may be more suitable than other composting technologies because the in-vessel method allows for containment and treatment of air to remove odours before release.

The requirement for a relatively small amount of land also increases the applicability of in-vessel composting in these settings.

6.9.4 Advantages and Disadvantages

The advantages and disadvantages of sludge composting are as follows:

a. Advantages

- Sludge composting reduces landfill space of sludge by utilizing sewage compost as a soil conditioner or fertilizer.
- Sludge compost has market value and revenue is expected by selling it.
- Sludge compost is easy to store, handle and use.
- Addition of sludge compost to soil increases the soil's phosphorus, potassium, nitrogen and organic carbon content.

b. Disadvantages

- Odour is produced at the composting site.
- Pathogens possibly survive and exist in sludge compost.
- Sludge compost lacks consistency in product quality with reference to metals, stability, and maturity.

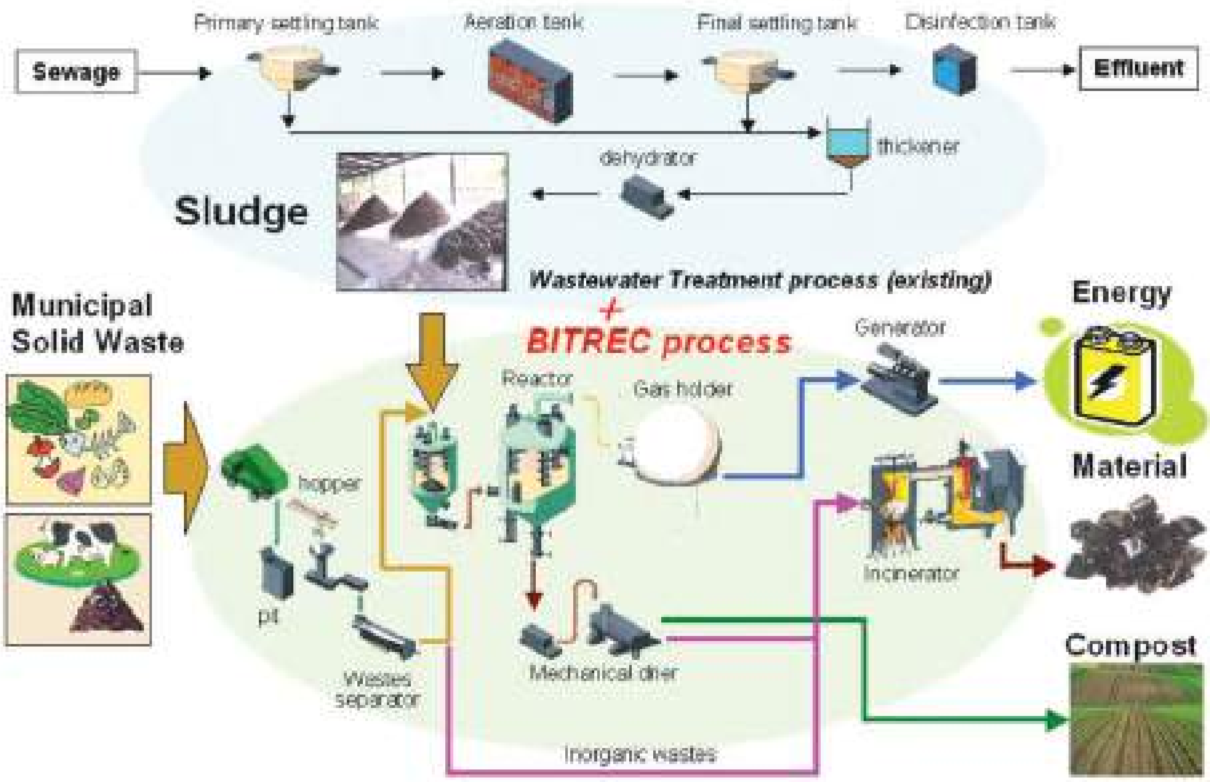
6.9.5 Design Considerations

The important matters related to design of sludge composting system are as follows:

- Sewage sludge characteristics such as moisture content, volatile solids content, carbon content, nitrogen content, and bulk density are to be considered when designing a composting system.
- It is essential for sludge composting to maintain uniform aerobic conditions during composting. Proper air supply procedures are required such as turning of piles or adequate aeration and selection of proper bulking agent is also necessary.
- Compaction of the composting mass should be avoided to maintain sufficient pore space for aeration.
- Bulking agent appropriate for sludge characteristics should be selected considering its characteristics such as size, cost/availability, recoverability, carbon availability, pre-processing requirements, porosity and moisture content.
- Metal content of the sewage sludge should be considered in the design to ensure a market for the final product.
- An odour control system should be considered especially for an in-vessel composting system.
- Composting detention time and temperature are determined considering the quality of product compost. For an aerated static pile or an in-vessel system, it should be kept at 55°C for at least 3 days and for windrow, 55°C for at least 15 days with 5 turns.

6.9.6 Mixed Composting of Sewage Sludge and Municipal Solid Waste

Dewatered sludge could be mixed with grounded organic municipal solid waste and could be used as a good soil conditioner (compost). However, this process needs proper policy guidelines, stringent regulations, standards and above all community awareness. Figure 6.20 shows a flow chart example of mixed composting of sewage and municipal solid waste.



Source: Ebara Engineering Service Co., Ltd. (there are many others also)

Figure 6.20 Mixed composting of sewage and municipal solid waste

6.10 SLUDGE DISPOSAL

6.10.1 Sludge Storage

6.10.1.1 General

Sludge storage facilities shall be provided at all mechanized STP. Appropriate storage facilities may consist of any combination of drying beds, separate tanks, additional volume in sludge stabilization units, pad areas or other means to store either liquid or dried sludge. The design shall provide for odour control in sludge storage tanks and sludge storage yard, covering, or other appropriate means.

6.10.1.2 Volume

Rational calculations justifying the number of days of storage to be provided shall be submitted and shall be based on the total sludge handling and disposal system.

Refer to Section 6.4 and 6.5 for anaerobic and aerobic digested sludge production values. Sludge production values for other stabilization processes should be justified on the basis of design. If the land application method of sludge disposal is the only means of disposal utilized at a STP, storage shall be provided based on considerations including at least the following items:

1) Inclement weather effects on access to the application land
2) Temperatures including frozen ground and stored sludge cake conditions
3) Haul road restrictions including spring thawing conditions
4) Area wise seasonal rainfall patterns
5) Cropping practices on available land
6) Potential for increased sludge volumes from industrial sources
7) Available area for expanding sludge storage
8) Appropriate pathogen reduction and vector attraction reduction requirements.

A minimum range of 120 to 180 days storage should be provided for the design life of the plant unless a different period is approved by the reviewing authority.

6.10.2 Disposal

Sludge is usually disposed of on land as manure to soil, or as a soil conditioner, or barged into sea. Burial is generally resorted to for small quantities of putrid sludge. The most common method is to utilize it as a fertilizer. Ash from incinerated sludge is used as a landfill. In some cases, wet sludge, raw or digested, as well as supernatant from digester can be constructed as lagoons as a temporary measure, but such practice may create problems like odour nuisance, groundwater pollution and other public health hazards. Wet or digested sludge can be used as sanitary landfill or for mechanized composting with city refuse. Disposal of sludge shall have to be as per the hazardous waste (handling and management) rules of MoEF if Table 6.14 and faecal coliform limits are violated.

6.10.2.1 Sludge as Soil Filler

The use of raw sludge as a soil filler directly on land for raising crops as a means of disposal is not desirable since it is fraught with health hazards. Application of sewage sludge to soils should take into consideration the following guiding principles:

1. Sludge from open air drying beds should not be used on soils where it is likely to come into direct contact with the vegetables and fruits.

2) Sludge from drying beds should be ploughed into the soil before raising crops. Top dressing of soil with sludge should be prohibited.

3) Dried sludge may be used for lawns and for growing deep rooted cash crops and fodder grass where direct contact with edible part is minimized.

4) Heat dried sludge is the safest from public health point of view. Though deficient in humus, it is convenient in handling and distribution. It should be used along with farmyard manure.

5) Liquid sludge either raw or digested is unsafe to use. It is unsatisfactory as fertilizer or soil conditions. If used, it must be thoroughly incorporated into the soil and land should be given rest, so that biological transformation of organic material takes place. It should be used in such a way as to avoid all possible direct human contact.

6.10.2.1.1 Heavy metal and Faecal coliform ceiling concentration in treated sewage sludge

The ceiling concentration of the heavy metals in the treated sewage sludge should not exceed the values mentioned in Table 6.14.

Table 6.14 Ceiling concentration of heavy metals in treated sewage sludge for use in agriculture

No.	Chemical	Ceiling concentration (A)	No.	Chemical	Ceiling concentration (A)
1	Arsenic	75	6	Chromium	500(total)
2	Cadmium	85	7	Selenium	100
3	Copper	4300	8	Zinc	7500
4	Mercury	57	9	Molybdenum	75
5	Nickel	420	10	Lead	840

(A)-Expressed as mg/kg on dry weight basis

If the concentrations in the treated sewage sludge exceed the values as in Table 6.14 land application shall not be permitted and the sludge shall have to be disposed or contained as per the hazardous waste (handling and management) rules of MoEF. In addition to the above heavy metal limits, the faecal coliform limit for sewage sludge shall be as under.

At least seven sewage sludge samples should be collected at the time of use or disposed and analyzed for faecal coliforms during each monitoring period. The geometric mean of the densities of these samples will be calculated and should meet the following criteria.

Less than 20,00,000 most probable number per gram of total dry solids (20,00,000 MPN / gTS)

Or

Less than 20,00,000 colony forming units per gram of total dry solids (20,00,000 CFU / gTS)

In general, digested sludge is indelicate, but definite value as a source of slowly available nitrogen and some phosphate.

It is comparable to farmyard manure except for its deficiency in potash.

It also contains essential elements to plant life and minor nutrients, in the form of trace metals.

The sludge humus also increases the water holding capacity of the soil and reduces soil erosion making it an excellent soil conditioner especially in arid regions by making available needed humus content which results in greater fertility.

6.10.2.2 Sludge Storage Yard

Dewatered cake typically is stored before additional treatment (e.g., heat drying) or being hauled off-site for use or disposal. Most flammable liquids would have been removed during dewatering and methane-generating microorganisms do not thrive in dry aerobic environments.

The amount of storage needed depends on the end use of the sludge cake. Often, biosolids will only be held for a few days or weeks before being treated further or hauled off-site. In this case, they are typically stored in large roll-off containers, 18 wheel dump trailers, concrete bunkers with push walls, or bins with augurs. However, if the biosolids will be applied on land or surface disposed, long-term storage may be required. In these cases, they often are stockpiled on concrete slabs or other impervious pads. When designing long-term storage facilities, it is needed to consider buffering, odour control, and accessibility. It is also needed to determine whether the storage facility should be open or covered.

Dried solids typically are stored either on-site or at a land-application site before disposal or beneficial use. They may be stored in stockpiles or silos. Because dried solids contain a significant amount of combustible organic material that can be released as dust, temperature control is important. If silos are used, it should be designed to promote cooling and maximize heat dissipation. Therefore, tall, narrow silos are better than wide ones. Narrow silos also make fires easier to control. However, if the silo is too narrow, it will make relief venting problematic. If multiple silos are used, there should be procedures to ensure that they are emptied cyclically to avoid exceeding safe residence times. Also, it is needed to consider the stored product’s thermal stability in case a prolonged plant shutdown or if silo blockage occurs.

6.10.2.3 Sanitary Land Fill

When organic solids are placed in a landfill, decomposition may result in odour if sufficient cover is not available. Besides surface water contamination and leaching of sludge components to the groundwater must be considered. Decomposition may result in soil settlement resulting in surface water pond above the fill. Typical depths of soil cover over the fill area are 0.2m after each daily deposit and 0.6m over an area that has been filled completely.

Surface topography should be finished to allow rainfall to drain away and not allow it to infiltrate into the solid landfill. Land fill leachate requires long term monitoring and should satisfy the relavent water pollution control standards for land applications. Vegetation must be established quickly on completed areas to provide for erosion control. It is general practice not to crop the landfill area for a number of years after completion.

Land fills are not usually recommended for disposal of STP sludge. In case they are adopted, the above points should be considered.

6.10.2.4 Disposal in Water or Sea

This is not a common method of disposal because it is contingent on the availability of a large body of water adequate to permit dilution at some sea coast sites. The sludge, either raw or digested, may be barged to sea far enough to make available the required dilution and dispersion. The method requires careful consideration of all factors including flora and fauna for proper design and siting of outfall to prevent any coastal pollution or interference with navigation.

6.11 CORROSION PREVENTION AND CONTROL

6.11.1 Anaerobic Sludge Digesters

In sludge digestion tank, digestion of sludge is carried out under anaerobic conditions for a long period. During the normal functioning of the digester, and more so, during faulty operations, various acids are produced for a temporary period. The waste may contain appreciable quantity of sulphates due to seepage of sea water in coastal regions or due to industrial wastes. Under anaerobic conditions in digester the sulphate will be converted to hydrogen sulphide. The corrosion due to hydrogen sulphide is in fact due to sulphuric acid formed in the presence of moisture. This will attack the digester walls and also the mechanical equipment to such an extent that breakdown may occur ultimately. Cement that is resistant against H_2S, such as blast furnace slag cement, should be used in the construction of digesters.

It is observed that the draft tubes inside the digester are sometimes provided of mild steel. This is not a good practice, since the life of such metallic tubes in the highly corrosive interior will be very limited. Hume or concrete pipes of thicker cross section are therefore, recommended for use as draft tubes. Use of guy ropes inside the digesters should also be discouraged. Screw pumps are provided in the digester for proper circulation of the tank contents. The blades of this screw pump should be of corrosion resistant materials. In many installations the sludge gas is collected and burnt or utilized for other purposes. If the gas contains H_2S, this will be very corrosive under moist conditions to the gas engines, gas meters and all the equipment and piping. It is therefore, necessary to remove H_2S by scrubbing in such cases.

6.11.2 Sludge Pumps

For pumps and pumping equipment, proper material selection is of paramount importance. The pump casing is normally of close grained cast iron capable of resisting erosion on account of abrasive material in the waste.

For handling corrosive sludge, the impeller is generally made of high grade phosphor bronze or equivalent materials. The wearing rings for impeller should be of good corrosion resistant material such as bronze. The shafts are normally made of high tensile steel and replaceable shaft sleeves are recommended.

For pumps and pumping equipments, painting is the usual protective measure. Both the interior and exterior surfaces of pumps should be painted after rust scale and deposits are removed by sand blasting, wire brushing or rubbing with sand paper.

6.11.3 Piping Requirements in Treatment Plants

Piping in STP would be required in sewage and sludge conduits, drains and water lines to chemical process piping. Materials for various pipe line applications are mentioned in Table 5-27 of chapter 5 in Part A Manual.

6.11.4 Modification of Materials

Normally, the materials that are most suitable under the circumstances likely to be encountered should be used to be commensurate with economy. If justified economically, corrosion resistant construction material can be used initially. This may not require any additional protective coating frequently. Stainless steel, aluminium and plastics are examples of materials of this nature. It is possible that the use of such corrosion-resistant materials would be cost-effective in the long run. However, in STP, it is found that it is usually less expensive to use ordinary structural steel to which protective coatings are applied.

6.12 UPGRADING AND RETROFITTING OF SLUDGE FACILITIES

Over a passage of time, systems need to be upgraded and retrofitted. Keeping this in view, plants which have (a) SDB can implement mechanical dewatering methods, (b) gravity thickening can implement mechanical thickening. The area occupied by the SDB are needed for reconstruction and rehabilitation of sludge facilities and hence, in places of land shortage, SDB can be demolished and mechanical dewatering be adopted in those areas. However, as a matter of providing standby SDB for about 15 % requirement should be provided so that they can be used in case of any accidental breakdown of mechanical sludge dewatering equipment, but the width of the SDB shall be restricted to 3 m to enable its covering if required during times such as monsoons, etc.

The reconstruction of the sludge treatment facility should take each of following into consideration.

1) Study of basic policy

   When reconstructing a sludge treatment facility, after studying the basic policy synthetically from the following viewpoint, it is necessary to determine details such as (a) Independent treatment or intensive treatment, (b) Simple reconstruction or improvement in functional use (addition, change, etc. of sludge treatment process), (c) Package of handling process, or reconstruction in each process and series unit, (d) Treatment of return flow from sludge treatment facility, (e) The future amount of sludge treatment, (f) The effective use (final disposal) method, (g) Energy saving, (h) Degree of aging, (i) Earthquake-proof, (j) Degradation of function, (k) Ease with maintenance and (l) Environment

2) Confirmation of space for reconstruction

   In cases where space in the existing building and at site is adequate for the needs without being able to remove existing facilities, it can be constructed in the space. In cases where there is no space, removal and reconstruction is done for every series and process, or construction in another land is considered.

3) Intensity confirmation of existing building

In cases where it uses existing building, after taking the load of apparatus into consideration, intensity calculation of building is done. In cases where it extends [altering building and], seismic capacity evaluation and earthquake-proof construction may be needed.

4) Reconstruction procedure

In the following case, reconstruction procedures differ.

a) Independent treatment or intensive treatment

b) The whole sludge treatment process reconstruction, or each process and series unit

During construction period, reduction of the amount of sludge treatment and taking out the sludge to other facilities may be needed.

After considering the safety and economical efficiency, the reconstruction procedure is studied so that construction period can be shortened as much as possible.

5) The necessity for temporary facilities

In cases where it cannot perform reduction of the amount of sludge treatment, and another treatment, or in cases where safety, economical efficiency, and O & M are advantageous, sludge treatment by temporary facilities is carried out.

The necessity for temporary facilities of each process is based on the following as reference.

a) Thickening

Is taking out of sludge possible by vacuum vehicle etc. to other facilities or not?

b) Digestion

Is taking out of sludge possible to other facilities or not?

c) Dewatering

In cases where reconstruction is in a small scale and for a short period of time, can mobile dewatering system be used or not?

In addition, to installing temporary facilities, it needs careful adjustment of the amount of sludge treatment, operation stop time at the time of change, installation period of adjustment of facilities, etc.

6.12.1 Energy Saving Measures

The following is related to energy-saving technologies in sludge treatment process.

According to reconstruction stage, energy-saving equipment is introduced intentionally, and also it is desirable to save the energy of the whole system.

1) Thickening

a) Improvement in thickening performance
b) Improvement in solid recovery rate
c) Reduction of machine thickening power

2) Digestion tank

a) Management of injection sludge concentration to digestion tank
b) Temperature management of digestion tank
c Strengthening of keeping warm of digestion tank
d) Low motorization of stirrer of digestion tank
e) Strengthening of heat insulation to steam piping (warming facilities)
f) Automatic control of boiler for warming and automatic control of hot water heater

3) Dewatering

a) Management of supply sludge concentration
b) Reduction in moisture content of dehydration sludge
c) Control of series of dehydrator also including conveyance facilities
d) Reduction of mechanical dehydration power

6.13 BIOMETHANATION AND ENERGY RECOVERY (CARBON CREDIT)

6.13.1 General

Sewage sludge includes organic matter made of carbon, hydrogen, sulphur, and so on, and is a potential energy source of high value. Energy utilization methods include the method of recovering digester gas as energy and using it as heating fuel or in power generation, and the method of using it as fuel after drying. The water content in sludge has a major effect on energy consumption and energy recovery aspects with regard to utilization of sludge as energy. Therefore, water content should be reduced as far as possible starting from the sludge thickening stage to the dewatering and incineration stages. It is important to improve the overall energy utilization rate. Overall energy utilization of sludge has just made a beginning. Henceforth, the energy self-sufficiency of STP will need to be enhanced and stability in operation of STPs will need to be ensured over the long term. Moreover, these energy uses need to be promoted so as to contribute to the protection of the global environment. Utilization of digester gas, dried sludge and carbonized sludge is described hereafter.

6.13.2 Digester Gas Utilization

The utilization of digester gas from an operating STP at Nesapakkam in Chennai is presented in Section 5.16.1.2, Table 5-28. Generally, digester gas is used as fuel in boilers for heating sludge digestion tanks; surplus gas is incinerated in biogas combustion units and discharged to the atmosphere. When surplus gas exists in large amounts, the energy possessed by the gas can be effectively utilized and energy savings can be achieved in the entire system.

When considering the effective utilization of this unused energy, if there is a demand for heat for direct use such as auxiliary fuel for incinerating sludge, fuel for boiler (for hot water supply, for cooling and heating), a simple and highly efficient system is preferable. In recent years, power generating equipments using heat engines including gas engines have been on the rise. Moreover, there are practical instances of heat recovered from exhaust gas as well as cooling water in addition to power generation from gas engines. However, as the system becomes complex, studies from the viewpoint of construction cost and O & M are necessary. When digester gas obtained from anaerobic fermentation of sewage sludge is to be used for power generation, siloxane included in minute quantities in the digester gas sticks to the internal surface of cylinders and gas engine plugs causing accidents due to misfiring and abnormal ignition; therefore, measures against siloxane need to be adopted. Silicone, which is the source from which siloxane is generated, is included in major proportion in shampoo and rinse, used in the bathroom.

For this reason, siloxane have often been found in sewage in recent years. Siloxane gets volatilized at room temperature. If the temperature is higher, larger amount of siloxane gets volatilized. A large amount of siloxane moves to the digester gas in the digestion tank during heating, and its concentration becomes 10-100 mg/Nm^3 approximately. Research on this feature has been carried out in recent years. It has been verified that the major part of siloxane can be removed by activated carbon adsorption and high pressure water absorption.

Other policies for effective energy utilization are fuel cells, microgas turbine power generation, and automotive fuel gas applications, which have been practically realized. The features of fuel cell are its high efficiency and no rotating parts. Thus, there is no noise, O & M are easy and exhaust is also clean. Although it is necessary to study the effects of construction cost, life of fuel cells, sulphur included in digester gas, effective utilization of digester gas may be anticipated in the future. Figure 6.21 is an example of flow of power generated from digester gas (gas engine).

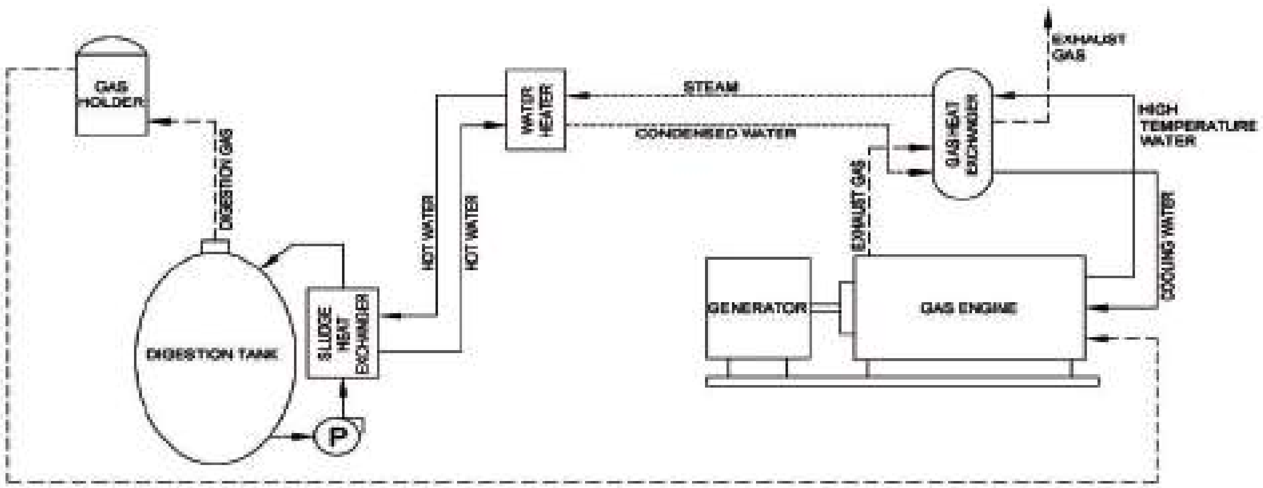


Figure 6.21 Example of flow of power generated from digester gas

It is important to study thoroughly the construction cost, quantity of digester gas generated outside of O & M cost, quantity of gas required for heating digester tank, quantity of surplus gas, and the temporal and seasonal variations of these and to confirm whether operation can be carried out at a high operating rate and at high efficiency.

6.13.3 Utilization of Dried Sludge

The utilization of dried sludge is mentioned in Section 6.14.2.

6.14 RECENT TECHNOLOGIES IN SLUDGE TREATMENT

6.14.1 Necessity of Sludge Treatment

With the progress of sewerage systems in urban areas, the amount of sewage volume increases and the amount of sludge generated during sewage treatment also increases naturally. Sewage treatment necessarily generates sludge and the bottom line of sewage treatment is effective, stable and lasting sludge treatment.

At present, in India, sludge is treated mainly by drying in sludge drying beds and dried sludge is utilized as soil filler. However, the increase in sludge volume, progress in urbanization and rise in the environmental awareness of people, etc., call for the adoption of new technologies such as mechanical dewatering, incineration, melting, etc., which are technologies for treating sludge more efficiently and reducing the sludge volume. The sludge volume is reduced gradually in each step of sludge treatment process. The sludge volume reductions in case of each step are roughly estimated as under in Figure 6.22.

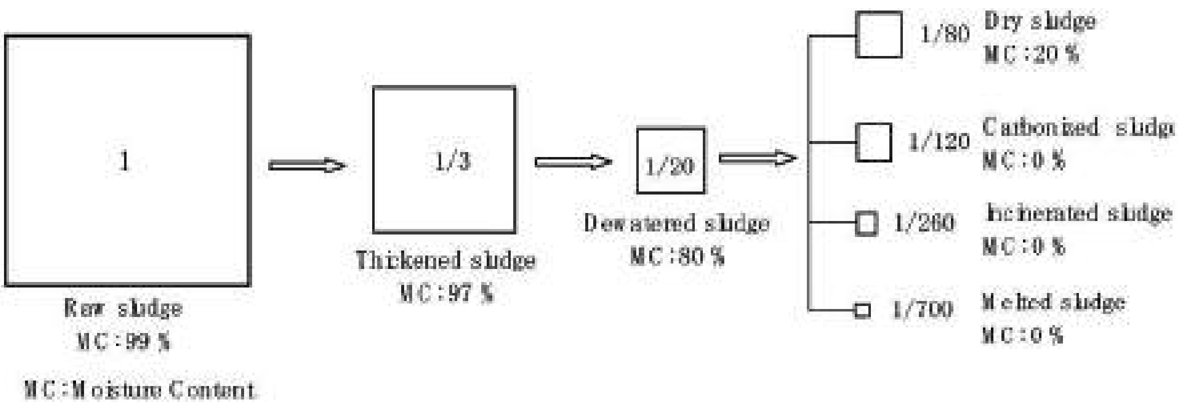


Figure 6.22 Sludge volume reduction of each step of sludge treatment process

On the other hand, sewage sludge is a useful resource, which consists of organic matter that can be used as fertilizer and inorganic substances like a soil-sludge that is used as immobilized bricks. Dry solids of sewage sludge also have calorific value near to coal. Therefore, it is desirable to use the sludge effectively as green farmland soil filler, construction materials, energy sources, etc. Proper utilization of sludge enables reduction of the quantity of sludge to be disposed off. Furthermore, adoption of measures against global warming is an important topic. Since sewage sludge is one of the typical biomass with carbon neutral character, its utilization is likely to contribute to reduction of greenhouse gas. This section describes new sludge treatment technologies based on the above viewpoints.

6.14.2 Types of Sludge Treatment Technologies

The new sludge treatment technologies are summarised in Table 6.15 overleaf.

Table 6.15 Types of sludge treatment technologies

Process	Objectives	Description
Soil sludge immobilization	Utilisation as material for immobilized blocks	This implies a well dried sludge which is non-volatile and mixing with controlled amount of clay and clinker or fly ash or cement and burning like bricks. These are suitable only for chemical precipitated sludge and which are fully dried and volatilized before use. The immobilized blocks are used as paver blocks in walkways or compound walls where these are only panel fillers and not load bearing.
Sludge Drying	Utilisation as Soil filler, Fuel	Sludge drying system is one of several methods that can be used to reduce the moisture content and volume and improve the quality of sewage sludge. Dried sludge is usually used as soil conditioner and soil filler, and is recently being used as fuel. These are also used as pre-treatment in sludge incineration, melting and carbonization.

6.14.3 Heat Drying

6.14.3.1 Description

Sludge drying methods include heat drying and solar drying. Heat drying systems are described in this section. Heat drying system is one of several methods that can be used to reduce the moisture content and volume and improve the quality of sewage sludge. Dried sludge is usually used as soil conditioner and fertilizer and is recently being used as fuel. Heat drying systems are also used as pre-treatment for sludge incineration, melting and carbonization. Heat drying systems are divided broadly into direct drying and indirect drying, according to the heating method. The two systems differ in the process flow and drying characteristics. In order to select a drying system, it is necessary to consider the desirable moisture content of dried sludge. The technology is to spread on concrete platforms on a side open with roofed sludge shed and blow hot air over the width by a moving facility with arm extending to full width and with downward air diffusers and the to and fro movement controlled by trip switches. The air heater is integral to the pipeline and the cable alone travels to and fro. The quantity of air, heat and the rate of travel are tapered as the heating progresses. Cross ventilation of the shed and surrounding farm forestry in two layers are needed.

6.14.3.2 Advantages and Disadvantages

The advantages and disadvantages of this process are as follows:

a. Advantages

- It produces a sludge that can be easily transported to point of use without spillages.

b. Disadvantages

- Requires careful on-site adjustments to prevent dried sludge from being blown up and stopping the drying when the moisture content reaches about 25%.
- High O&M cost due to power requirement for heat generation.

6.14.3.3 Design Considerations

These are situation specific and the only guideline is to apply sludge in not less than 20 cm layers and not over 30 cm layers. To start with, and include a tiller in the hot air arm that can be used once in a while by manipulating the control as a level arm and a length of pipelines over the bed not exceeding 3 m.

6.14.3.4 Applicability

Heat drying is an effective sewage sludge management option for many facilities that need to reduce sewage sludge volume while also producing an end product that can be beneficially reused.

Heat drying is applicable to urban settings because it requires a relatively small amount of land and facility design allows process air to be captured for treatment. These are not yet applicable in India. When India has a huge agrarian base it is not correct to destroy the biological sludge which is a good soil filler.

6.14.4 Solar Drying

6.14.4.1 Description

The main reason for drying the sludge is the high cost of sludge disposal. Hence, every ton of water extracted from the sludge lowers the disposal cost for the STPs. Drying of the filter cake through thermal means, is one of the technically viable schemes. However, the energy requirement of the thermal drying process makes the operating cost prohibitively high.

Since solar radiation is the cheapest form of thermal energy, solar drying is a techno economic solution for sludge drying.

6.14.4.2 Main Components of Solar Sludge Drying System

The main components of a solar sludge drying system are as follows:

a. Sludge Drying Hall

This is like a Greenhouse, trapping solar radiation and ensures that the rain is kept out.

b. “TURNING” machine

The solar radiation warms the sludge’s surface. The rise in the temperature forces the water molecules out into the surrounding air. The moist air transports the water and has to be evacuated. However, while the surface dries the lower parts remain moist and have to be turned. This is achieved by a machine, which turns and also conveys the sludge across the floor of the drying hall. This also eliminates anaerobic areas that generate bad odour during sludge drying.

c. Control Panel

A PLC based control panel to ensure that the sludge drying process is monitored and controlled so that sufficient dryness is achieved in the final product.

d. Instruments

Various instruments to monitor parameters like Temperature, Humidity, Wind, Rain, etc., are provided to monitor the drying process.

The schematic of the process and infrastructure are shown in Figure 6.23 and Figure 6.24.

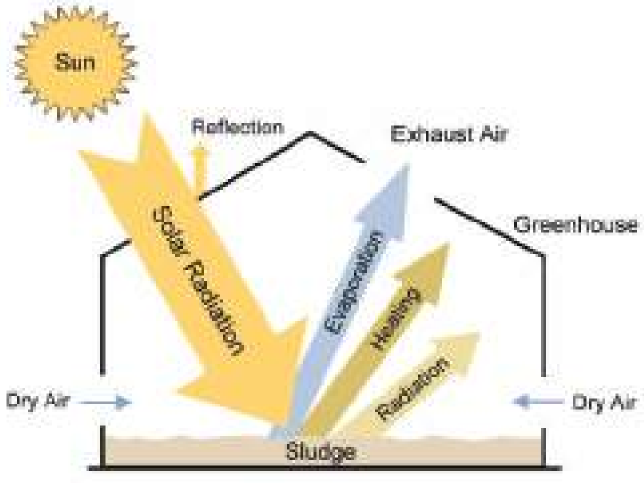
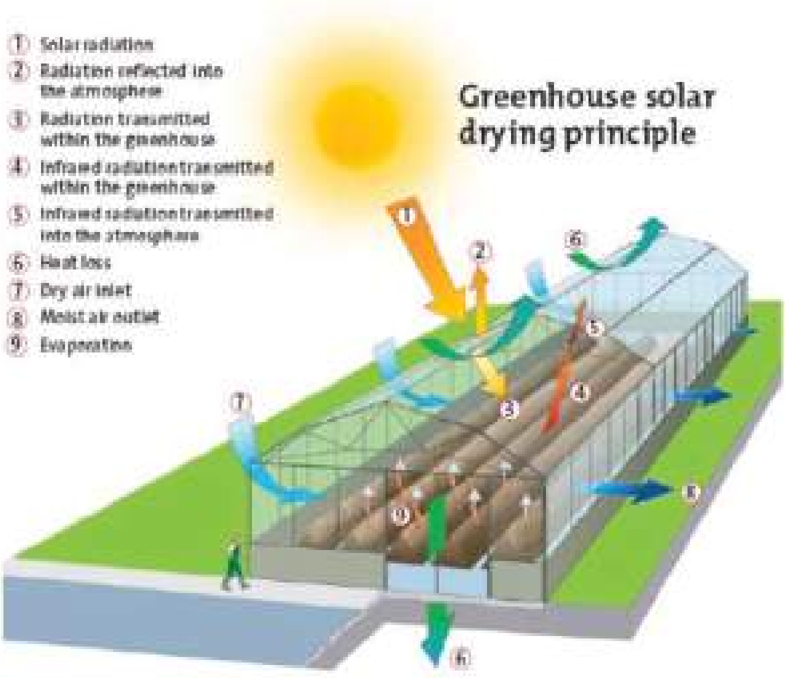


Figure 6.23 Typical operations in solar sludge drying



Source: http://www.veoliawaterst.com/solia/en/

Figure 6.24 Typical solar sludge drying

6.14.4.3 Advantages and Disadvantages

a. Advantages

- The cost of disposal is reduced by 75% to 80% as the solids content in the sludge is increased from 20% to 80% during drying.
- The process produces easy to handle bulk pellets from the sludge.
- Uses solar heat for the drying process instead of electricity thereby reducing the operating cost.
- The covered sludge drying area ensures continuous operation throughout the year, even during the rainy season.
- This ensures complete aeration and turning of the sludge in the entire sludge drying area.
- Eliminates anaerobic areas that generate bad odour during sludge drying
- Completely automatic process, without any human intervention during the drying process.

b. Disadvantages

- Proper precaution has to be ensured so as to ensure no fire hazard takes place in the solar sludge drying bed.

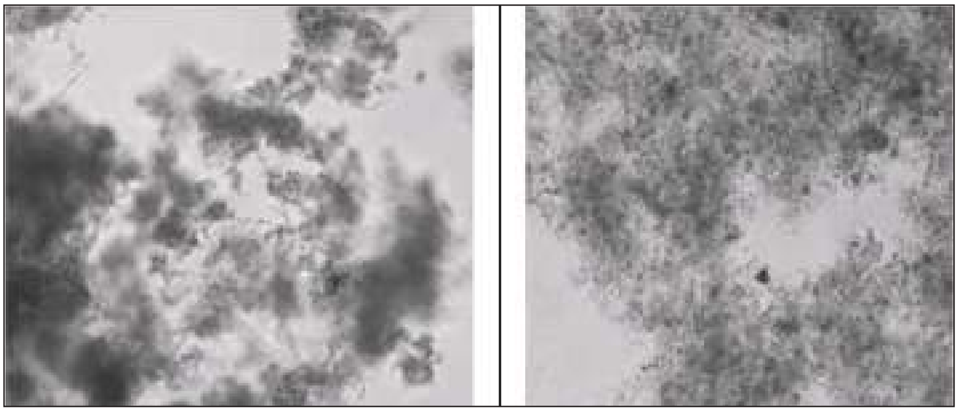
6.14.4.4 Potential Uses for Dried Sludge

- The dried sludge can be used as manure/soil conditioners.
- The dried sludge pellets can also be used as a fuel source in coal fired power plants and in cement kilns.

6.15 ULTRASONIC DISINTEGRATION OF FEED SLUDGE TO DIGESTER

A technology to disintegrate the sludge biosolids by using ultrasonic frequency (sonication) before feeding to anaerobic sludge digesters has been engaging the interest of a few research studies. A research study from Gdansk University of Technology, Poland and Vilnius Gediminas Technical University, Lithuania reported that the disintegration is stated to result in a more dispersed and homogenous flocs of activated sludge as in Figure 6.25 (overleaf) and intensified the biogas production and improved the quality of digested sludge.

The experiment is stated to have been carried out in laboratory scale digester using two reactors of each 15 days hydraulic retention time under mesophilic conditions. The results are reported to have showed a 20% increase of biogas and 10% increase of methane as compared to that from the digester fed with sludge biosolids without the ultrasonic disintegration. The authors concluded that due to sludge disintegration, the organic compounds were transferred from the sludge solids into the aqueous phase resulting in an enhanced biodegradability.



Source: Renata Tomczak-Wandzel. et. al., 2011

Figure 6.25. Activated Sludge before (left) and after (right) Ultrasonic Disintegration

The experiment demonstrated that this technology has the advantages of increase of sludge digestion efficiency, increase of about 20% of the volume of biogas produced, increase of about 10% of the methane content in biogas and reduction of the volume of residual sludge by 12%.

In another study by the Centre for Advanced Water Technology and the Public Utilities Board of Singapore, the ultrasound disintegration technology is stated as potentially useful to disintegrate feed sludge solids and enhance anaerobic digestion and gas yield. Two full scale digesters of 5000 cum capacity, each are reported as tested simultaneously in Singapore one with and the other without ultrasonic disintegration of feed sludge. It is reported that in comparison with the control, i.e., without sonication, the five-month field study showed that ultrasound pre-treatment of the sludge increased the daily biogas production up to 45% and there were no significant differences in biogas composition from the two digesters. It is inferred that an increase in sludge solids removal of up to 30% is expected under optimal operation conditions. (Source Rongjing Xie, et.al., 2007).

These technologies appears to hold promise towards energy savings and reduction in volumes of anaerobic digesters for sewage treatment.

6.16 GUIDING PRINCIPLES OF SLUDGE TREATMENT TECHNOLOGIES

Every effort should be made to go eco-friendly in dealing with biological sludge from STPs. They need to be dried to about 20% moisture and then integrated with the agriculture and farm forestry. If needed to be applied on sensitive lawns, Gamma ray irradiation of the sludge is mandatory before such application. The advancement in anaerobic sludge digestion in the coming years may address these processes, whereby the raw sludge will be preheated from 60°C to 80°C for pasteurization and mixed with recycled hot digester gas.

If needed, it will be supplemented with steam thus, bringing about biological hydrolysis, which can generate more renewable energy than conventional thermal hydrolysis as in the present day digesters.

CHAPTER 7: RECYCLING AND REUSE OF SEWAGE

"No higher quality water should be used for a purpose that can tolerate a lower grade" UN Council Resolution-1958

"Many of the wars this century were about oil, but those of the next century will be over water."- Ismail Serageldin, Vice President, World Bank-1995

Question to finalists

Technology is good for comfortable life; It is also blamed for environmental problems; How do you link Technology & Environmental Conservation?
Answer by winning finalist
Agrarian economy must reuse water safely
From the Miss Earth contest, Manila, Sponsored by WHO 2001

"Water should not be judged by its history, but by its quality." Dr. Lucas van Vuuren, one of the pioneers of the Windhoek water reclamation system.

7.1 INTRODUCTION

With 80 countries and 40% of the world's population facing chronic water problems and with the demand for water doubling every two decades, these extracts mentioned above merit action. The largest source of reuse resides in agriculture and the equally largest misplaced resource is sewage in the habitations. In the "Handbook on Service Level Benchmarking" by MoUD, reuse and recycling of sewage is defined as the percentage of sewage recycled or reused after appropriate treatment in gardens and parks, irrigation, etc. and, is to be at least 20% to begin with. The objective of this chapter is to bring out guiding principles for practice in India.

7.1.1 Overview of Current Practices in India

In India treated sewage is being used for a variety of applications such as (a) Farm Forestry, (b) Horticulture, (c) Toilet flushing, (d) Industrial use as in non-human contact cooling towers, (e) Fish culture and (f) Indirect and incidental uses. They are briefly mentioned hereunder.

a) The CMWSSB has been promoting the growth of farm forestry in Chennai from the 1980s and this helps to promote a micro climate in a city environment.

b) The Indian Agricultural Research Institute, Karnal has carried out research work on sewage farming and has recommended an irrigation method for sewage fed tree plantations.

c) The University of Agricultural Sciences, Dharwad, Karnataka has found that sewage could be used in producing vermicompost to be used for tree plantations provided its details with respect to composition of toxic substances are known.

d) Chandigarh is using treated sewage for horticulture needs of its green areas.

e) Delhi has put in place planned reuse of treated sewage for designated institutional centres.

f) The Government of Karnataka has issued an official directive to take all necessary steps to ensure that only tertiary treated sewage is used for non-potable purposes, like all gardening including parks, resorts and golf course. The Bangalore Water Supply and Sewerage Board will make all arrangements including construction of filling points, installation of vending machines at STP for supply of tertiary treated sewage in multiples of thousand litres and that non-compliance of the directions attracts penal provisions in accordance with section 15 and section 17 of the Environment (Protection) Act 1986.

g) In major metropolitan cities like Delhi, Mumbai, Bangalore and Chennai treated grey water is being used for toilet flushing in some of the major condominiums and high rise apartment complexes on a pilot scale. Care should be taken to ensure that Ultra filtration membranes are used in the treatment process to safeguard against chances of waterborne diseases.

h) Secondary treated sewage is purchased and treated for use in cooling water makeup in the industrial sector from as early as 1991 in major industries like Madras Refineries, Madras Fertilizers, GMR Vasavi Power plant in Chennai as also in Rashtriya Chemicals and Fertilizsers in Maharashtra and most recently in the Indira Gandhi International Airport in Delhi and Mumbai International Airport.

i) In Kolkata, the Mudiali fish farm occupying an area of 400 hectares is used for growing fish, which is then sold for human consumption.

j) The UNDP conducted a detailed study in the 1970s and identified a sand basin on the coast of Bay of Bengal, where secondary treated sewage of the Chennai city can be infiltrated through percolation ponds and extracted for specific industrial use in the nearby petro-chemical complex. However, this project has not been implemented.

k) The Bengaluru city is facing a freshwater crisis and it has been considered to study a pilot model of the Singapore NEWater for indirect augmentation of water by advanced treatment of secondary facilities. At present, this project proposal is a statement of capability to formulate a technically feasible and financially viable project and of course the biggest challenge of going through and obtaining public acceptance is understandably a long drawn out process.

7.1.2 Overview of Current Practices in the World

The use of treated sewage elsewhere in the world is listed herein and in Appendix 7.3.

a) Agriculture: It is used for irrigation in certain places in Africa, Israel, Mexico and Kuwait.

b) Farm Forestry: Treated sewage is used for watering urban forests, public gardens, trees, shrubs and grassed areas along roadways in certain places in Egypt, Abu Dhabi, Woodburn in Oregon USA. It is also used for timber plantation in Widebay Water Corporation in Queensland, Australia. It is used for alfalfa plantation in Albirch Palestine.

c) Horticulture: Certain places in Elpaso in Texas, Durbin Creek in Western California in USA.

d) Toilet flushing: Certain locations in Chiba Prefecture, Kobe City, and Fukuoka City and Tokyo Metropolitan in Japan.

e) Industrial and commercial: essentially used for cooling purposes in Sakaihama Treated Wastewater Supply Project, Japan, Bethlehem Steel mills, USA. Sewage reclaimed as high quality water is supplied to Mondi Paper Mill and SAPREF Refinery in Durban, South Africa. Landscape and golf course irrigation in Hawai,

f) Fish culture: It is used in fish hatcheries / fish ponds in Vietnam and in Bangladesh

g) Groundwater recharge: Orlando and Orange County Florida, Orange County California, Phoenix (Arizona), Santa Rosa (California) Recharge Project all in USA.

h) Indirect recharge of impoundments: Restoration of Meguro River in Japan, NEWater project in Singapore, Windhoek in Namibia, Berlin in Germany

i) Other uses: Coach cleaning, subway washing and water for building construction is being practised in Jungnang, Nanji, Tancheon, Seonam in Seoul and treated sewage sprinkled on the water retentive pavement that can store water inside paving material at Shiodome Land Readjustment District (Shio Site) in Tokyo and this reduces the surface temperature.

7.2 CASE STUDIES IN RECYCLING AND REUSE OF SEWAGE

7.2.1 Raw Sewage Treatment and Reuse as Cooling Water at M/S GMR Vasavi Power Plant, Chennai, India

7.2.1.1 The Pride of Place

This plant is the first of its kind in Asia commissioned as early as 1999, where the raw sewage of Chennai city is treated to recover (a) water of grade suitable for makeup in the cooling water and (b) is also further treated to recover a water of boiler grade.

7.2.1.2 Treatment Schematic

This is shown in Figure 7.1.

7.2.1.3 Raw and Recovered Sewage Qualities

These are given in Table 7.1 and Table 7.2

7.2.1.4 Step by Step Reasoning of the Treatment

Need for Equalization Basin Up Front: The flow of sewage is not uniform throughout 24 hours and the biological treatment plant can absorb the fluctuations and still yield a fairly steady level of treated sewage quality. However, in the Lime addition of the chemical system, the flow rate should be necessarily uniform, to facilitate dosing of chemical uniformly. This implies flow equalization necessarily at some point. It was decided to have this before the biological treatment itself, so that the dosage of Sodium bicarbonate whenever needed (before the primary clarifier to ensure adequate bicarbonate alkalinity for biological nitrification in the aeration tank) can be controlled easily in a steady state, which will further avoid unduly high Sodium in the resulting sewage. An example of sizing the equalization tank volume is given in Appendix A.5.2.

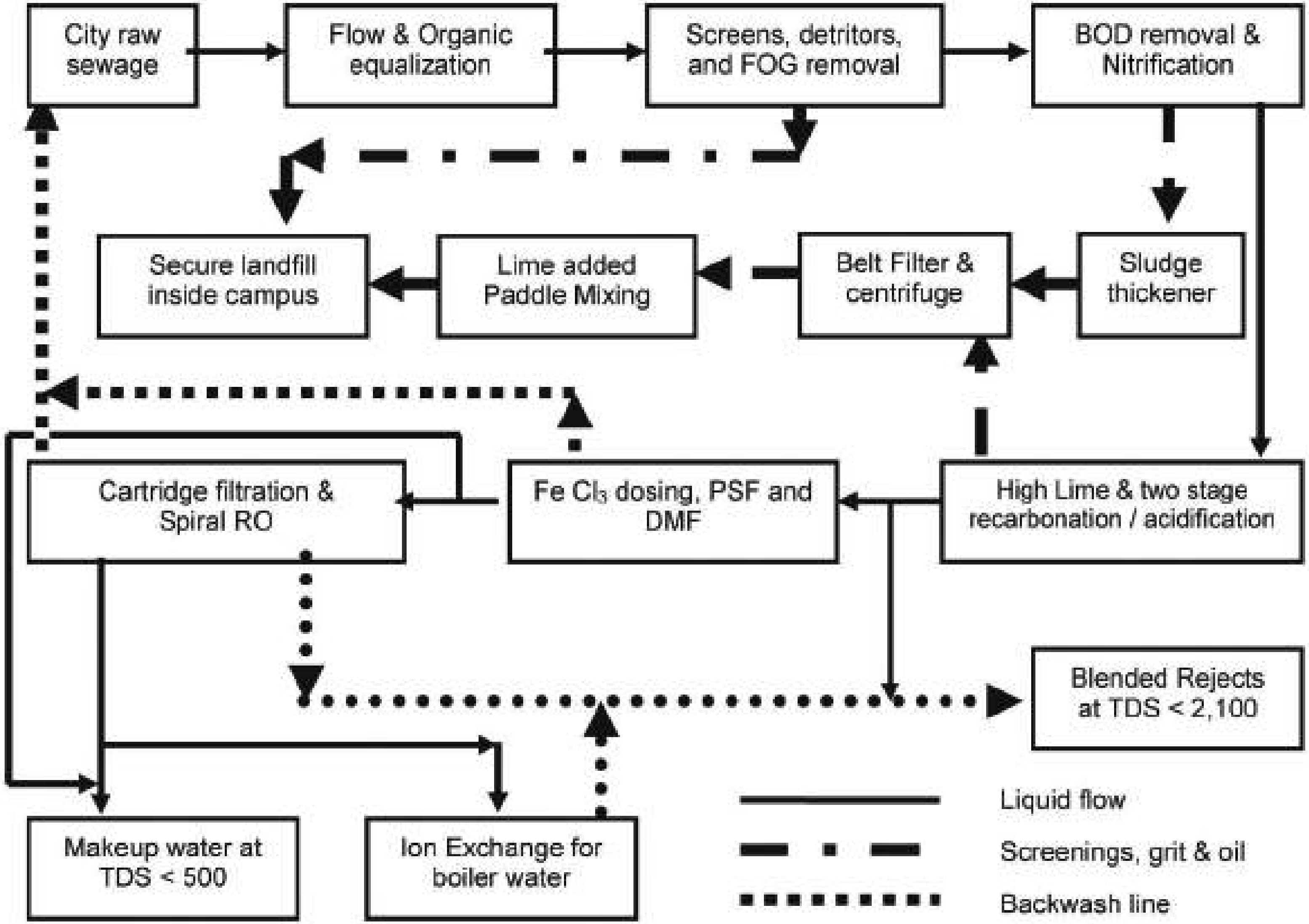


Figure 7.1 Treatment schematic of the M/S GMR Vasavi power plant sewage reuse plant

Table 7.1 Raw sewage quality variations taken for the design

No.	Parameters	Max	Min	No.	Parameters	Max	Min
1	pH	7.9	7.6	14	Phe. Alk. as $CaCO_3$		
2	Total Dis. Solids	1360	980	15	Total Alk. as $CaCO_3$	670	530
3	Inorg. Dis. Solids	95%	94%	16	Sodium as Na	370	280
4	Suspended Solids	590	460	17	Chloride as Cl	400	380
5	BOD 5 days 20°C	540	350	18	Sulphates as SO_4	130	130
6	COD, 2 hours reflux	850	520	19	Oil and grease	16	6.4
7	Ammonia N as N	60	37	20	Phenolics	0.5	0.45
8	TKN as N	105	83	21	Surfactants	7.8	5.8
9	Ortho P as PO_4	33	28	22	Sulphide as S	16	11
10	Total P as PO_4	39	35	23	Fluoride as F	1.2	0.9
11	Ca as $CaCO_3$	170	120	24	Silica as SiO_2	38	32
12	Mg as $CaCO_3$	230	180	25	Summer temp. as °C	39.5	39
13	Tot Hardness, $CaCO_3$	400	300	26	Winter temp. as °C	29	28

Table 7.2 Makeup water quality required for recirculation cooling

No	Parameters	Range	No	Parameters	Range
1	Langelier Index	0.1 to 0.2	4	Phosphate	Nil
2	Total Dis. Solids	550 to 600	5	Ammonia	Nil
3	Silica as SiO_2	2 to 3	6	Others	As arising

In 1997, the aerators used were mostly slow speed surface aerators, and it was not easy to adjust the air input unlike the diffused aeration, where the VFD controlled motor permitted variations of air flow and hence, the oxygen input into aeration system based on the peak, average and lean flow durations could not be pro-rata adjusted. Hence, it was necessary for upfront equalization.

Before feeding to the RO plant, the ammonia and phosphorous have to be removed as these can cause corrosion of some metal surfaces and biofouling in the circulating water respectively.

For this purpose, ammonia removal options of high lime induced air stripping, biological nitrification, chlorination and Clinoptinolite resin exchange were evaluated and it was decided to follow the biological nitrification route, given its high degree of reliability and the fact that nitrates can be rejected in the RO and residual presence in permeate was not prohibited.

The combined BOD removal and ammonia nitrification in the same aeration tank was chosen because of its well proven performance in the Chennai TWAD Board R&D unit. Studies on field scale pilot plant validations of localized design criteria were carried out between 1965 to 1980 at Kodungaiyur R&D facility, which eventually became the forerunner for such prototype plants in India.

The desired cooling water makeup needed a quality where silica was to be restricted to less than 3 mg/l. The raw sewage silica content was about 38 mg/l. The magnesium (Mg) content was 180 mg/l at the minimum. This when precipitated at pH above 10.5, will reduce Mg to less than 25 mg/l. Thus Mg removal could be 155 mg/l. This can co-precipitate silica by 155/5 = 31 mg/l, which meets the requirement of silica removal before entering RO to less than 5 mg/l, further by restricting the recovery at about 75%, the silica in the permeate can be held down to 1 mg/l and reject silica can well be about 20 mg/l.

Maximum removal of Mg is possible only at a pH of above 10 and hence, the high lime process of the biologically treated sewage was decided upon.

The incidental advantages of complete precipitation of phosphorous, alkaline oxidation of residual organic matter, destruction of colour and especially inactivating the pathogenic organisms after at least 45 minutes contact time and precipitation of heavy metals as their oxides were all recognized as incidental to the high lime process and hence this was chosen.

The neutralization of high lime treated effluent was chosen to be used through two stage carbonation, whereby the first stage will be cut off at a pH of about 9.3 to enable precipitation of the originally available calcium and the added calcium. The second stage neutralization will be to reach the pH of close to 6.5 which is the desired limit in RO feed water. The proposed carbon dioxide cylinders are easily available locally. Alternative acidification is also used.

The restriction in the disposal of the plant reject is governed by a TDS limitation of 2,100 mg/l. In order not to exceed this limit, the biological and chemical treatment segments were of higher capacity, than actually required to feed to RO to obtain the required permeate volume. The excess volume with a TDS reduced by about 550 mg/l in the chemical treatment segment was used to dilute the RO rejects.

Even though phosphorous precipitation was expected to be complete in the high lime stage, a backup was provided by dosing FeCl3 on line and providing a static mixer after the carbonation in the feed pipe line of pressure sand filters.

A dual media filter was chosen to further filter out the chemically treated water, thereby ensuring that even a chance occurrence of phosphorous in the RO feed is avoided entirely. By this, the contributory cause of bio fouling of the RO membrane could also be avoided. The treatment schematic as in Figure 7-1 (overleaf) above was thus chosen to be implemented.

7.2.1.5 Key Design Criteria

- Primary and secondary clarifiers were generally as per CPHEEO guidelines
- Sludge withdrawal was by direct suction to ensure adequate velocity of drawal
- The F/M value was 0.25 and HRT for DWF was 8 hours in aeration tank
- Alpha and Beta Factors were 0.75 and 0.95 with residual DO of 2 mg/l
- The MLSS was designed at 2,500 mg/l and was adjusted based on field conditions

- Mixing power was maintained at 20 watts per cum of aeration tank volume
- HRT in excess lime and first carbonation clarifiers were as per CPHEEO guidelines
- RO system design was as per the membrane manufacturer.

7.2.1.6 Performance Results

The plant is in continuous O&M ever since 1999 and has attained the desired key criteria of TDS less than 600, silica almost nil, etc. besides clear and colourless nature of the RO feed ever since.

7.2.1.7 Pointers for the Future

The expected precipitation of calcium in the first stage carbonation was sometimes erratic leading to calcium escape. Though this was dissolved into bicarbonate in the second stage carbonation, at times this was difficult and the neutralization was switched over to use of hydrochloric acid. This increased the calcium content in feed to RO and required readjustments of the blending to peg the TDS in the product water. The use of a solids contact clarifier instead of the plain clarifier could have been a better choice, but the possibility of the calcium carbonate sludge solidifying therein, and choking the sludge withdrawal lines were the other issues.

Conventional precipitation of Ca and Mg could have been tried instead of high lime carbonation, but the fact that phosphorous even at 0.1 mg/l prevents solids liquid separation of the precipitated carbonate thus, defeating the objective was the reason. All the same, future plants need to carefully assess these options.

The original RO membranes of brackish water grade though were guaranteed for only 4 years by the manufacturer, lasted for as many as 7 years before the recovery dropped by 10%. This is clearly traceable to the total sterility of the water exiting the high lime stage.

Lately, the use of UF membrane has increased in India. The historical Water Factory 21 in California has also originally used the high lime carbonation route, but later changed over to microfiltration route. Though this may look attractive prima facie, the need to look into phosphorous removal, which can be fully possible only in high lime has to be borne in mind. Whereas the water factory was tackling the raw sewage phosphorous of only single digit, the plant cited here was tackling as much as 35 mg/l. In that case, there is no way, the MF or UF can eliminate phosphorous, especially, if it is in colloidal form.

Moreover, soaps and detergents in India continue to bring in inorganic phosphates in the raw sewage. As far as sewage is concerned, this single factor may lead to bio-fouling of RO membranes. A combination of high lime and UF would perhaps be the best available technology.

With respect to the sludge cake, the wet biological sludge from the filter press machines, was blended with the high Lime precipitated sludge, and the cake was further treated to raise the pH of the same to about 9.3 by a paddle mixer. The resulting sludge mass was being used to raise the low lying areas and served as a secure land fill in the clayey soil. This pH adjustment of the sludge cake was worthy of consideration.

7.2.2 Reuse Plant at Indira Gandhi International Airport, Delhi, India

This is the latest plant in India. The raw water is taken from local piped water as also from local bore well water as a backup & treated in RO. The rejects from this system, the sewage from the new terminal building and from the old infrastructure are all blended as a sort of pseudo sewage as different from conventional city sewage in regard to ammonia to BOD ratio being much higher. This was due to the nature of usage of toilets in the terminal building almost entirely for urination as compared to the water closets. This pseudo sewage was taken to a biological nitrification-denitrification reactor using the MLE process. Further, the biological treated sewage is equalized before being filtered through DMF after on line injection of FeCl to remove possible colloidal phosphorous, and thereafter through UF, cartridge filter and RO. The bio-reactor is unique in shape so that the plug flow configuration can be covered by a funicular polygon, if needed later on. The water balance in this plant is shown in Figure 7.2.

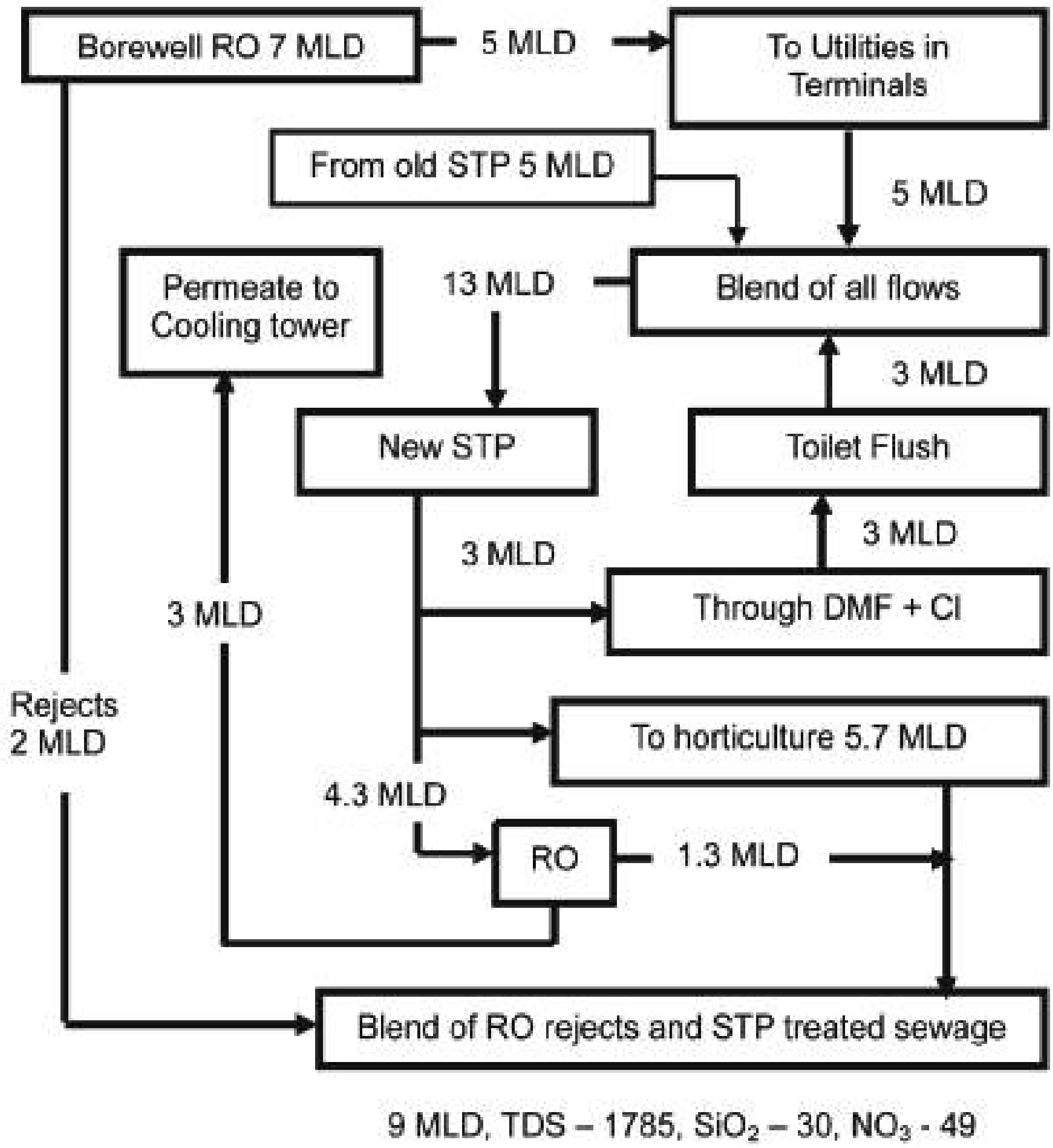


Figure 7.2 Water balance in the New Delhi IGI airport sewage reuse plant

7.2.2.1 Water Routing of this Plant

As shown in Figure 7-2, bore well water will be treated in RO and the product water supplied to the new Terminal building for utilities and potable uses. The treated sewage will be equalized for flow and filtered through Dual Media Filters (DMF) with on line $FeCl_3$ addition and reused partly in toilet flush as a water conservation initiative. This component will actually be a closed loop. The rest of the treated sewage will be split into two streams one for greenery and the other for RO to use the permeate as cooling tower makeup water. The rejects from the RO plants and the bypass of treated sewage will be blended and used for sustaining the greenery with maximum water conservation and ensuring blended TDS of less than 2100 mg/.

7.2.2.2 Key Design Criteria

1. An important issue is the relatively higher presence of ammonia as passengers in terminal building do not use the water closet very often, but they use the urinal.
2. The temperature for biological design was 37°C and 10°C in summer and winter respectively
3. The raw sewage BOD was taken as maximum of 200 mg/l and SS of 400 mg/l
4. The peak factor was taken as 1.5 as the terminal building was used almost continuously
5. The RAS was at unity and IRR from aeration tank was twice the DWF
6. HRT in anoxic tank was at 0.5 hours based on all flows through it
7. HRT in aeration tank was 18 hours based on DWF
8. Alpha value was consciously restricted to 0.6 as Kla was retarded in this sewage
9. Nitrification oxygen was taken as 4.8 and oxygen credit was taken as 2.86
10. Mixing air was taken as 30 cum/minute/1000 cum of aeration tank
11. The phosphorous leaving the DMF in dissolved form was allowed to go through UF and RO
12. The only solid waste from the plant is the biological sludge cake. This is used in the root zone of trees in the greenery as a soil filler/organic fertilizer
13. The quality of the blended discharge for greenery meets the requirements of pollution control
14. The plant is user friendly with PLCs and permits off-site monitoring
15. Simplified treatment scheme is shown in Figure 7.3.

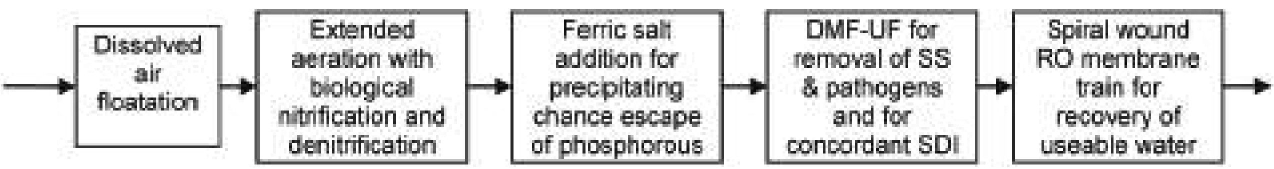


Figure 7.3 Simplified treatment scheme for IGI airport Delhi STP

7.2.2.3 Performance Results

The STP receives raw sewage from the airport terminal building and though it has higher nitrogen content as compared to domestic sewage because passengers use the urinals much more than toilets. Yet the design wetted by IIT Delhi is able to handle the biological nitrification-denitrification. The performance of the STP for raw sewage and biological treated sewage is shown in Table 7.3.

Table 7.3 Performance of the STP at DIAL for higher Nitrogen loaded sewage

No	Location	BOD	Ammonia Nitrogen	SS
1	Raw Sewage	110 to 130	45 to 53	240 to 280
2	Biological Treated sewage	8 to 10	1.5 to 3	10 to 15

The design of bioreactor is based on mixed liquor return at twice the average flow, return sludge at equal to average flow, volume of anoxic tank at 30 minutes of all flows and volume of aeration tank at 18 hours of average flow. The results reported here are at 67% of design flow

7.2.3 Reuse Plant at Mumbai International Airport Limited (MIAL), India

This is a circular SBR tank followed by hypo chlorination, pressure sand filtration and ultra filtration to produce a grade of recovered sewage free of colour, organics and odour. The treated effluent is partly used for toilet flush and the rest is put through RO membranes to recover make up grade water for HVAC.

The sewage from the terminal building is disproportionately high with urine and nitrogen as compared to normal domestic sewage and MIAL has evolved its own process design for SBR based on extended aeration.

The design for 4 MLD average flow has 2 numbers of SBR basins of each 20 m diameter and 6.5 m liquid depth, floor mounted fine bubble diffusers, floating floor level anoxic mixers, floating decanter, Alpha value as 0.85, Beta value as 0.95, F/M as 0.08, MLSS as 4500, Kg Oxygen for Kg BOD at 1.25, Kg Oxygen for Kg ammonia at 4.6, number of cycles per day as 5, alkalinity used up as 7.2 mg / mg of nitrified N, alkalinity released as 3.6 mg / mg of denitrified N and the performance at 50% of design flow as in Table 7.4.

Table 7.4 Performance of the SBR at MIAL for higher Nitrogen loaded sewage

No	Location	BOD	COD	SS	TKN	Alkalinity	TP	pH
1	Raw Sewage	102.0	230.00	55.0	42.0	190	8.0	6.95
2	SBR effluent	3.4	15.26	6.5	5.2	10	1.7	6.90

A photo of one of the SBR tanks with floating decanter and central draft tube anoxic mixer is shown in Figure 7.4.

Figure 7.4 Circular SBR based STP at MIAL with floating decanter

It shows that the heavily loaded nitrogen relative to organic matter can still be treated successfully in biological SBR at the design criteria as above and a hydraulic detention time of 24 hours in the SBR.

The final disinfection is by hypochlorite and the baffled chlorinating tank is circumventing the SBR.

7.2.4 Pointers for the Future

1. In dealing with these institutional pseudo sewage of types similar to airport terminals, the MLE process of biological nitrification-denitrification works well even when the ammonia content is higher, but then it takes about 3 months to establish the microorganisms culture with a steady dosage of micro nutrients as in Table 7.5 (overleaf). This is very important.

2. The use of extended aeration is to be preferred as conventional ASP with F/M in the range of 0.3 to 0.5 may suffer upsets when ammonia dominates in the sewage.

3. Biological phosphorous removal in upstream anaerobic reactor may or may not yield expected results in this type of sewage, where ammonia dominates the BOD at various times.

4. As long as the phosphorous is ensured to be in dissolved form it can be allowed through the UF and RO membranes, and there is no need for an exclusive phosphorous removal unit.

5. The raw sewage pump sets were of the centrifugal screw impeller in wet submersible sumps. Though it had to be imported, they were considered to be fail-proof to handle raw sewage in this sensitive location without getting choked by unexpected obstructing matters entering the sewage and may instantaneously affect the air conditioning in the terminal.

Table 7.5 Micro nutrients to be added to biological systems where microbial growth is detectable as retarded (as done at IGI Airport STP)

	A	B	C	D	E	F	G	H	I	J
1	Designer to use this space for his notes									
2	Designer to use this space for his notes									
3	Designer to use this space for his notes									
4				Flow in mld	4	BOD at Inlet	300			
5			Atomic Weight	mg/mg of BOD, as element	Market Chemical	Chemical Formula	Molecular Weight	Calculation of weight of compound	mg/mg of BOD, as compound	kg needed, once a month
8	Calcium	Ca	40	62 ×10 power minus 4	Calcium carbonate	$CaCO_3$	100	62* POWER((10),(-4))*G7/C7	0.0155	18.60
9	Cobalt	Co	58	13 x 10 power minus 5	Cobaltic chloride	$CoCl_2(6H_2O)$	238	13* POWER((10),(-5))*G8/C8	0.0005	0.64
10	Copper	Cu	64	15 x 10 power minus 5	Cupric sulphate	$Cu(SO_4)$	160	15* POWER((10),(-5))*G9/C9	0.0004	0.45
11	Iron	Fe	56	12 x 10 power minus 3	Ferrous ammonium sulphate	$FeSO_4 (NH_4)_2(SO_4)$ (6 H_2O)	392	12* POWER((10),(-3))*G10/C10	0.0840	100.80
12	Magnesium	Mg	24	30 x 10 power minus 4	Magnesium chloride	$MgCl_2$	95	30* POWER((10),(-4))*G11/C11	0.0119	14.25
13	Manganese	Mn	55	10 x 10 power minus 5	Manganese chloride	$MnCl_2(4H_2O)$	198	10* POWER((10),(-5))*G12/C12	0.0004	0.43
14	Molybdenum	Mo	96	43 x 10 power minus 5	Molybdic acid	MoS_2	106	43* POWER((10),(-5))*G13/C13	0.0005	0.57
15	Potassium	K	39	45 x 10 power minus 4	Potassium chloride	KCl	75	45* POWER((10),(-4))*G14/C14	0.0087	10.38
16	Selenium	Se	79	14 x 10 power minus 10	Selenium chloride	$SeCl_4$	228.625	14* POWER((10),(-4))*G15/C15	0.0041	4.87
17	Sodium	Na	23	5 x 10 power minus 5	Sodium chloride	NaCl	59	5* POWER((10),(-5))*G16/C16	0.0001	0.15
18	Zinc	Zn	66	16 x 10 power minus 5	Zinc oxide	ZnO	82	16* POWER((10),(-5))*G17/C17	0.0002	0.24

The M S Excel sheets for calculating these for continuous and batch flow systems are given as Appendix 7.1 and 7.2 in the soft copy version.

7.2.5 Reuse Plants at Chennai Petroleum Corporation Ltd. (CPCL) and Madras Fertilizers Ltd. (MFL)

These are the earliest plants designed and constructed in India between 1989 and 1991 for recovering makeup grade cooling water from Chennai city sewage. These plants however, received only the secondary treated sewage from Chennai city STPs. All the same, they still provide the biological nitrification and thereafter, high lime acidification and then RO. The RO rejects are let into the backwater zone and not directly into the marine area. The flow schematics of these plants are in Figure 7.5 and Figure 7.6. The plants treat about 12.5 mld and 17.5 mld, respectively.

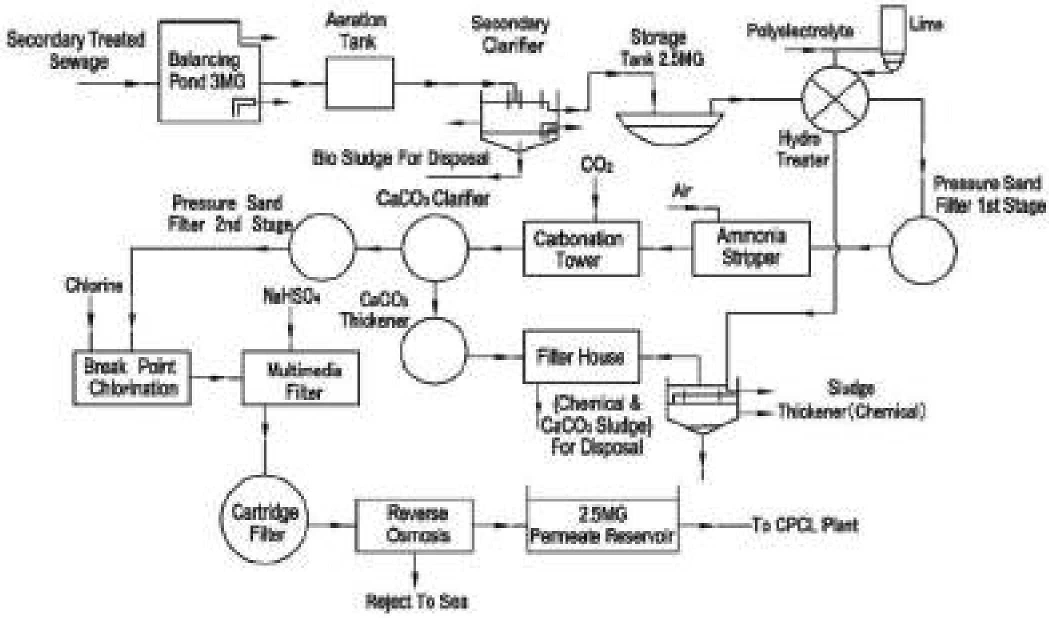


Figure 7.5 Sewage reuse schematic at M/S CPCL, Chennai

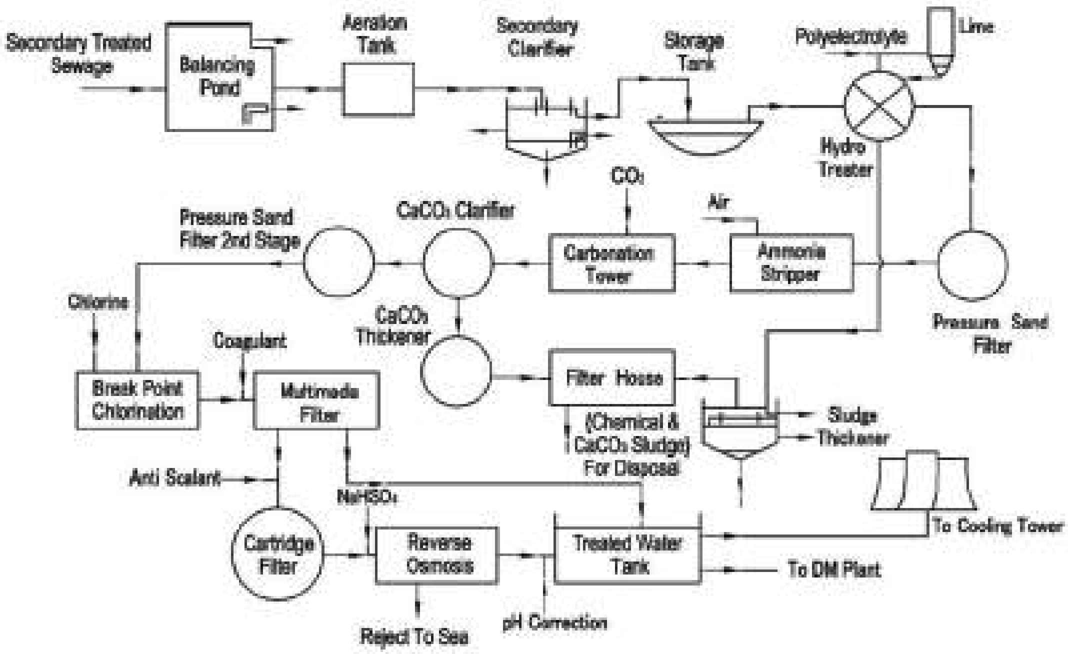


Figure 7.6 Sewage reuse schematic at M/S. MFL, Chennai

7.2.6 Sewage Reuse Plant at M/S Rashtriya Chemicals and Fertilizers, Mumbai

This is a plant receiving raw sewage and recovering cooling grade makeup water. Its flow scheme is shown in Figure 7.7.

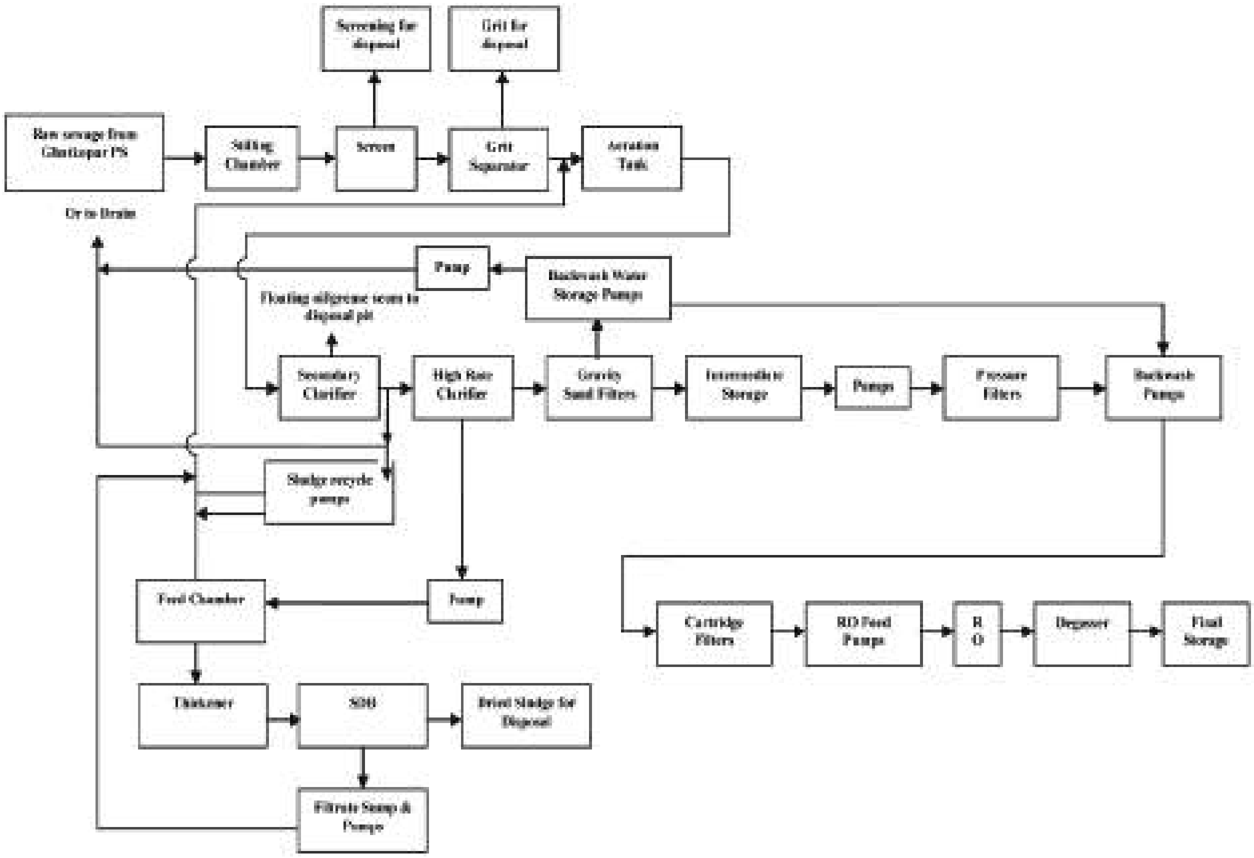


Figure 7.7 Sewage reuse schematic at M/S RCF, Mumbai

The 23 mld STP commissioned in the year 2000 treats a complex municipal sewage heavily contaminated with various industrial wastes. Though originally conceived with a single step chemical treatment after biological treatment, subsequently some additional treatment steps like use of UF became necessary in order to improve the quality of the water reaching the RO system (keeping the silt density index, SDI < 3.0) owing to the more polluted nature of the influent sewage.

This is a classic scenario of the need to design the treatment process to be flexible for impacts by industrial effluents in the raw sewage and especially the trace metals and heavy metals. These are possibly better served by the high lime and neutralization than a single stage chemical treatment.

7.2.7 Bengaluru, India

The Bengaluru city freshwater crisis is so high that its present quota of freshwater from the river Cauvery will get exhausted soon and the demand of the city may overtake the supply. It was considered to study a pilot model of the Singapore NEWater for indirect augmentation by advanced treatment of secondary facilities.

The proposal was to add biological nitrification denitrification and tertiary treatment in chemical precipitation of phosphorous and cascade the treated water over 20 km and a 65 m fall in a river course.

The runoff will be intercepted and subjected to conventional water treatment with clariflocculators and rapid sand filters and then pumped back through the 65 m rise by a pipeline with chlorination. Thereafter, it will be put through dual media and activated carbon filtration followed by UF and RO membranes.

The idea was to ensure removal of endocrine disruptor chemicals (EDCs) by activated carbon and enteroviruses by ultra filtration membranes. The RO will bring back the TDS to the freshwater levels and ensure additional removals of virus if any. The RO rejects will be put through accelerated evaporation spray ponds.

The RO permeate will be let into a freshwater river course to travel about 8 km before it enters a freshwater impoundment. The detention period calculated by the volume of the impoundment and the volume of renovated water will be close to two years to bring out limnological equilibrium of the blended water through the seasons before drawal into a conventional water treatment plant (WTP) and chlorination before being blended with the freshwater supplies.

The sludge from the WTP will be stored in secure landfills subject to further studies on soil sludge immobilization for making paver blocks for walkways and compound walls.

The volume of such indirect augmentation will become close to 140 mld compared to the availability of treated sewage of 1500 mld by the time the project could be completed after due public hearing, subject to which, the project has been accorded sanction by the JnNURM as a pilot project. Understandably, such projects will take time to materialize.

The schematic of this treatment is shown in Figure 7.8 (overleaf).

A subsequent thinking is to explore the possibility of a dedicated cascade channel along the 20 km river course if the river purification gets into time delays by the time these two are to dovetail in the future. The cost of the renovation was Rs.15.6 per kilo litre of water produced and compares favourably with the cost of freshwater production at Rs.14.2 per kilo litre.

At this time, this project proposal is a statement of capability to formulate a technically feasible and financially viable project and of course the biggest challenge of going through and obtaining public acceptance is understandably a long drawn out process.

The treatment sequence proposed in this project is shown in Table 7.6 and is compared with the treatment sequence used in other similar known installations elsewhere.

7.2.8 Karnal, India

The Indian Agricultural Research Institute at Karnal, India has carried out work on sewage farming and has recommended that growing tree on ridges 1 m wide and 50 cm high with even untreated sewage in furrows can still be considered.

Table 7.6 Comparison of treatment barriers used in projects in the World for indirect potable reuse of treated sewage

No.	Location	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Windhoek, Namibia	√						√	√	√					√	√
2	Occoquan, USA				√	√	√	√								
3	Singapore	√									√	√			√	√
4	V. Valley, Bangalore	√	√	√	(*)	NR-1	√	√	√	√	√	NR-2	√	√	√	√

Explanatory Notes of Treatment Components

A	Tertiary treatment including removal of nitrogen and phosphorous besides organics	F	Sand Filtration to hold back escaping suspended solids and improving carry overs	K	R O membrane filtration for reducing the hardness of recovered water as needed
B	18 Km travel of tertiary treated sewage in river course for natural self rejuvenation	G	Chlorination in pumping main with 6 hours contact in booster doses en route	L	7 km of travel of pathogen free water in natural river valley for naturalization
C	Storage in pick up weir for disinfection by sun rays & natural aquatic equilibrium	H	Activated Carbon Treatment for removing trace organics and chance THMs	M	450 days in open reservoir for disinfection by sun rays & natural aquatic equilibrium
D	High Lime for inactivation of pathogenic organisms & heavy metal precipitation	I	Ultra Filtration for removal of enteric pathogens and specifically viruses	N	Conventional water treatment for floating matters and hardness reduction
E	Two stage carbonation to remove the increased TDS due to high Lime treatment	J	Micro filtration for removal of upto bacteria alone (but not viruses)	O	Residual chlorination of treated water to ensure detection of sterility

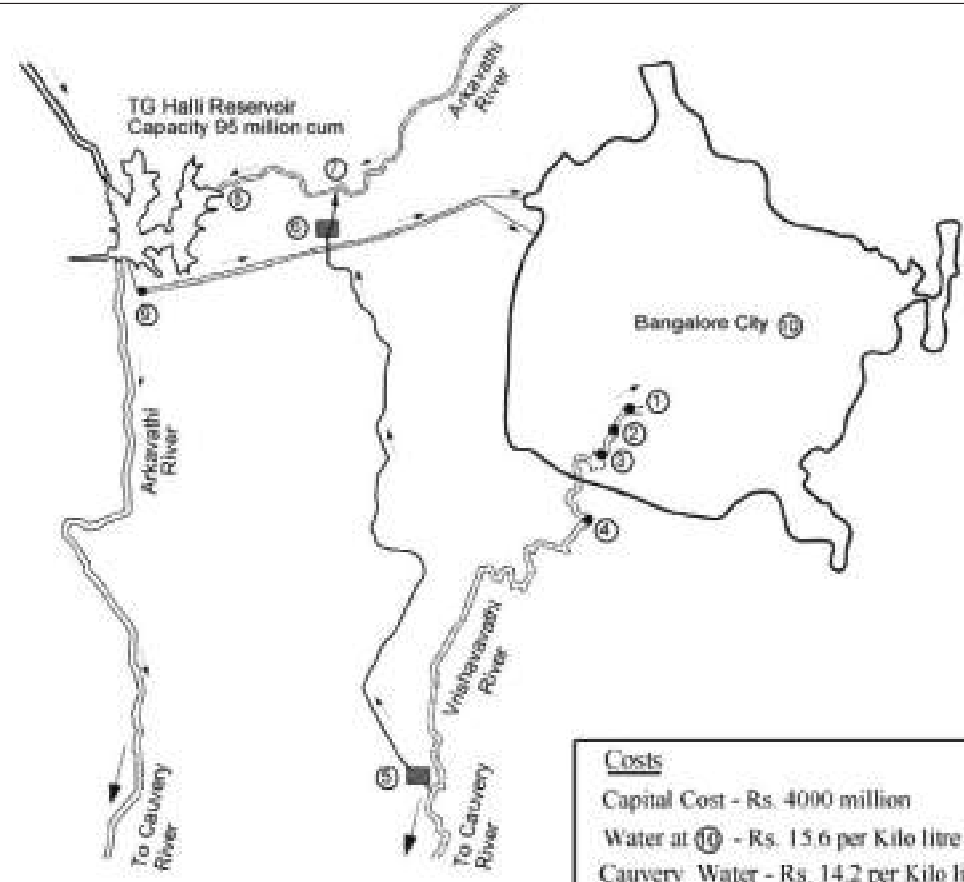
(*) Pathogen removal in high lime in Occoquan is instead met by UF in the proposed project and avoids problems of chemical sludge.

NR-1 The 2 stage carbonation as practiced in Occoquan is not required in the proposed project as the pH is not raised in the treatment.

NR-2 Hardness Removal is achieved in the Lime water treatment process in the final stage in the chain of treatment.

Source: Bangalore Water Supply and Sewerage Board, 2008

The treatment sequence adopted in the V. Valley scheme at Bangalore is more rugged and has multiple layers of sequential safety. It is too early to embark on this till the results of the Bengaluru piloting are validated.



① Virshavhavathi STP - 180 MLD
② Mailasanvra STP - 75 MLD
③ Kengeri STP - 60 MLD
④ Chodenapura STP - 40 MLD
④ - ⑤ 65 metre Cascade Fall in River over 20 Km
⑤ Water Treatment Plant at Ampapura - 152 MLD
⑤ - ⑥ 1700 mm Pumping main for 33 Km - 147 MLD
⑥ U F and R O plant at Tavarakere - 147 MLD
⑥ - ⑦ Pathogen free water to river valley - 140 MLD
⑦ Entry location into Arkavathi fresh water course
⑧ Entry into T. G. Halli fresh water reservoir
⑨ Water treatment plant with hardness removal
⑨ - ⑩ Pumping mains to Bangalore City distribution
⑩ Bangalore City

Costs

Capital Cost - Rs. 4000 million
Water at ⑩ - Rs. 15.6 per Kilo litre
Cauvery Water - Rs. 14.2 per Kilo litre

Quality Renovation by the Advanced Treatment

	Colour	Odour	B O D	C O D	N	P	TDS	pH	Pathogens
Raw Sewage	Grey	Fowl	350	750	As occurs		650	7.2 - 7.5	As occurs
Water at ⑩	Clear	Fresh	Nil	70	Nil	Nil	275	6.7 - 7.3	Nil

Status of Bangalore Water Demand and Availability

Year	Population million	Million Liters Daily (MLD)		
		Demand	Availability	Shortage
2001	5.4	870	540	330
2007	7.5	1219	840	379
2015	8.8	1720	1500	220
2021	10	2125	1500	625
2036	12.5	2550	1500	1050

Status of Bangalore Water Demand and Availability

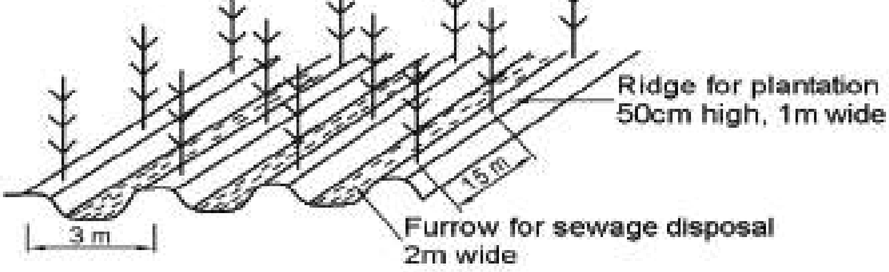
Pilot Indirect Potable Water Augmentation by 135 MLD by Advanced Reuse from City Sewage at Bangalore
Sanctioned in January 2008 under JnNURM Grant Funding for Seeking Public Acceptance Which is of course a Long Drawn Process

Figure 7.8 Schematic of treatment for reuse in Bangalore

The amount of the sewage / effluents to be disposed of depends upon the age, type of plants, climatic conditions, soil texture and quality of effluents. The total discharge of effluent is so regulated that it is consumed within 12-18 hours and there is no standing water left in the trenches. Through this technique, it is possible to dispose of 0.3 to 1.0 ML of effluent per day per hectare. This technique utilizes the entire biomass as living filter for supplying nutrients to soil and plant; irrigation renovates the effluent for atmospheric re-charge and ground storage. Further, as forest plants are to be used for fuel wood, timber or pulp, there is no chance of pathogens, heavy metals and organic compounds to enter into the human food chain system, a point that is a limiting factor when vegetables or other crops are grown with sewage.

Though most of the plants are suitable for utilizing the effluents, yet, those tree species which are fast growing can transpire high amounts of water and are able to withstand high moisture content in the root environment are most suitable for such purposes. Eucalyptus is one such species, which has the capacity to transpire large amounts of water and remains active throughout the year. Other species suitable for this purpose are poplar and leucaena. Out of these three species, eucalyptus seems to be the best choice as poplar remains dormant in winter thus, cannot bio-drain effluent during winter months. However, if area is available and the volume of effluent is small, a combination of poplar and eucalyptus is the best propagation.

This technology for sewage water use is relatively cheap and no major capital is involved. The expenditure of adopting this technology involves cost of making ridges, cost of plantation and their care. This system generates gross returns from the sale of fuel wood. The sludge accumulating in the furrows along with the decaying forest litter can be exploited as an additional source of revenue. As the sewage water itself provides nutrients and irrigation ameliorates the sodic soil by lowering the pH, relatively unfertile wastelands can be used for this purpose. This technology is economically viable as it involves only the cost of water conveyance from source to fields for irrigation and does not require highly skilled personnel as well. The institute mentions this technology to be the most appropriate and economical viable proposition for the rural areas as this technology is used to raise forestry, which would aid in restoring the environment and to generate biomass. The irrigation method is shown in Figure 7.9.



Source: CSSRI, Karnal

Figure 7.9 Irrigation method for sewage fed tree plantation as per Karnal Institute

7.2.9 Mudiali (Kolkata), India

About 400 hectares of fish ponds are in use at Kolkata. Individual ponds are about 40 hectares in area and have five distinct phases covering pond preparation, primary fertilization, fish stocking, secondary fertilization and fish harvesting. The photosynthetic activity in the pond is the basis for biological purification of the sewage. Once the water turns completely green, stocking of fish is initiated and repeated several times in a year. Catla (Catlacatla), Rohu (Labeo rohita), Mrigal (Cirrhinus Mrigala) and Bata (Labeo Bata) are mainly grown in bulk for the stock consisting of mrigal. Exotic fish like Silver Carp (Hypophthalmichthys molitrix), Grass Carp (Ctenopharyngodon idella) and common carp (Cyprinus carpio) are stocked as a small percentage. However, the popularity of tilapias (Oreochromis niloticus and O. mossambicus) is increasing. Sewage is drawn at 1 to10% of the total volume of water in the pond at intervals throughout the culture period and thereafter, continuous inflow and outflow are maintained by allowing the same level of water to flow out of the pond. Aquatic weeds like water hyacinth are grown along pond dikes of larger ponds to break waves and prevent damage to dikes. In addition, these weeded areas, provide shelter to fish when the temperature rises, prevent poaching of fishes to some degree and most importantly serve as filters to extract nutrients and metals from the system. When these weeds grow in excess, they are periodically harvested and decomposed in the pond to enhance fertility of water. In sewage fed farms, bacterial diseases are not common. Even when there were problems with Epizootic Ulcerative Disease (EUS) in recent years with carps in other areas, carps in these sewage-fed ponds remained uninfected. However, parasitic infections by Lernea (anchor worm) and Argulus are common and there is a need to develop techniques for the control of this problem. This has been partly attributed to the good nutrition obtained from the rich plankton growth in ponds. Figure 7.10 shows the procedures in tending to the ponds as a routine and Figure 7.11 and Figure 7.12 shows the ponds and the catch.

Left to Right, Sewage fed ponds are pumped out to dry, Drying of ponds is undertaken during winter, Silt is removed at least once in three years, Dried water hyacinth is kept in heaps in the ponds for decomposition.

Figure 7.10 Stages of fish farming

Figure 7.11 Ponds and high rise buildings

Figure 7.12 Full grown & fresh harvested fish

Studies on infections carried by the fish revealed that though there were stray concerns, the fact that all fish is well cooked before eating negates any risk of ingestion. The farm is a staple supplier of edible fish to Kolkata and thus, the demand is steady throughout the year.

7.2.10 Orange County California, USA

Water scarcity is a major issue in Southern California. A scarcity of freshwater resources, combined with the threat of saline ingress from the Pacific Ocean, create an urgent need for alternative resources. As a result, an ambitious water reclamation project, “Water Factory 21” (WF21)-the first groundwater recharge project allowed in California, was started in 1971 by the Orange County Water district (OCWD). Its purpose was to create a seawater intrusion barrier by injecting a 50:50 blend of reclaimed water and other water (deep well water or imported freshwater from neighbouring river basins like Colorado River) in infiltration facilities. The recycled water was pumped to spreading basins and followed the same natural path as rainwater runoff. The water produced was of very high quality due to a multi-barrier process involving multimedia filtration. In 1977, reverse osmosis treatment (RO) was added. Considering the consistent high quality of water produced, the ratio of reclaimed water was progressively increased, and in 1991 the WF21 obtained a permit to inject from 67% to 100% reclaimed water. The plant produced 57,000 m^3/d of reclaimed water.

In 1997, the OCWD launched a new project, using membrane technology: the Groundwater Replenishment System (GWRS). As a result, WF21 was shut down to allow the construction of an improved and larger high-tech purification plant, called the Advanced Water Purification Facility (AWPF). The new plant started operation in January 2008, which used a multi-barrier process involving microfiltration (MF), RO and UV and hydrogen peroxide disinfection, and produced up to 265,000 m^3/d of near-distilled quality water. Of this, approximately 132,500 m^3/d is pumped into injection wells to create a seawater intrusion barrier. Remaining 132,500 m^3/d is pumped to OCWD percolation basins in Anaheim where the GWRS water naturally filters through sand and gravel to the deep aquifers of the groundwater basin as in Figure 7.13 and Figure 7.14 (overleaf).

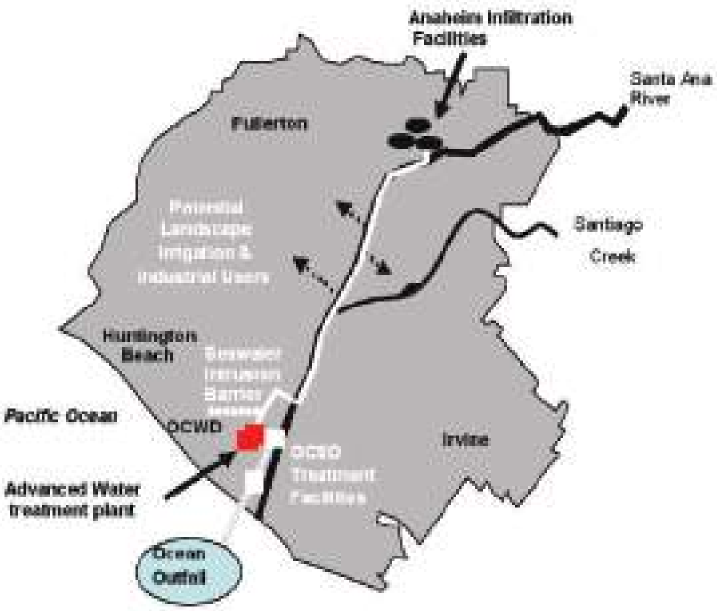


Figure 7.13 Relative locations of treatment plant and infiltration facilities at Anaheim

Source:OCWD home page

Figure 7.14 Purified water from the GWRS is piped to OCWD's percolation ponds in Anaheim, California

The AWPF has many advantages over other solutions for water production, especially compared to the old WF21 plant: the MF stage occupies less space, requires less maintenance and improves the performance of the downstream process compared to the conventional pre-treatment used at WF21. This project was accepted after a cost-benefit analysis that showed that the construction of the AWPF was the most cost-effective solution. The reclaimed water was expected to be produced for approximately 0.39$/m^3, whereas desalinated water would cost at least twice as much. Moreover, the cost of the GWRS is less than the cost of treated imported water, and a study showed that reclaimed water was 50% less energy-consuming than water importation.

The main purpose of the GWRS remains the recharge of the aquifer: it will be able to supply about 22% of the water needed to recharge the Orange County groundwater basin in the year 2020, which is forecast to reach over 500 million m^3/y. Groundwater recharge will limit seawater intrusion and thus, improve regional water quality by lowering salinity in the water supply, especially since this high quality water will contain less dissolved solids than imported water from the Colorado River. Furthermore, the GWRS will be used to reduce peak-flow under wet weather conditions and the excess flow can be diverted through the GWRS. As a result, there is no immediate need for a new ocean outfall, which is beneficial both from an economic and environmental point of view (Mediterranean Wastewater Reuse Working Group, 2007).

7.2.11 Tokyo Metropolitan, Japan

In Tokyo, municipal sewerage service covers almost the whole area (2,187km^2) and 5.5 million m^3 sewage is treated in a day. Tokyo Metropolitan Government (TMG) is now promoting the reuse of wastewater for toilet-flushing by area-wide water recycling system. In this system, the secondary effluent of municipal sewer system is treated by tertiary or advanced process and reclaimed water is supplied to buildings for toilet-flushing use.

In 1984, a model business of area-wide water recycling system was started, which supplied reclaimed wastewater to commercial buildings in Shinjuku for toilet-flushing use (Figure 7.15).

Source:Tokyo Metropolitan Government webpage

Figure 7.15 Schematic of recycling system in Shinjuku

This project is the first milestone of area-wide water recycling system in Japan. Now, 4,000 m^3 of secondary effluent is treated by rapid sand-filtration system in Ochiai Wastewater Treatment Plant and supplied to 28 high-rise buildings. This type of area-wide recycling system is continuously introduced into bay side redeveloped area. In 2006, approximately 3 million m^3 of reclaimed water (daily average amount was 8,400 m^3) was produced at three STP and supplied to 129 buildings in five areas and two more areas will be added into supply plan. To promote this type of water reuse further, TMG asks owners of buildings to install dual pipe systems when they construct large buildings having a certain scale.

In addition, Ochiai STP also discharged tertiary treated wastewater by rapid sand-filtration system to urban rivers (Meguro River) to the amount of 110,000 m^3/day in 2005. Reclaimed water is also distributed to artificial streams or ponds in adjacent parks (after RO treatment), industries, incineration plants of domestic waste, a railway company, and tanks for fire fighting use. In order to improve colour and odour of reclaimed water, TMG had developed a reclamation system with "Ozone-resistant membrane". This system is composed of pre-ozonation, bio-filtration, ozonation and micro-filtration after secondary treatment (*Yamada, et al*).

7.2.12 Restoration of Meguro River, Japan

The Meguro River, which flows through a residential area in Tokyo, had been abandoned by residents due to the decreasing flow of water and increasing pollution with an unpleasant colour and odour due to ever increasing urbanization since the Meiji Period. In order to restore river water quality and biodiversity, the TMG used highly treated effluent from the Ochiai Water Reclamation Centre to discharge into the river. Located very close to the sub-centre of the Shinjuku area, the Ochiai Water Reclamation Centre is environment-friendly and thoroughly controlled as a water reclamation centre surrounded by residential districts.

The treatment area includes most of Nakano-ward and a part of Shijuku-ward, Setagaya-ward, Shibuya-ward, Suginami-ward, Toshima-ward and Nerima-ward, totalling 3,506 ha in area. The treatment units of the reclamation centre include grit chamber and primary sedimentation tank as preliminary and primary treatment, activated sludge process (ASP) as secondary treatment and A2O process for nutrient removal, sand and membrane filtration and UV radiation as tertiary treatment. The schematic of the sequence of various treatment units of the Ochiai Water Reclamation Centre is presented in Figure 7.16.

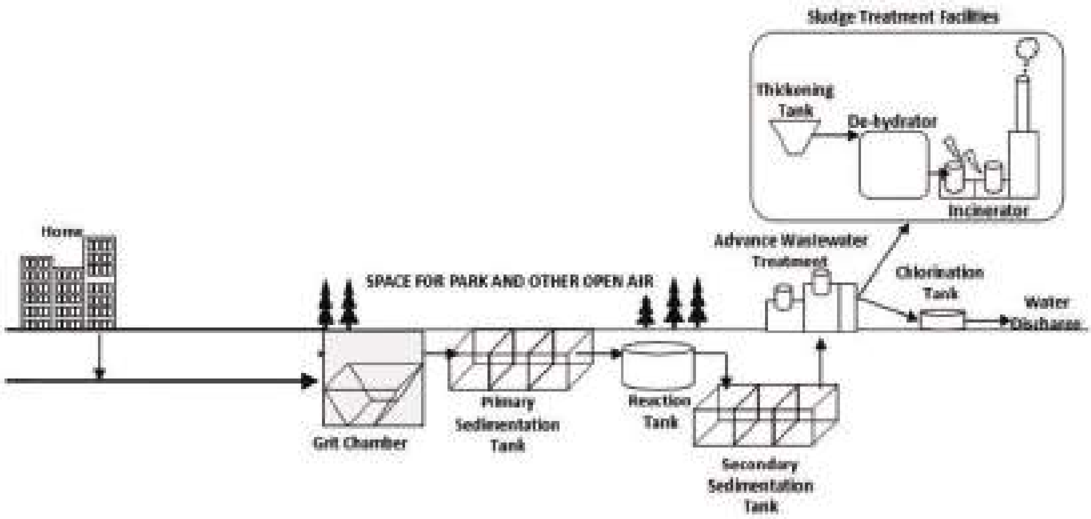


Figure 7.16 Sequence of treatment units in Ochiai Water Reclamation Facility for wastewater reclamation

The highly treated water (Table 7.7) is discharged for restoration of streams in Meguro River and other two rivers, which nearly dried up in the southern downtown area of Tokyo.

Table 7.7 Average influent and effluent water quality for the Ochiai Water Reclamation Facility

Parameters	Intake water		Discharge Water	Regional water quality standards
	Low stage	High stage	High stage	
BOD_5 (mg/l)	220	190	1	25 or below
COD (mg/l)	92	92	7	-
TN (mg/l)	31.7	27.9	11.5	30 or below
TP (mg/l)	3.7	3.0	1.5	3.0 or below

Source: Tokyo Metropolitan Government

Some parts of the treated water is used effectively for flushing water in toilet in buildings of Nishi-shinjuku and Nakano-sakaue districts. The generated sludge is pumped through pressure pipelines to Tobu sludge plant for treatment. With the drastic improvement in water volume and quality, various living species have returned to the Meguro River. The condition of the Meguro River before and after restoration is shown in Figure 7.17 (overleaf).

Source:Tokyo Metropolitan Government

Figure 7.17 Condition of Meguro River (a) before and (b) after the restoration using reclaimed wastewater

Many insects and small animal populations have been re-established, and fish such as Japanese trout, striped mullets and gobies also returned to the river after the introduction of highly treated water. The biodiversity and environmental amenities have thus, been restored effectively with wastewater reuse.

7.2.13 Road Washing and Subway Coach Cleaning, Republic of Korea

Around 16.07 billion tons of treated water is produced annually from 4 sewage treatment plants (Jungnang, Nanji, Tancheon, Seonam) in Seoul. Out of this amount, around 48.7 million tons are reused and Seoul is planning further to extend the scope of reusing as in Figure 7.18.

Source:Tokyo Metropolitan Government, Bureau of Sewerage webpage

Figure 7.18 Water from Jungnang sewage treatment plant reused for road cleaning

Around 16,546,000 tons of treated water from Jungnang sewage treatment plant is used as cleaning or wiping water every year within the plant. About 19,000 tons is provided to be used for cleaning roads outside the plant. Every year 82,000 tons of water from the plant is used at nearby subway coach depot for cleaning of coaches as in Figure 7.19.

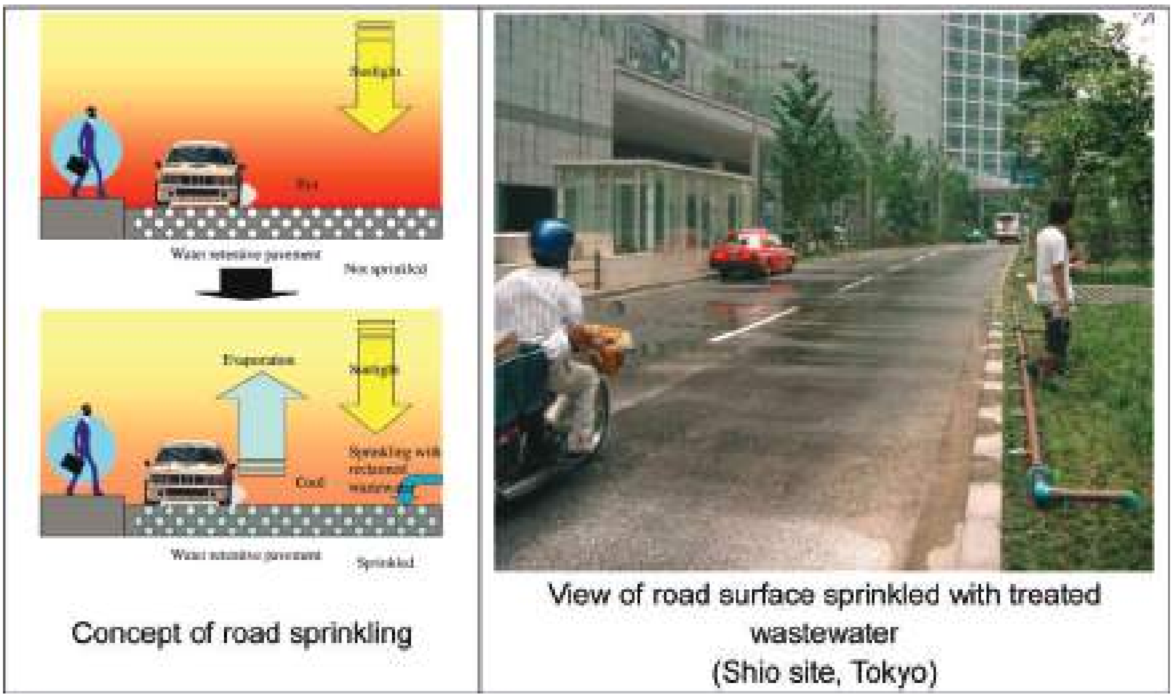
Source:Tokyo Metropolitan Government, Bureau of Sewerage webpage

Figure 7.19 Highly treated sewage used at subway coach depot for cleaning

7.2.14 Ground Cooling, Japan

In Japan, reclaimed wastewater has been recycled widely for non-potable urban applications such as toilet flushing, landscape irrigation, cleaning roads and snow melting. The government policy of “Sewerage Vision 2100” suggested creating sound water cycles by using reclaimed wastewater effectively. In this respect, attention has focused on a new way of using reclaimed wastewater; sprinkling it on roads for heat island mitigation as in Figure 7.20 (overleaf).

The term “heat island” refers to urban air and surface temperatures that are higher than nearby rural areas. Heatstroke is one of the impacts caused by the heat island. “The Policy Framework to Reduce Urban Heat Island Effect” was set in March 2004 including countermeasures such as: reduction of anthropogenic heat; urban surface improvement to regain natural cooling effect of soil and plant evapotranspiration; improvement of urban structure with green and water network to reduce the heat from the city; and promotion of energy saving life. Sprinkling reclaimed wastewater or other unused water on roads in urban areas is one of the means of improving urban surface in summer.



Concept of road sprinkling

View of road surface sprinkled with treated wastewater (Shio site, Tokyo)

Source:Yamagata et al., 2008

Figure 7.20 Road sprinkling with treated wastewater

As a pioneer, treated wastewater is sprinkled on the water retentive pavement that can store water inside paving material at Shiodome Land Readjustment District (Shio Site) in Tokyo. Research projects have been carried out there on the heat island mitigation by sprinkling reclaimed wastewater on roads. Treated wastewater from plants using advanced processes such as biological filtration, ozonation, or microfiltration, is used for spreading on the roads (Yamagata, et al, 2008).

It was observed that sprinkling treated wastewater on water retentive pavement decreased the road surface temperature by 8 degrees during the daytime and by 3 degrees at night and the road surface temperature was decreased to same level as that on planting zones. Especially, the decrease of road surface temperature continued all the night by sprinkling water in the evening.

7.2.15 Singapore NEWater

The island of Singapore suffers from serious issues of water scarcity, as Singapore's domestic resources only meet about 50% of its needs and purchases water from nearby Malaysia. Water reuse has always been an important component of Singapore's water management: reuse of tertiary quality effluents for industrial activities began in the early 1970s. Nowadays, water recycling is part of the government's "Four Taps Strategy" to ensure a sustainable water supply by diversifying its water resources: imported Malaysian water, seawater desalination, collection and treatment of local surface run-off and water reuse.

Singapore's Public Utility Board (PUB) manages the nation's water resources. In 1998, the PUB launched a joint initiative to determine the suitability of reclaimed water, which underwent advanced treatment processes of multi-barriers filtration and ultraviolet (UV) disinfection as a source of raw water for drinking water production.

A 10,000 m^3/d demonstration plant was built in 2000, and its successful performance resulted in the launch of the NEWater project. At that time, two full-scale plants were launched within one week of each other: Kranji (40,000 m^3/d) and Bedok (32,000 m^3/d) as in Figure 7.21.

Source:Mediterranean Wastewater Reuse Working Group, 2007

Figure 7.21 Map showing location of NEWater facilities in Singapore

In 2001, PUB officially named "NEWater" the recycled water produced by the reclamation plants. In 2003, potable and non-potable production of NEWater was officially opened with direct supply from three NEWater plants at Bedok and Kranji with a combined capacity of 72,000 m^3/day. In 2004 and 2007, two new reclamation plants were opened. They are Seletar (24,000 m^3/d) and Ulu Pandan (116,000 m^3/d) respectively.

Most of the NEWater is used for non-potable applications in the wafer fabrication/ microelectronics industry. Concerning indirect potable reuse (IPR) (through recharge of the freshwater catchment reservoirs) it consists currently of 1% of the total volume of water consumed daily. The NEWater produced is blended with the raw water in the surface reservoirs. Then this water undergoes conventional water treatment to produce drinking water. Only a small part (1%) of tap water contains diluted, blended and treated NEWater. Two programs have also been undertaken to assess the quality of water being produced. They are Sampling and Monitoring Program (SAMP) and Health Effects Testing Program (HETP).

7.3 GUIDING PRINCIPLES FOR INDIA

Though the possibilities of using treated sewage for various uses in other parts of the world are inspirational, still a blanket adoption in India needs to be tempered with local factors of affordability, sustainability and above all public acceptance. Moreover, though there are exotic quality guidelines brought out for specific applications, the inheritance of these may not be pragmatic in that some of the parameters mentioned there are not easily decipherable leave alone being carried out.

Moreover, each situation needs to be evaluated on its own and beyond the secondary treatment, all technologies are necessity driven are discussed hereunder and much less the treatment chain is utility purpose driven. As such, there is a need to have a set of guidelines for the mentioned reuse prospects.

7.3.1 Agriculture

7.3.1.1 Key Principles

Following key principles should be paid attention before deciding use of treated sewage for agriculture:

a) Being an agrarian economy, this is a very compelling use for India, but should never be used for edible crops or plants that produce millets, etc.

b) The use of untreated sewage for whatever form of agriculture leads to a situation where the treated sewage entering another basin from its parental basin creates issues of water rights and as far as possible, inter basin transfer of such reuse are not to be encouraged

c) Agricultural use being more pertinent in rural settings, local sewage is best treated with stabilization ponds followed by maturation ponds

d) Rotational crop pattern shall be investigated for an all the year round utilization and designed such that the runoff of treated sewage in summer is minimized

e) As far as possible, manual direct handling shall be avoided and field channels are better suited as compared to sophisticated drip irrigation etc

f) The discharge standards for disposal on land is prescribed by the MoEF are mentioned in Chapter 5 of Part A manual

g) Specific limitations on individual parameters when the treated sewage is to be considered for irrigation are addressed herein

The quality of water for irrigation is determined by the effects of its constituents both on the crop and the soil. The deleterious effects of the constituents of the irrigation water on plant growth can result from

(i) direct osmotic effects of salts in preventing water uptake by plants,

(ii) direct chemical effects upon the metabolic reactions in the plant and

(iii) any indirect effect through changes in soil structure permeability and aeration.

The suitability of the irrigation water is judged on the basis of soil properties, quality of irrigation water and salt tolerance behaviour of the crop grown in a particular climate. The water quality ratings along with the specific soil conditions recommended are shown in Table 7-8 (overleaf).

These limits apply to the situations where the groundwater table is always at 1.5 m below the ground level. The values have to be reduced by half, if the water table comes up to the root zone.

Table 7.8 Water quality ratings

Nature of soil	Crop to be grown	Permissible limit of Electrical Conductivity of water for safe irrigation (micro-mhos/cm)
Deep black soils and alluvial soils having a clay content more than 30%	Semi-Tolerant	1,500
Fairly to moderately well drained soils	Tolerant	2,000
Heavy textured soils having a clay content of 20-30%	Semi-Tolerant	2,000
Soils well drained internally and having good surface drainage system	Tolerant	4,000
Medium textured soils having a clay content of 10-20%	Semi-Tolerant	4,000
Soils very well drained internally and having good surface drainage system	Tolerant	6,000
Light textured soils having a clay content of less than 10%	Semi-Tolerant	6,000
Soils having excellent internal and surface drainage	Tolerant	8,000

Source: CPHEEO, 1993

If the soils have impeded internal drainage either on account of presence of hard stratum, unusually high amounts of clay or other morphologic reasons, the limit of water quality should again be deliberately reduced to half. In cases where canal irrigation exists during the lean period, treated sewage of higher electrical conductivity could be used.

7.3.1.2 Osmotic Effects

When water is applied for cultivation on land, some of it may run off as surface flow or be lost by direct surface evaporation, while the remainder can be infiltrated into the soil. Of the infiltrated water, a part of it can be for consumptive use, and part is held by the soil for subsequent evapotranspiration and the remaining surplus percolates or moves internally through the soil. The water retained in the soil is known as the ‘soil solution’ and tends to become more concentrated with dissolved constituents as plants take relatively purer water. An excessive concentration of salts in the soil solution prevents water uptake by plants. Table 7-8 mentions the permissible levels of electrical conductivity (EC) and hence, total salts in water for safe irrigation in the four types of soils. It may be pointed out that good drainage of the soils may be a more important factor for crop growth than the EC of the irrigation water, as leaching of soils results in maintaining a low level of salt in soil solution in the root zone.

7.3.1.3 Toxic Effects

Individual ions in irrigation water may have toxic effects on plant growth. Table 7-9 lists some of the known toxic elements and their permissible concentration in irrigation waters when continuously applied on all soils and also when used on fine texture soils for short terms.

Table 7.9 Maximum permissible concentration of toxic elements in irrigation waters

Element		Maximum permissible concentration (mg/l)	
		On all soils in continuous use or acidic soils	For short term use of textured alkaline soils
Aluminium	Al	1.0	20.0
Arsenic	As	0.1	2.0
Beryllium	Be	0.1	0.5
Boron	B	0.5	1.0
Cadmium	Cd	0.01	0.05
Chromium	Cr	0.10	1.0
Cobalt	Co	0.05	5.0
Copper	Cu	0.2	5.0
Iron	Fe	5.0	20.0
Lead	Pb	5.0	10.0
Lithium	Li	2.5	2.5
Manganese	Mn	0.20	10.0
Molybdenum	Mo	0.01	0.01
Nickel	Ni	0.20	2.0
Selenium	Se	0.005	0.01
Vanadium	V	0.10	1.0
Zinc	Zn	2.0	10.0

Source: Environment Studies Board, 1973, CPHEEO, 1993

Many of these are also essential for plant growth. The suggested values for major inorganic constituents in water applied to land are presented in Table 7.10.

Table 7.11 presents the suggested limits for salinity in irrigation waters.

Table 7.10 Suggested values for major inorganic constituents in water applied to the land

Problem and Related Constituent	Impact on the land		
	No problem	Increasing Problem	Severe
Salinity			
Conductivity of Irrigation water (millimhos/cm)	< 0.75	0.75 – 3.00	> 3.00
Permeability			
Conductivity of Irrigation water (millimhos/cm)	< 0.50	< 0.50	< 0.20
Sodium absorption ratio (SAR)	< 18.0	18.0 - 26.0	26.0
Specific Ion Toxicity (from root absorption)			
Residual Sodium Carbonate (RSC), (meq/l)	< 1.25	1.25 – 2.5	> 2.5
Sodium (Na, %)	(A)	(A)	(A)
Chloride, meq/l	(B)	(B)	(B)
Chloride, mg/l	< 142.00	142.00 -355.00	> 355.00
Boron, mg/l	< 1	1– 4	> 4
Specific Ion Toxicity (from foliar absorption, sprinklers)			
Sodium (Na, %)	(C) < 40	(C) 40 - 60	(C) > 60
Chloride. meq/l	< 250	250 – 1,000	> 1,000
Miscellaneous			
NO_3(mg/l) for sensitive crops	(D)	(D)	(D)
pH	6.5 – 8.5		

Source: IS: 10500

Note :

(A) – No guidelines laid down, but increasing concentration affects soil structure and permeability

(B) - No guidelines laid down, but may have direct toxic effect with sodium

(C) – No guidelines laid down, but these are recommended values

(D) – No guidelines – it is an essential plant nutrient, but excess may delay the maturity of seed growth in some plants.

Table 7.11 Suggested limits for salinity in irrigation waters

Crop Response	TDS mg/L	EC, mhos/cm
No detrimental effects will usually be noticed	500	0.75
Can have detrimental effect on sensitive crops	500 – 1,000	0.75 - 1.50
May have adverse effects on many crops	1,000 – 2,000	1.50 - 3.00
Can be used for salt tolerant plants on permeable soils with careful management practices	2,000 – 5,000	3.00 - 7.50

Source: CPHEEO, 1993

7.3.1.4 Sodium Hazard

In most normal soils, calcium and magnesium are the principal cations held by the soil in replaceable or exchangeable form. Sodium tends to replace calcium and magnesium when continuously applied through irrigation water. An increase of exchangeable sodium in the soil causes deflocculating of soil particles and promotes compaction, thereby impairing soil porosity and the water and air relations of plants.

The sodium hazard of irrigation water is commonly expressed either in terms of percent soluble sodium (PSS) or sodium adsorption ratio (SAR).

$$PSS = \frac{100 \times Na^{+}}{Na^{+} + Ca^{++} + Mg^{++} + K^{+}}$$

or

$$= \frac{100 \times Na^{+}}{(Total\ Cations)} \qquad (7.1)$$

and

$$SAR = \frac{Na^{+}}{\left(\frac{Ca^{++} + Mg^{++}}{2}\right)^{\frac{1}{2}}}$$

where the cations are expressed as meq/l.

Generally the sodium hazard of soil increases with the increase of PSS or SAR of irrigation water and exchangeable sodium percentage of the soil. The maximum permissible value of PSS in irrigation water is 60. Where waters with higher PSS values are used, gypsum should be added to the soil occasionally for soil amendment. SAR values greater than 18 may adversely affect the permeability of soils.

7.3.1.5 Residual Sodium Carbonate

Hazardous effect of sodium is also increased, if the water contains bicarbonate and carbonate ions in excess of the calcium and magnesium. In such cases there is a tendency for calcium and magnesium to precipitate as carbonates from the soil solution and thereby increase in the relative proportion of exchangeable sodium. Values of residual sodium carbonate (RSC) less than 1.25 mg/l are considered safe and above 2.5 mg/l as unsuitable.

$$RSC = (CO_3^{2-} + HCO_3^{-}) - (Ca^{2+} + Mg^{2+}) \qquad (7.2)$$

where all ionic concentrations are expressed as mg/l. However, it has been witnessed in some arid locations that after the micro irrigation and spray irrigation on land, the bicarbonate salt of Calcium precipitates as its carbonates due to incremental water loss and heating from sun rays. In sensitive locations as lawns in recreational areas, it may be better to convert the bicarbonate to chlorides by acidification if needed. The effect of potassium on soil is similar to that of sodium, but since the concentration of potassium is generally quite small in irrigation waters, it is often omitted from consideration.

7.3.1.6 Organic Solids

While stable organic matter improves porosity of soil, thereby facilitating aeration, an excessive application of unstable organic matter would lead to oxygen depletion in the soil. Depositing of sediments especially when they consist primarily of clays or colloidal material may cause crust formations, which impede emergence of seedlings. In addition, these crusts reduce infiltration with the consequent reduction of irrigation efficiency and less leaching of saline soils.

7.3.1.7 Other Considerations

Soils are usually well buffered systems. The pH is not significantly affected by application of irrigation water. However, extreme values below 5.5 and above 9.0 will cause soil deterioration. Development of low pH values in soils promotes dissolution of elements such as iron, aluminium or manganese in concentrations large enough to be toxic to plant growth. Similarly, water having high pH values may contain high concentration of sodium, carbonates and bicarbonates, the effect of which has been discussed earlier.

Chlorides and sulphates are toxic to most crops in high concentrations. Ordinarily, the detrimental effects of salinity on crop growth become perceptible first. Excessively high or low temperature in irrigation water may affect crop growth and yields. A desirable range of water temperature is from 12 to 30°C.

7.3.1.8 Design and Management of Sewage Farms

Optimum utilization of sewage in agriculture means the complete and judicious use of its three main components, viz., water, plant nutrients and organic matter on the farms in such a way that (a) the pathogenic infection is neither spread among the farm workers, nor among the consumer of sewage farm products, (b) the groundwater is not contaminated, (c) there is maximum output per unit volume of sewage, (d) there is no deterioration of the soil properties and (e) none of the three components are wasted.

7.3.1.9 Management of Water in Sewage Farming

The principle to be borne in mind in irrigation management is to irrigate only when it is required and only to the extent it is required by the crop. The water requirement depends on the soil type, the crop and the climate. The water requirement (cm) of main soil types to be wetted to a depth of 30 cm required by most of the crops is given in Table 7.12.

Table 7.12 Water requirements (cm) to wet different soils to a depth of 30 cm

No.	Soil	Requirement	No.	Soil	Requirement
1	Sandy	1.25	4	Clay Loam	6.25
2	Sandy Loam	2.50	5	Clayey	7.50
3	Loam	5.00			

Source: CPHEEO, 1993

Water requirement of crops vary with the duration of their growing season and the amount of growth in unit time. Details for some of the Indian crops that can be grown on sewage farms are given in Table 7.13.

Table 7-13 Water requirements of crops

No.	Crops	Growing period,(days)	Total water requirement ,cm)	Optimum pH range
1	Soybean	110-120	37.50	6.0-8.5
2	Mustard	120-140	37.50-55.00	6.0-9.5
3	Sunflower (kharif)	100-110	37.50	6.0-8.5
4	Sunflower (rabi)	110-120	87.50	6.0-8.5
5	Barley	88	35.25	6.5-8.5
6	Cotton	202	105.50	5.0-6.05
7	Jowar	124	64.25	5.5-7.5
8	Maize	100	44.50	5.5-7.5
9	Linseed	88	31.75	5.0-6.5
10	Rice	98	104.25	5.0-6.0
11	Milling Varieties of sugarcane	365	237.50	6.0-8.0
12	Wheat	88	37.00	5.5-7.5

Source: CPHEEO, 1993

7.3.1.9.1 Hydraulic Loading

The elements to be considered in determining hydraulic loading are the quantity of effluent to be applied, precipitation, evapotranspiration, percolation and runoff. For irrigation systems, the amount of effluent applied plus precipitation should equal the evapotranspiration plus the amount of percolation. In most cases, surface runoff from fields irrigated with sewage effluent is not allowed or must be controlled.

The water balance.

Precipitation + Sewage application = Evapotranspiration + Percolation

Seasonal variations in each of these values should be taken into account by evaluating the water balance for each month as well as the annual balance. The irrigation requirement of any crop is not uniform throughout its growing period. It varies with the stage of growth. For example grain crops require maximum irrigation during the time of ear-head and grain formation. Sugarcane requires more frequent irrigation from about the sixth or the seventh month onwards. In case of fruit trees the irrigation has to be stopped during their resting period. If the irrigation is not given at critical growth stages of the crop, it results in lower yields.

The water requirement of crop at different stages of growth can be determined either directly (gravimetrically) or indirectly by use of tensiometers or irrometers or gypsum blocks. Normally, when there is about 50% depletion of available moisture in the soil, irrigation is recommended. The crop plants themselves show signs of moisture stress. One has to be always on the lookout for such first symptoms to determine the need for irrigation. Some plants like sunflower also serve as good indicators of stress symptoms. Sunken screen pan evaporimeter could also be used for estimating use of water by crop plant and scheduling irrigation.

The extent of irrigation depends on the depth of irrigation to be given and volume of water required for wetting the soil to the required depth. If tensiometers or gypsum blocks are embedded at the required depths, they would indicate the stage when the soil at that depth is saturated. Nearly about 70% to 80% roots of most crops are found in the first 30 cm of the soil. Some may go deeper to the next 30 cm. Normally, in irrigating medium type of soil it is wetted to about 30 cm depth or a little more. If the figures for water requirements for crops as mentioned in Table 7.13 are to be satisfied, much higher hydraulic loadings have to be applied since a portion of sewage after its passage through the soil is carried away by the sub-soil under drainage system. The extent of desirable percolation rate depends upon the salinity of the irrigation water. The applicable hydraulic loadings of settled sewage are therefore dependent upon the type of soil and the recommended rates are given in Table 7.14.

Table 7.14 Recommended hydraulic loadings

No.	Soil	Hydraulic Loading (cum/ha/day)	No.	Soil	Hydraulic Loading (cum/ha/day)
1	Sandy	200 - 250	4	Clay Loam	50 - 100
2	Sandy Loam	150 - 200	5	Clayey	30 - 50
3	Loam	100 - 150			

Source: CPHEEO, 1993

Sewage conforming to the norms should be applied to the soil by strip, basin or furrow irrigation, Wild flooding should not be adopted. Sprinkler irrigation could be used for adequately treated sewage. The distribution channels should be properly graded to avoid ponding and silting. It is advisable that the main distributary channel is lined.

7.3.1.9.2 Organic Loading

Values of 11.0 to 28.0 kg/ha/day of organic loading in terms of BOD_5 is needed to maintain a static organic matter content in the soil that helps to condition the soil by microorganisms without solid clogging. Higher loading rates can be managed depending on the type of system and the resting period. When primary effluent is used, organic loading rates may exceed 22.0 kg/ha/day without causing problems.

7.3.1.9.3 Irrigation Interval

Resting periods for surface irrigation can be as long as 6 weeks, but is usually between one and two weeks during which, the soil bacteria break down organic matter and the water is allowed to drain from the top few centimetres, thus restoring aerobic condition in the soil. It depends upon the crops, the number of individual plots in the rotation cycle and management consideration.

7.3.1.10 Management of Soil

A well-planned programme of crop growth and harvesting can help to maintain a soil receptive to effluent application. Crop uptake of nutrients followed by removal of the crop from the field increases the capacity of the land for removal of nutrients from the next effluent application. It is necessary that the soil is given rest for about 3 to 4 months every alternate or third year preferably in summer months. This can be achieved if the farm is designed on the basis of water requirement in the winter season. After the harvest of the crop, the soil may be opened up by deep ploughing and cultivated appropriately to make it as porous and permeable as possible before the next crop is raised.

The maintenance of soil oxygen level is very important as it is required for root respiration and a number of biological processes in the soil. Refilling of oxygen in the pores in the surface layers of soil depends upon the reestablishment of contact of the soil with the atmosphere. This process can be accelerated by suitable cultural practices and by providing sufficient irrigation intervals. It is therefore, desirable that an intercultural operation is followed as soon as the soil condition allows working after every irrigation. It should always be seen that the soils of sewage farm should have a surplus of oxygen than that normally required in the ordinary farm because the soil oxygen has to perform an additional job of satisfying the BOD of sewage.

The intercultural operation following one or two irrigations is all the more necessary in the case of clayey soil. In areas where rainfall is low, it is desirable to flood the soils with irrigation water at least once a year to leach down the salts accumulated in the soil. If the soil salinity and alkalinity pose a serious problem, amendment of soil with the required quantity of gypsum should be carried out. Subsoil drainage is very important. Poor drainage should be improved by installing underground drains. Sewage farm fields must be laid out in accordance with the natural slope of the terrain to eliminate the irregularities of distribution.

On sewage farms, no sewage should be allowed to flow beyond the farm boundaries. With this in view, protection banks are arranged along the lowest lying boundaries of each crop rotation field.

7.3.1.11 Nutrients Loading

Sewage contains 26-70 mg/1 of nitrogen (N), 9-30 mg/l of Phosphate (P_2O_5) and 12-40 mg/l or even more of potash (K_2O). The recommended dosages for N, P and K for majority of field crops are in the ratio of 5:3:2 or 3 respectively. The figures for N, P, and K contents of sewage on the other hand show that sewage is relatively poor in phosphates. Excess potash is not of significance, but a relative excess of nitrogen affects crop growth and development.

Crops receiving excessive dosage of nitrogen show superfluous vegetative growth and decrease in grain or fruit yield. The phosphate deficit of sewage, therefore, should be made good by supplementing with phosphate fertilizers, the extent of phosphate fortification depending upon the nature of crop and its phosphate requirements. As the availability of phosphate is low in the irrigation water it would be desirable to apply the required quantity of phosphatic fertilizer at the time or even (about a fortnight) before the sowing or planting of the crop.

Even when sewage nutrients are balanced by fortification, irrigation with such sewage may supply excessive amount of nutrients resulting in waste or unbalanced growth of plants with adverse effects on yields. It may therefore be necessary to dilute the sewage. Dilution also helps in reducing the concentration of dissolved salts and decomposable organic matter in the sewage thus, decreasing hazards to the fertility of the soil. It is desirable to limit the BOD and total suspended solids of sewage to be disposed on land for irrigation, as per relevant standards. There is a need to take caution on describing nutrient supply capacity of sewage particularly in the case of availability of phosphorus because there is a possible conversion of available phosphorus in unavailable mode in the presence of heavy metals present in the sewerage. This happens commonly in high as well as low pH soils.

7.3.1.12 Land Requirements for Hydraulic and Nitrogen Loadings

The field-area requirement for farming based on the hydraulic loading rate is calculated by:

$$A = [3.65\ Q/L] \tag{7.3}$$

where

A = Field-area in hectares

Q = Flow rate in cum/day

L = Annual liquid loading, cm/year

For loading of constituents such as Nitrogen

$$A = [0.365\ CQ/L_c] \tag{7.4}$$

where

C = Concentration of the constituents, mg/l.

Lc = Loading rate of the constituent, kg/ha/year.

7.3.1.13 Alternative Arrangement during Non-irrigating Periods

During rainy and non-irrigating seasons, sewage farm may not need any water for irrigation. Even during irrigating season, the water requirement fluctuates significantly. Hence, satisfactory alternative arrangements have to be made for the disposal of sewage on such occasions either by storing the excess sewage or discharging it elsewhere without creating environmental hazards. The following alternatives are generally considered:

a) Provision of holding lagoons for off-season storage. They enable irrigation of a fixed area of land to varying rates of crop demand. They may also serve as treatment units such as aerated or stabilization lagoons, provided the minimum volume required for treatment is provided beyond the flow-balancing requirement.

b) Provision of additional land where treated sewage is not required on the main plot of land

c) Discharge of surplus treated sewage to river or into sea with or without additional treatment. Combining surface discharge facilities with irrigation system is quite common and often quite compatible.

d) Resorting to artificial recharge in combination with an irrigation system where feasible.

7.3.1.14 Protection against Health Hazards

Sewage farms should not normally be located within 1 km of sources of centralized water supply, mineral springs in the vicinity, where water bearing layers prevail; or on areas with groundwater levels less than 2 m below the surface. Measures should be taken to prevent pollution of artesian water. Sewage farms must be separated from residential areas by at least 300 m horizontal distance. The public health aspects of sewage farming should be considered from the viewpoints of exposure of farm workers to sewage and that of the consumers to the farm products.

Evidence is on the increase to show that labourers working on the sewage farms suffer from a number of aliments directly attributed to handling of sewage. In view of this it is desirable to disinfect sewage and where feasible mechanize sewage farm operation.

Sewage of individual enterprises engaged in the processing of raw material of animal origin or hospitals, bio-factories and slaughter houses should in addition be disinfected before they are taken to the sewage farms. Agricultural utilization of sewage containing radioactive substances is to be guided by special instructions.

The staff of sewage farms must be well educated in the sanitary rules on the utilization of sewage for irrigation as well as with personal hygiene. All persons working in sewage farms must undergo preventive vaccination against enteric infections and annual medical examination for heliminthoses and be provided treatment if necessary.

Sewage farms should be provided with adequate space for canteens with proper sanitation, wash-stands and lockers for irrigation implements and protective clothing. Safe drinking water must be provided for the farm workers and for population residing within the effective range of the sewage farms.

All farm workers should be provided with gum boots and rubber gloves, which must compulsorily be worn while at work. They should be forced to observe personal hygiene such as washing after work as well as washing before taking food. The use of antiseptics in the water used for washing should be emphasized. The farm worker should be examined medically at regular intervals and necessary curative measures enforced.

Cultivation of crops that are eaten raw should be banned. Cultivation of paddy in bunded fields is likely to give rise to sanitation problems and hence is undesirable. Growing of non-edible commercial crops like cotton, jute, fodder, milling varieties of sugarcane and tobacco would be suitable. Cultivation of grasses and fodder legumes, medicinal and essential oil yielding plants like menthal and citronella may be allowed. Cultivation of cereals, pulses, potatoes and other crops that are cooked before consumption may be permitted, if sewage is treated and care is taken in handling the harvests to ensure that they are not contaminated. Cultivation of crop exclusively under seed multiplication programmes would be advantageous as these are not consumed. As an additional safeguard, sewage irrigation should be discontinued at least two months in advance of harvesting of fruits and berries, one month for all kinds of vegetables and a fortnight for all other crops. Direct grazing on sewage farms should be prohibited.

7.3.2 Guiding Principles — Farm Forestry

Much of the provisions in Section 7.3.1 shall apply here also except that the SAR and RSC criteria may not be of serious importance. Besides, the non-water needs in rainy periods are to be borne in mind for diverting the treated sewage away from the farm forestry. It will be a better proposition to carry out all treatments at the STP itself and not split between STP and farm.

7.3.3 Guiding Principles - Horticulture

Same as in Section 7.3.2 above, except that the TDS limit shall not exceed the TDS limit of the groundwater at any time and even if the RSC limitation is met, the alkalinity to be moved from bicarbonates to chlorides or sulphates to prevent scaling of the tender leaves and petals in high summer and also choking the soil pores by evaporation of the temporary hardness.

7.3.4 Guiding Principles - Toilet Flushing

Considering that the Indian water closets when flushed can sprout and splash the flush water above the rim and onto the foot rest areas, it is necessary that such reuse shall be only after activated carbon and ultra filtration membranes. It shall not be made mandatory in layouts and confined condominiums and multiplexes and encouragement and persuasion shall be adopted, than a collision course or mandating it which is not justifiable by any means for if nothing else, sentimental reasons which rule high in Indian way of life. Similarly, small layouts being mandated to provide STP is to be viewed as decentralized sewerage and the sustainability of these by the proposed number of plot owners shall be assessed before sanctioning them, as otherwise, the policy of septic tanks on-site followed by twin drain shall be encouraged as a practical possibility. In any case, small layouts shall not be forced to erect reuse practices as absence of proper O&M can only create a mini epidemic of sorts.

7.3.5 Guiding Principles - Industrial and Commercial

The reclaimed water from the sewage renovation plant of M/S MFL is reported to have been demonstrated for its quality by the engineer drinking it himself before the team of the World Bank during 1995. However, it shall not be accepted as an endorsement of drinkability. All the same, It should be accepted as a statement of the capability, in the country to build a plant of such advanced technology and encourage similar widespread uses in other industrial sectors for non-human contact reuse. Industrial reuse of treated sewage includes the following:

- Once through cooling water
- Recirculating evaporative cooling water
- Boiler feed water
- Non-human contact process water
- Irrigation of landscape around industrial plants

Once through Cooling Water:

In addition to recommended surface water discharge standards, > 1 mg/l residual chlorine can serve the purpose (US EPA 2004).

Re-circulating or Evaporative Cooling Water:

In addition to the once through cooling standards, additional criteria are salt build-up that is discussed in the following section. Additional treatment is usually provided to prevent scaling, corrosion, biological growths, fouling and foaming (US EPA 2004).

Boiler Feed Water:

It requires extensive treatment to reduce hardness and even demineralization. Hence, RO or ion exchange process with suitable pre-treatment are required to achieve boiler feed water quality for high pressure boilers.

Process Water:

It depends on the quality of process water required by specific industry on a case-by-case basis.

Irrigation and Maintenance of Landscape around Industrial Plants:

Please refer the guiding principles discussed in above sections.

7.3.5.1 As Cooling Water

Reuse as cooling water is one of the most common industrial applications of reclaimed treated sewage. Typical guidelines for cooling water quality are given in Table 7.15 (overleaf) and may be used where specific requirements are not given.

Table 7.15 Cooling water quality guidelines

No.	Parameter/Condition	Recommended value
(A)-In make-up water		
1	pH	6.8-7.0 (variation less than 0.6 units in 8 hours
2	Average TDS value (with variation + 25% permissible on 8 hour average)	Cycles of concentration in re-circulating water
	3,000 mg/l	2.0
	1,000 mg/l	3.5
	500 mg/l	6.0
3	Oil & grease	Absent
4	BOD (5day, 20°C)	Less than 5.0 mg/l
5	Chlorides (Cl)	Less than 175 mg/l
6	Ammonia	No appreciable amount
7	Caustic alkalinity	Absent
8	Methyl orange alkalinity (as $CaCO_3$)	Less than 200 mg/l
(B)-In re-circulating water		
9	Silica (As SiO_2)	Less than 150 mg/l
10	Phosphates, sulphates	Not to exceed solubility limit in re-circulating water
11	Alkyl Benzene Sulponate (ABS)	Foam not to persist more than 1 minute after 10 seconds of vigorous shaking of re-circulating water
12	Langelier index at skin temperature of heat exchange surface	0.5±0.1
13	Ryzner Stability Index	6.0 to 7.0

Source:CPHEEO, 1993

To determine the quality and quantity of water required for reuse in a cooling system, where an open re-circulating system is adopted for air conditioning cooling water, the amount of water to be kept for re-circulating in the system is approximately 11 litres/min for every ton of refrigeration capacity when the temperature drop is 5°C in the cooling tower. For such a situation, the water lost in evaporation (E) is about 1% of the re-circulating water.

Windage loss (W) is of the order of 0.1 to 0.3% of the recirculating water when mechanical draft towers are used, but increases to 0.3 to 1.0% for atmospheric towers. Blow down requirement (B) is estimated from the following equation if the maximum permissible cycles of concentration (C) are known

$$B = \frac{E + W(1 - C)}{C - 1} \qquad (7.5)$$

where, B, E and W are all in lpm.

For trouble free operation and minimum use of water quality control chemicals in the recirculating water, the cycles of concentration are generally kept at 2.0 to 3.0 and, in no case, more than 4.0 in cooling towers where reclaimed water is used.

The quality guidelines for cooling water are included in Table 7 15. Hence, for a 100-ton air-conditioning plant recirculating 1100 litres/min of water with a temperature drop of, say 10°C through a mechanical draft tower where cycles of concentration are to be restricted to 2.0

E = 2% × 1100 = 22 litres/min

W = 0.2% × 1100 = 2.2 litres/min

$$B = \frac{22 + 2.2 \times (1-2)}{2-1} = 20\ liters/\min\ (approximately)$$

The total make-up water requirement thus equals 44.2 litres/min (= 22 + 2.2 + 20) or 63.4 m^3/day for 24 h working of a 100-ton plant.

Similarly, if 3.0 cycles of concentration are permissible, the total requirement of make-up water reduces to 47.7 m^3 /day for a 100 ton plant.

When cycles of concentration equal 3.0, the various stable constituents (e.g. chlorides) in make-up water are theoretically increased by a factor of 3.0 in the re-circulating water. If the concentration of various constituents in the make-up water lies within the range of values given in column (A) to (F) of Table 7.16 the corresponding concentration in the recirculating water can be readily estimated. For example, if Cl is 60 mg/l in the make-up water, they will increase to 180 mg/l in the re-circulating water. However, the pH of the re-circulating water cannot be estimated in this manner.

The assumption is frequently made that in the absence of phenolphthalein alkalinity, the pH of the water leaving the cooling tower will be between 8.0 and 8.3, due to elimination of free carbon dioxide in the tower. Sometimes, for other reasons, a lower or higher pH may be observed. Thus knowing the pH, the concentrations of calcium, alkalinity and total dissolved solids in the re-circulating water and the temperature in the hottest part of the system, one can determine the Langelier index and Ryzner stability index and the tendency of the water to scale or corrode. Assuming that the recirculating water shows the tendency for deposition of scale, reduction in hardness and in alkalinity is the usual means of control. Since nothing can be done to reduce temperature, reduction in total solids would not have much effect on the Index

For this reason, partial zeolite softening (by blending the softened water with by-passed hard water), plus acid feeding if required for reduction of alkalinity provide a relatively simple and flexible means of preventing excessive scaling in this type of installation.

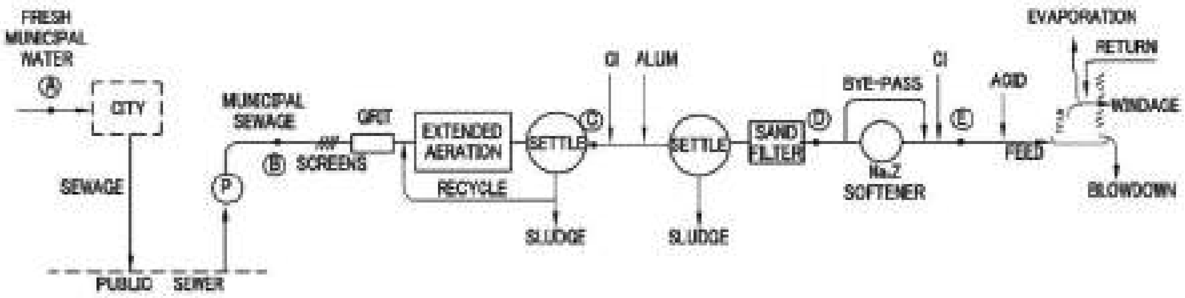
The blending ensures a certain amount of hardness in the water, which is useful to protect against corrosion of ferrous heat exchanger surfaces. The acid treatment (using H_2SO_4) depends for its functioning on the fact that calcium and magnesium sulphates are much more soluble than the carbonates, with the usually adopted dosages and the cycles of concentration obtaining in the system, calcium sulphate concentrations obtaining in the system.

Calcium sulphate concentrations are well below the solubility limit. Similarly, calcium phosphate is also kept within the solubility limit.

Automatic dosing and control equipment is normally not provided in plants in India. The clear water storage tanks helps to maintain uniformity of quality of water pumped to the cooling towers. Storage ensures that pH, total dissolved solids, etc., do not vary much from hour to hour and the wide variations in inflow quantities are balanced out.

Pre-chlorination is done as the water enters the coagulation tanks, while post-chlorination is mainly in the form of periodic “shock” doses to control lime and algal growths. The latter are likely to form owing to the presence of nitrates and phosphates in the treated water and the warm and sunny climate of India.

A typical flow sheet for making sewage water fit for reuse as once through cooling water is given in Figure 7.22.



Source:CPHEEO, 1993

Figure 7.22 Flow sheet for treatment of municipal sewage for reuse as cooling tower make-up water

For reuse as make up water in cooling systems, the treatment processes given earlier in Figures 7-1 to 7-6 are to be evaluated and suitable scheme evolved. The removal of ammonia by air stripping is not to be used as this leads to air pollution issues. Instead biological nitrification-denitrification could be used.

Table 7.16 (overleaf) gives an illustrative example of the change in water quality as fresh municipal water becomes sewage and is gradually renovated for reuse as cooling and process water.

Where nitrates and phosphates in the make-up water are necessary to be reduced, the biological treatment given to sewage at the secondary stage can itself be modified to include nitrification-denitrification and the addition of alum or ferric done in the final settling tank or add on tertiary high lime treatment with recarbonation is opted to precipitate phosphates.

If high lime is used, it will also knock out silica in proportion to mg and can be enhanced in removal by adding dolomite to increase the Mg content.

7.3.5.2 As Boiler Feed Water

Reuse as boiler feed water may require additional treatment over that required for cooling purposes. As boiler feed, the quality of water depends on the boiler pressures at which steam is to be raised. The higher the boiler pressure, the purer the water required.

Table 7.16 Water quality changes as freshwater becomes sewage and is gradually treated for reuse (illustrative example)

			Water quality at different treatment steps			
Item	Fresh municipal water	Raw domestic sewage	After biological treatment	After coagulation and filtration	After softening & chlorination	After DS and DM
	(A)	(B)	(C)	(D)	(E)	(F)
pH	7.6-7.8	7.15-7.65	7.2-7.8	7.1-7.3	7.1-7.2	≈7.00
Total Hardness mg/l as $CaCO_3$	35-40	120-160	120-160	120-170	40 (a)	As arising
M.O. Alkalinity mg/l as $CaCO_3$	40-45	125-200	125-200	110-180 (b)	110-180	As arising
Chlorides mg/l, as Cl	15-20	60-130	60-120	60-130	60-130	As arising
Sulphates mg/l as SO_4	1.5-2.5	10-20	10-15	15-25	15 -25	As arising
Phosphates mg/l as PO_4	Traces - 0.1	6-16	3-10	0.2-0.5	0.2-0.5	Nil
Nitrates mg/l as NO_3	1.0-2.0	1.0-3.0	13-19	13-19	13-19	As arising
Silica mg/l as SiO_2	8-24	10-24	10-24	10-20	10-20	As arising
Dissolved solids mg/l	80-500	850–1,350.	850–1,350	800-1,350	600-1150	< 500
Suspended solids mg/l	5-10	350-450	15-30	Nil.	Nil.	Nil
Turbidity SiO_2, Units	5-10	Turbid	10-20	2.0-3.0	2.0-3.0	2.0–3.0
BOD, mg/l	0.1-1.5	200-250	6-10	1.0- 2.0	1.0-1.5	Nil
COD, mg/l	1.0-2.0	350-500	150-270	150-270	100-180	< 70
Bacteriological quality (as per coliform standards)	Safe	Unsafe	Unsafe	Safe	Safe	Safe
Specific conductance						700

a) Softened water is blended with un-softened water to give a final hardness of 40 mg/l as in fresh municipal water.

b) Alkalinity is reduced by acid treatment just prior to use in cooling towers. This increases sulphate content somewhat since H_2S0_4 is used.

Source: CPHEEO, 1993

Table 7.17 gives an indication of the water quality required for low and medium pressure boilers.

Characteristic	Requirement for Boiler pressure			Method of test (Ref to clause	
	Up to 2.0 Nm/m^2	2.1 to 3.9 Nm/m^2	4.0 to 5.9 Nm/m^2	IS: 3550 (a)	IS: 3025 (b)
Feed water					
a) Total hardness (as $CaCO_3$) mg/l, max	1.0	1.0	0.5		16.1
b) pH Value	8.5 to 9.5	8.5 to 9.5	8.5 to 9.5		8
c) Dissolved oxygen mg/l, max	0.1	0.02	0.01	25	
d) Silica (as SiO_2) mg/l, max		5	0.5	16	
Boiler water					
a) Total hardness (of filtered sample) (as $CaCO_3$) mg/l, max	Not Detectable				16.1
b) Total alkalinity (as $CaCO_3$) mg/l, max.	700	500	300		13
c) Caustic alkalinity (as $CaCO_3$)mg/l, max	350	200	60		15
d) pH value	11.0 to 12.0	10.0 to 12.0	10.5 to 11.0		8
e) Residual sodium sulphite (as Na_2SO_3) mg/l	30 to 50	20 to 30			21
f) Residual Hydrazine (as N_2H_4) mg/l	0.1 to 1 (if added)	0.1 to 0.5 (if added)	0.5 to 0.3	26	
g) Ratio Na_2SO_4 caustic alkalinity (as NaOH)		Above	2.5		20.2 and 15
Or					
Ratio $NaNO_3$ total alkalinity (as NaOH)		Above	0.4		48 and 13

a) Methods of test for routine control for water used in Industry

b) Methods of sampling and test (Physical and Chemical) for water used in Industry.

Source: CPHEEO, 1993

For low pressure boilers, the quality of water required is more or less similar to that for reuse in cooling purposes. For high pressure systems, the treatment required can be quite substantial as can be seen from the water requirements given in Table 7.18 (overleaf).

Table 7.18 Requirements for feed water, boiler water and condensate for water - tube boilers (drum type)

Characteristic	Requirements for boiler pressure mm/m^2 (in the drum)				Method of test to Clause No. of		See also Clause
pH	6.0-7.8	7.9-9.8	9.9-11.8	Above 11.8	IS: 3550 (a)	IS: 3025 (b)	
Total hardness(1)	Nil	Nil	Nil	Nil		16.1	
pH value	8.5-9.5	8.5-9.5	8.5-9.5	8.5-9.5		8	2.1.1(a)
Oxygen, O_2 mg/l, max	0.01	0.005	0.005	0.005	25		2.1.1(b)
Iron + copper mg/l, max	0.02	0.01	0.01	0.01	(2)		2.1.1(c)
Silica (SiO_2) max	0.05	0.02	0.02	0.02	(3)		2.1.1 (d)
Oil mg/l, max	Nil	Nil	Nil	Nil		59	
Residual hydrazine(2)	0.05	0.05	0.05	0.05	26		
Conductivity (3)	0.5	0.3	0.3	0.3	7		
Oxygen consumed (4)	Nil	Nil	Nil	Nil		51	

a) Methods of Test for routine control for water used in Industry
b) Methods of sampling and test (Physical and Chemical) for water used in Industry.
(1) as ($CaCO_3$) mg/l, max, (2) as (N_2H_4) mg/l, max, (3) after passing through cation exchange column at 25 deg C microsimens/cm, max, (4) in 4 hours mg/l, max

Source: CPHEEO, 1993

A typical flow sheet given in Figure 7.23 includes tertiary treatment in the form of chlorination, chemically aided sedimentation, sand filtration, sodium zeolite softening followed by cation exchange on hydrogen cycle, degasification and weak base anion exchange to give practically complete demineralization. Where the TDS is higher, technologies such as RO, evaporation, etc., should also be considered.

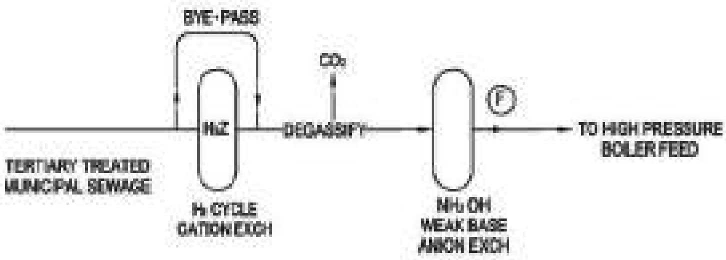


Figure 7.23 Flow sheet for reuse of treated sewage as high pressure boiler feed when TDS in sewage is very low in the region of about 300 mg/l or so

7.3.5.3 As Process Water

In order to keep treatment to a minimum for reuse as process water, one benefits from identifying those processes that must have freshwaters of high quality and those processes that can do with reclaimed water of low quality (e.g. similar in quality to that used for cooling or for low pressure boilers). This is done by having a multiple quality water supply system within the industry. Indian standards for quality tolerances for a few industrial uses are noted below:

- IS: 201 Water quality tolerances for the textile industry
- IS: 2724 Water quality tolerances for the pulp and paper industry

- IS: 3957 Water quality tolerances for ice manufacture
- IS: 4251 Water quality tolerances for the processed food industry
- IS: 4700 Water quality tolerances for the fermentation industry

It may be noted that generally all the processes in an industry do not require water of the relatively high quality given in the above noted Indian Standards. There are always several unit processes and operations where water of lesser quality can be tolerated. The issue of relevance in respect of industrial use is the crucial factor of reducing the TDS to about 500 mg/l as compared to the typical sewage value of 1,200 mg/l and above. This at some stage implies recourse to RO and the difficulty of dealing with rejects. It will be better to allow the user to discharge it back in the sewer itself at an appropriate location and charge an additional fee for the same, so that the practice will find a wider acceptance and take off towards a real success. Further the example of increasing the treatment capacity in the pre RO section to such a quantity that the bypassed stream before RO and the RO rejects stream when blended will be able to hold the TDS to less than 2,100 mg/l, the permissible value needs to be followed.

7.3.6 Guiding Principles - Fish Culture

Recognizing that Kolkata city is unique in having fish as an acceptable food and thus, the demand being steady. Fish ponds otherwise referred to as pisiculture cannot be looked upon as a method of stand-alone sewage treatment. However, treated / diluted sewage if used for pisiculture on the lines of the on-going East Kolkata wetlands, this needs to be strictly monitored by Department of Health and Department of Environment / SPCB and also the social acceptability.

7.3.7 Guiding Principles - Groundwater Recharge

At this point of time, the spread in the infiltration basins is fraught with many challenges as dust control, algae problems and essentially silica ingress into groundwater, in addition to the danger of the TDS and nitrate of applied sewage being higher than the TDS and nitrate of the groundwater. Further, financial ability of local bodies may not support this expenditure, which does not generate revenue except on a notional scale. On the contrary, if it is deep well injection on the lines of the first version of the Orange County plant, where injection wells were driven to about 100 m below ground to prevent seawater intrusion in water short coastal area, can be encouraged with appropriate safeguards.

7.3.8 Guiding Principles - Indirect Recharge of Impoundments

It is too early to embark on this till the results of the Bengaluru piloting are validated.

7.3.9 Guiding Principles – Indirect Reuse as Potable Raw Water Source

The indirect use of treated sewage has been going on in many ways and is detailed below. The reason that indirect water reuse is not to be considered to pose a health risk is that the treated wastewater benefits from natural treatment from storage in surface water and aquifers and is diluted with ‘ordinary’ river/ground water before abstraction to ensure good drinking water quality (part of a multi barrier approach in the water safety plan).

The storage time provides a valuable buffer to measure and control quality (Source: http://www.ciwem.org/policy-and-international/current-topics/water-management/water-reuse/potable-water-reuse.aspx).

7.3.9.1 Treated Sewage into Perennial Rivers

When sewage is treated and discharged into perennial flowing rivers and the blended river water is drawn downstream of the point of such blending as raw water for treatment in public water supply schemes. This is indirect potable use after blending. This is historical and ongoing all around. However, of late, the organic load due to the discharged treated, partially treated and non point sewage becomes in excess of the self purifying capacity of the river. Thus, the river water is not actually freshwater. The water quality of Yamuna river for Agra water supply scheme requires to be first treated in MBBR to purify the river water to a level as raw water for the downstream WTP. When it passes through flowing surface water it has the potential disadvantages of contamination by human and animal activities adding organic matter and waterborne pathogens unless the river stretch is protected from such activities. The guiding principle in such cases for the ULBs will be to at least intercept the sewage outfalls and provide adequate STPs and follow the recommended quality criteria for the treated sewage as in Table 5.20 of Chapter 5 in Part A manual.

7.3.9.2 Treated Sewage into Non-Perennial / Dry River Courses

There are locations where the rivers are not perennial or almost dry throughout the year except some monsoon runoff. In this case the discharged treated sewage sinks into the aquifer zone and is extracted by infiltration wells or galleries. The advantage of direct dilution from surface water is lost, but the additional purification in the soil and dilution from the aquifer water are happening. An example is the case of the Palar river course in Tamilnadu. The surface water flow in this occurs only for about a week if the monsoon is normal and if the water spills beyond the upstream impoundments. The aquifer however supports the public water supply of over 30 habitations along its dry tract of nearly 80 km before the sea. The partly treated sewage of the en route habitations do reach this river course as intervals. So far, no epidemics have been met with. This may be due to the above said additional purification in the soil and dilution by aquifer water. However, if these are exceeded by the contamination load, there can be immediate health problems. The guiding principle in such cases for the ULBs will be (a) to keep a check on the raw water quality from the infiltration wells to detect sudden increase in contaminants and (b) at least intercept the sewage outfalls and provide adequate STPs.

7.3.9.3 Treated Sewage into Surface Water Reservoirs

This may occur when the surface water reservoirs receive the inflow from rivers and become impoundments from which the raw water is taken for public water supplies. Here also, the same position as in 7.3.9.1. would apply.

When it passes through reservoirs, it has the potential issues of evaporation losses and algae. The algae creates taste and odour concerns and metabolic products of dead and decaying organisms as precursors which on chlorination are suspected to cause Trihalomethanes.

Upon chlorination, the residuals of insecticides and fertilizers are referred to as Endocrine Disruptor chemicals (EDCs) and this requires the use of Granular Activated Carbon filtration and sanitary protection of the catchments.

The case of Hosur in Tamilnadu is an example. The Kelavarapalli river was impounded here to take the raw water to the conventional WTP for the town. During 2004, the partial drought resulted in reduced flow in the river and the impoundment water slowly concentrate with dead and living algae.

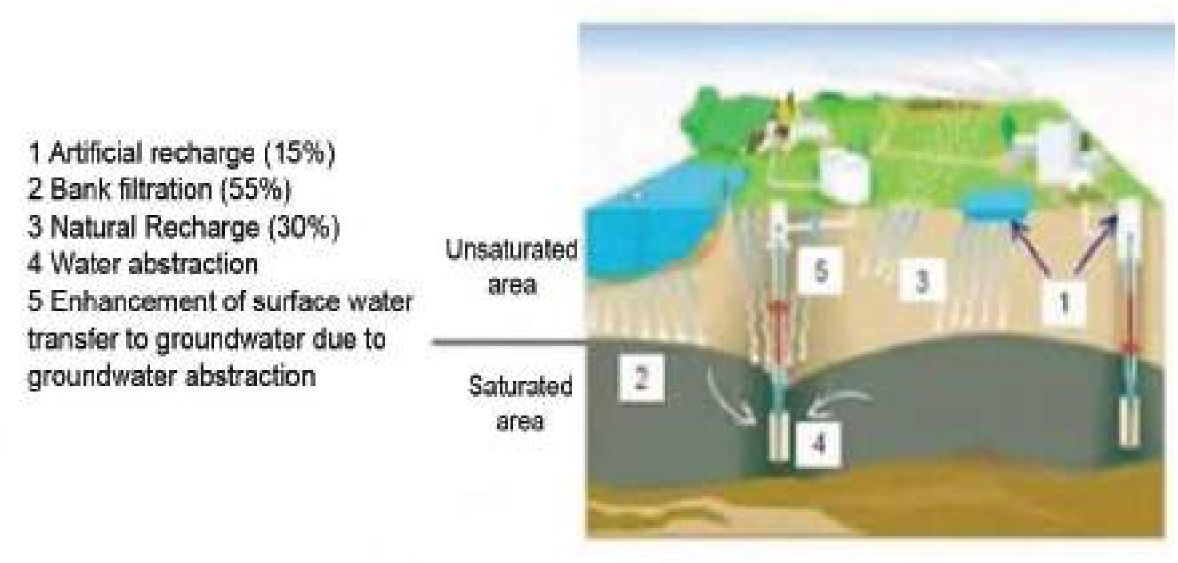
The conventional WTP could not treat the foul smelling coloured water and the high Lime and carbonation technology had to be retrofitted into the WTP on an urgent scale as otherwise, the town had to be evacuated. The cost of treatment jumped from Rs 4 to 14 per Kilo litre, but there was no other option. The guiding principle in such cases for the ULBs will be to ensure sanitary protection of the catchment, monitor the quality of the water entering the reservoir and keep contingency plans to switch the WTP into an appropriate mode to meet the raw water quality and also keep a contingency budget allocation for the increased O&M costs in such periods.

7.3.9.4 Treated Sewage into Conjunctive Uses in Surface Water and Aquifer

Conjunctive use of surface water and aquifer water is also being practiced as a method of indirect potable reuse.

Berliner Wasserbetriebe is reported to treat the 248 MLD of treated sewage to recharge surface water lakes, and the surface water is in turn used to recharge aquifers through artificial infiltration ponds and bank filtration by means of natural lakes.

The groundwater is stated to be then abstracted to supply 3.4 million people in Berlin with drinking water without chlorination. A schematic depiction is shown in Figure 7.24.

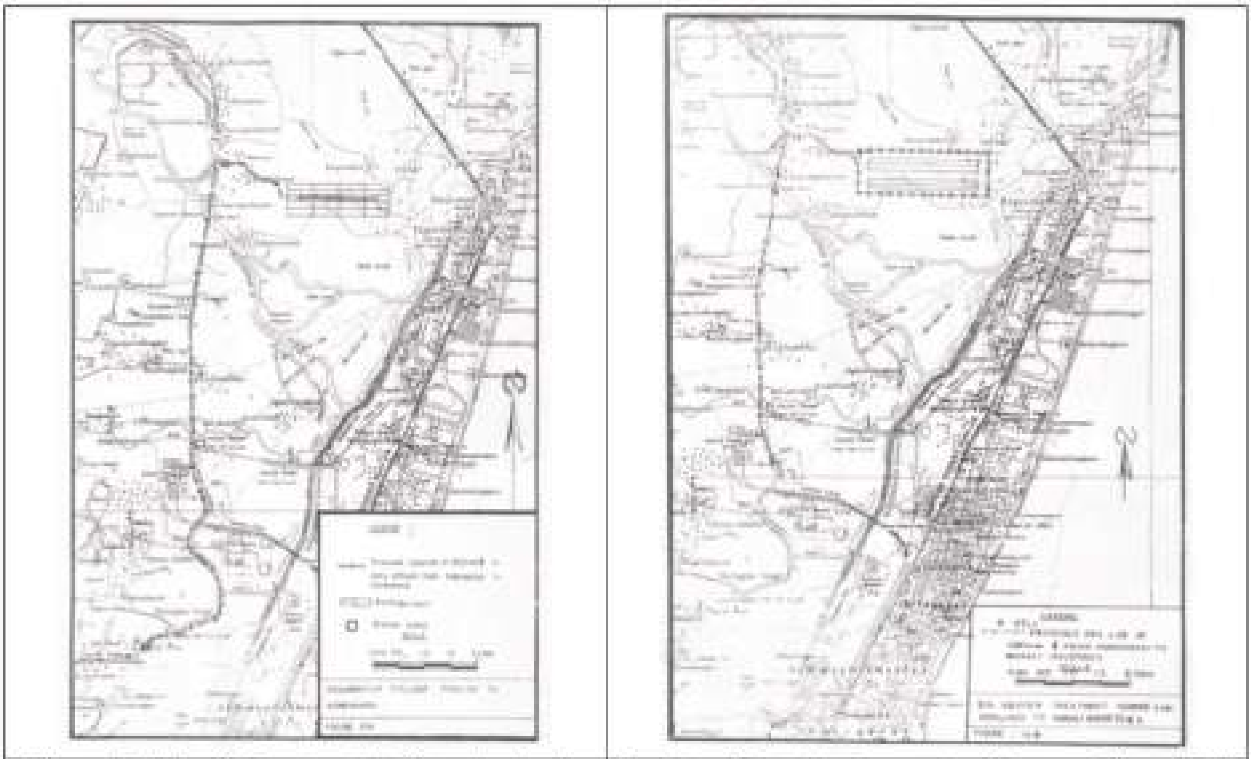


Source: Chartered Institution of Water and Environmental Management (CIWEM) webpage

Figure 7.24 Depiction of aquifer recharge in Berlin

7.3.9.5 Treated Sewage into Soil Zone and Reuse as Industrial / Agricultural Raw Water

This is referred to as Soil Aquifer Treatment (SAT). This indirectly conserves freshwater, which would have been otherwise used up in industry and agriculture. The Chennai UNDP studies established the technical and financial feasibility of treated city sewage to be applied on spreading basin and the SAT water extracted from bore wells around the periphery for cooling needs of petro-chemical industries some 20 km away is shown in Figure 7.25.



Chennai Soil Aquifer Treatment site Firmed up by the UNDP and is shown as the Rectangular Layout	Peripheral Extraction Bore wells of the same Chennai Soil Aquifer Treatment site

Figure 7.25 Chennai Soil Aquifer Treatment site firmed up by the UNDP

However, even after 2 decades, the proposal is yet to take shape.

This is due to public fear of such recharge getting into contact with nearby freshwater aquifers used by the population and irreversibly contaminating them.

At the same time, similar schemes are successfully operated in many parts of the world for meeting the agricultural water needs notably in Israel and USA, where the proximity to habitations issues are not arising and engineered artificial sandy tracts are constructed amidst sand dunes. The issue here is to that extent, the freshwater is conserved and hence, this is also an indirect reuse. The guiding principle for ULBs in this case will be to ensure that such SAT is prima facie only after the sewage is treated to the limits as in Table 5-20 of Chapter 5 in Part A manual and the availability of a confined aquifer for purposes other than agriculture and industry.

In any case, an assessment of potential impacts on underlying groundwater aquifers shall be determined comprehensively by extensive public consultation processes.

7.3.9.6 Treated Sewage Deep Well Injection as Seawater Intrusion Barrier

The seawater intrusion into inland coastal aquifers is a classic problem in many coastal locations. This occurs mainly due to the extraction of the freshwater from the freshwater lens beyond the safe yield limits. Once the seawater has intruded, this is not easy to be reversed. The Orange County Water District example in California, USA is perhaps the first in the world during the 1970s to demonstrate the use of tertiary treated, RO processed sewage to be blended with ground water and injected into a series of such barrier wells. It is reported that there are over 600 such wells in USA on the east and west coasts. The quality of the treated sewage before it qualifies itself to be considered for such an injection is governed by many aspects like volatile and non-volatile organic compounds, inorganic chemicals, Radionuclides and is typically with quality ranges as TDS of 44 to 416, chloride of 8 to 60, sulphate of 5 to 148, Total Organic Carbon of 1 to 3 and pH of 7 to 8 (EPA-The Class V Underground Injection Control Study, Volume 20, EPA/816-R-99-014, September 1999). Though this may be technically possible, the issue is the costs involved in the extra treatment of Reverse Osmosis (RO) and disposal of RO rejects into the sea after meeting the discharge limit in Table 5-20 of Chapter 5 in Part A manual In addition, the issues of consistent reliability of treated quality and safeguards of system redundancies also arise. The other issue is the cost comparison between the water conserved by such sea water intrusion barrier and that produced by sea water desalination with the added cost of distributing it all over the habitation by piping. In the case of the conserved water by seawater barrier injection, it is available for the user to extract it at his convenience right in the aquifer where he lives. Thus, the guiding principles in the case of seawater barrier injection option are primarily, public acceptance and the financial sustainability besides risk mitigation to get over system redundancies.

7.3.10 Guiding Principles - Other Uses

These are already happening anyway and can be continued with appropriate safeguards, the essentials being the adequate chlorination to maintain residual chlorine of 0.5 mg/l at the point of use and the colour to be aesthetically acceptable especially while applying in public places.

7.3.11 Standard of Treated Sewage and its Uses

In addition to the guiding principles mentioned earlier the recommended treated sewage quality as in Table 7.19 (overleaf) is proposed to be achieved for the stated uses. Hence, in order to achieve the desired water quality, excess chlorination, granular activated carbon adsorption / ozonation and/or various kind of filtration including membrane are recommended. For recreational impoundments for non-human contact, residual chlorine is not required so as to protect aquatic species of flora and fauna.

However, for use in Wetlands, Wildlife habitat and Stream augmentation the recommended water quality in Table 5.2 (of Chapter 5 in Part A manual) for inland surface water discharge suffice the purpose. For uses in the construction industry like (a) Soil compaction, (b) Dust control, (c) Washing aggregate the recommended water quality in Table 5-2 for inland surface water discharge guidelines are sufficient. While for preparing concrete mix, the acidity < 50 mg/L as $CaCO_3$, SO_4 < 400 mg/L, TDS < 3,000 mg/L, Chloride < 500 mg/L respectively as in IS: 456 are to be considered.

Table 7.19 Recommended norms of treated sewage quality for specified activities at point of use

	Parameter	Toilet flushing	Fire protection	Vehicle Exterior washing	Non-contact impoundments	Landscaping, Horticulture & Agriculture			
						Horticulture, Golf course	crops		
							Non edible crops	Crops which are eaten	
								raw	cooked
1	Turbidity (NTU)	<2	<2	<2	<2	< 2	AA	< 2	AA
2	SS	nil	nil	nil	nil	nil	30	nil	30
3	TDS	2100							
4	pH	6.5 to 8.3							
5	Temperature °C	Ambient							
6	Oil & Grease	10	nil	nil	nil	10	10	nil	Nil
7	Minimum Residual Chlorine	1	1	1	0.5	1	nil	nil	nil
8	Total Kjeldahl Nitrogen as N	10	10	10	10	10	10	10	10
9	BOD	10	10	10	10	10	20	10	20
10	COD	AA	AA	AA	AA	AA	30	AA	30
11	Dissolved Phosphorous as P	1	1	1	1	2	5	2	5
12	Nitrate Nitrogen as N	10	10	10	5	10	10	10	10
13	Faecal Coliform in 100 ml	Nil	Nil	Nil	Nil	Nil	230	Nil	230
14	Helminthic Eggs / litre	AA	AA	AA	AA	AA	<1	<1	<1
15	Colour	Colourless	Colourless	Colourless	Colourless	Colourless	AA	Colourless	Colourless
16	Odour	Aseptic which means not septic and no foul odour							

All units in mg/l unless specified; AA-as arising when other parameters are satisfied;
A tolerance of plus 5% is allowable when yearly average values are considered.

7.3.12 Public Education

Education is the key to overcoming public fears about a reuse system, particularly fears that relate to public health and water quality. A broad, in-depth public relations programme and a demonstration project are especially helpful when the reuse project is the first of its kind in the state.

7.3.13 Piping and Cross-connection Control

A residual chlorine > 0.5 mg/l in the distribution system is recommended to reduce odours, slime and bacterial growth (US EPA 2004). It is crucial to be able to differentiate between piping, valves and outlets that are used to distribute treated effluent or reclaimed water and those that are used to distribute potable water. One method used for this purpose is colour-coding the components used to distribute reclaimed water not intended for drinking water. Another method is to post areas such as parks and yards with warning signs stating that the piped water there is not for human consumption. The signages should be in all the major languages of the region.

7.4 LEGAL ISSUES

The legal rights over the sale and revenue issues of reclaimed water are an emerging issue and this is addressed in Part C as a management aspect.

CHAPTER 8: DECENTRALIZED SEWERAGE SYSTEM

8.1 DEFINITION

Decentralized sewerage system is defined as the collection, treatment, disposal / reuse of sewage from individual homes, clusters of homes, isolated communities or institutional facilities, as well as from portions of existing communities at or near the point of waste generation. Typical situation in which decentralized sewerage management should be considered or selected include:

1. Where the operation and maintenance of existing on-site systems must be improved.
2. Where individual on-site systems are failing and the community cannot afford the cost of a conventional sewage management system.
3. Where the community or facility is remote from existing sewers.
4. Where localized water reuse opportunities are available.
5. Where freshwater for domestic supply is in short supply.
6. Where existing STP capacity is limited and financing is not available for expansion.
7. Where, for environmental reasons, the quantity of effluent discharged to the environment must be limited.
8. Where the expansion of the existing sewage collection and treatment facilities would involve unnecessary disruption of the community.
9. Where the site or environmental conditions that require further sewage treatment or exportation of sewage are isolated to certain areas.
10. Where residential density is sparse.
11. Where regionalization would require political annexation that would be unacceptable to the community.
12. Where specific sewage constituents are treated or altered more appropriately at the point of generation.

8.2 CHALLENGES IN SUSTAINING A CENTRALIZED SEWERAGE

A centralized sewerage de facto is perceived as an underground sewer system to collect the sewage from all over a habitation and involves the challenges as described below.

8.2.1 Financial Sustainability

It implies a huge capital cost and mandates a full-fledged occupation of the coverage area to generate the revenue for its upkeep. In practice however, in the peri-urban areas and rural habitations, these are nearly impossible and the situation is escalating.

8.2.2 Idle Volumes and Time in Conventional Sewerage

Invariably, the sewers as a convention are designed for the ultimate population some 30 years away and the realization of the sewage volumes to use the designed sewer capacities results in idle volumes and idle expenditures as in Figure 8-1 and the underground sewers laid there merely become defunct with time and eventually go into repair. This is a non-productive expenditure in a sense, implying that the investment could have been utilized elsewhere as brought out in a classical illustration in Figure 8.1 .

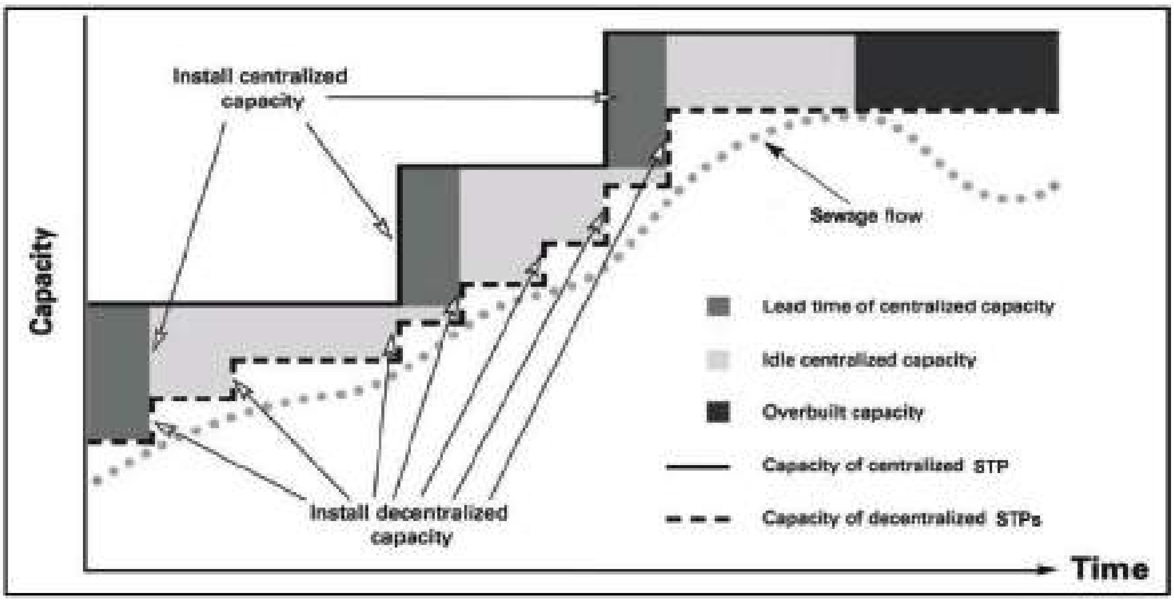


Figure 8.1 Logistics of capacity building in sewerage provision in centralized planning and decentralized planning of the collection and treatment facilities

8.2.3 Idle Investment in Conventional Sewerage

In conventional sewerage, the sewer sizes are also bigger and this brings in additional redundancy as in Figure 8.2.

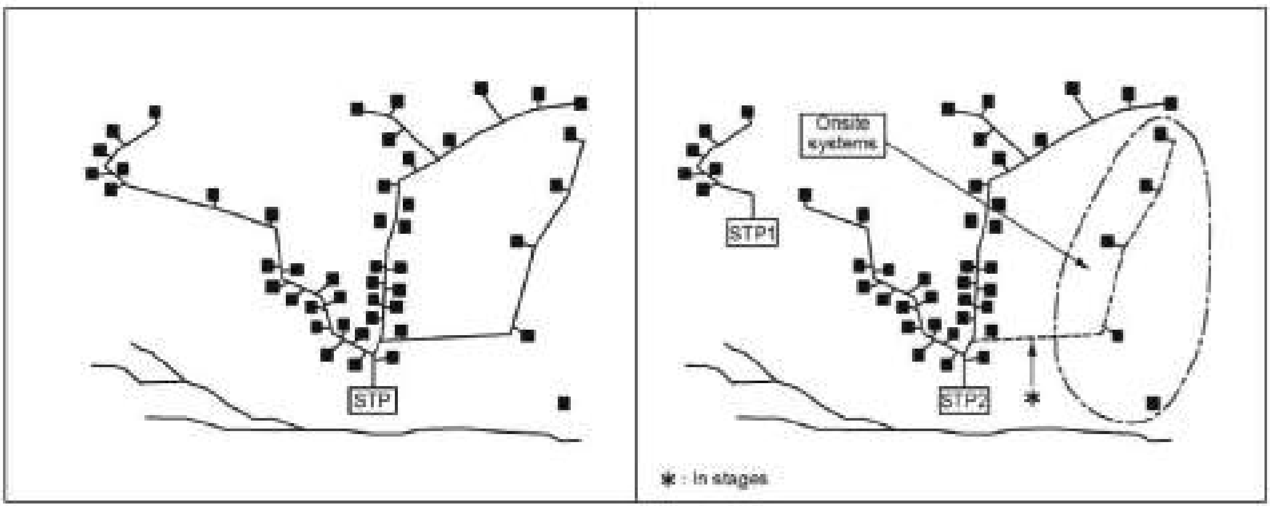


Figure 8.2 Concept of centralized system at left Vs. decentralized system at right

For example, by considering the illustration in Figure 8-2, the map to the left shows the typical conventional sewerage with all the sewers funneling to a single STP and the sector to the top right is actually sparsely developed and the sewers are designed for a flow some 30 years hence. This results in a situation where the manhole covers get stolen and people start using the manholes as a virtual garbage bin, which in turn is compounded by rainfall and lead to a near complete choking of the sewer system. The net result is, if and when the sector gets populated, a massive rehabilitation programme of the sewer system becomes implied often leading to indiscriminate cutting open of the roads. A further difficulty is the STP, which is grossly underutilized and the treated sewage quality suffers due to prolonged hydraulic retention. By contrast, if we consider the same sector to be served by a decentralized sewer system as in the map to the right, it can be seen that the above problems are surmounted not only physically, but also financially investments are saved to begin with.

8.2.4 Problems of House Service Connections

It is also a fact that while the investment on provision of sewerage is usually met out of capital grant funding, the cost of house service connections is to be met by the house owners and herein lies another conflict. Whereas houses have not come up in some sectors, these house service connections get time deferred and to that extent, repeated road cuts become a perpetual affair over a long time. As and when the houses are built, service connection requests arise. An approach that has been tried out is the provision of house service connection sewers even in the beginning itself and blank it at the property boundary and connect it only when the house gets built up and the applicant pays up the costs thereof.

Here again, it is a question of idle investment at start with no foreseeable return of the same on the house service connection costs.

Another issue is surreptitious connections by house owners and the impracticality of checking each and every such connection by the limited staff of the local body and may well be connivance also. By opting for decentralized sewer system, first of all, the command area to be supervised for such surreptitious connections get much smaller and the monitoring mechanism becomes effective.

8.2.5 Conflict of Levies for Recovering the Sewer Costs

Whereas the capital costs are mostly met out of grant funding, the O&M expenses are to be generated by the local body at most times. The meagre revenues generated by taxes and water and sewerage charges are too meagre to even break even in the local body accounts, leave alone increasing the reserve funds. When an unwieldy coverage of a conventional sewerage is implemented, the problem gets compounded all the more because the house service connections do not keep pace and the revenues are meagre. Thus, even the cost spent on the house sewer connections becomes a virtual write-off over a period of time.

8.3 CONCEPT OF DECENTRALIZED SEWERAGE

The decentralized sewerage concept implies localized collection and localized treatment of excreta and sullage in micro zones within a major habitation, keeping it in tandem with densification and progressively duplicating it, as and when other micro zones densify.

It will ensure that every micro zone owns up its excreta and sullage management and cannot expect a faraway habitation to receive and inherit it a prospect, which will sooner or later lead to inter conflicts and destabilize progress. Thus, the provision of both the collection system and treatment can be made compatible to the pace of development by juxtaposing on-site sanitation as well in its fold. The treatment systems of sewage in the on-site system and the off-site system are shown in Figure 8.3 hereunder.

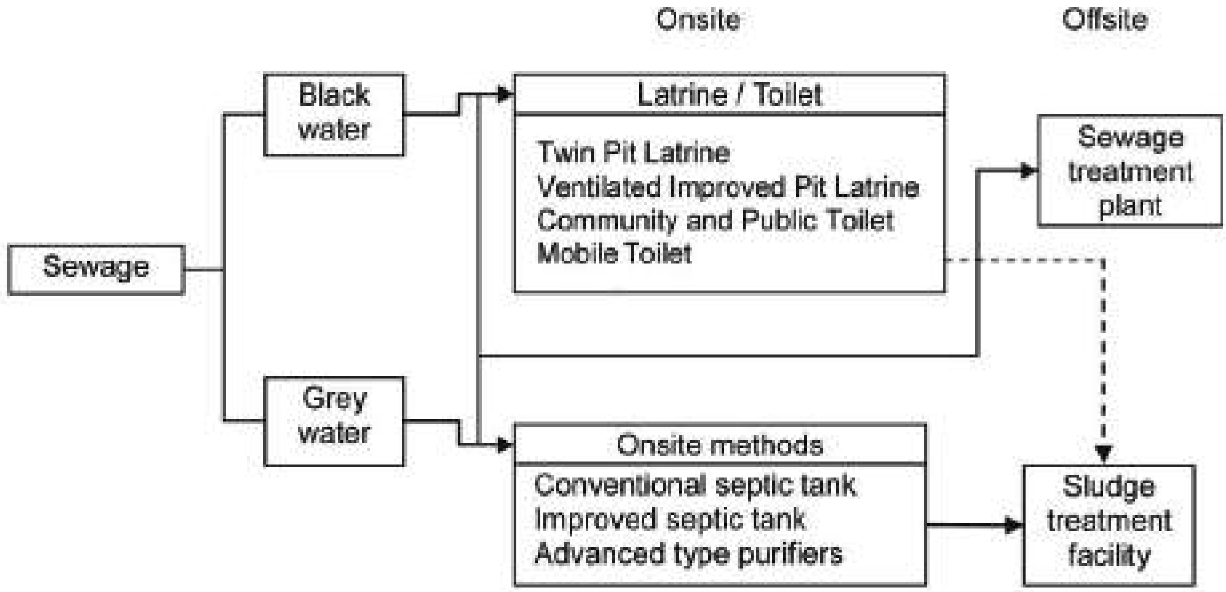


Figure 8.3 On-site and Off-site sewage treatment system

Note: There can be cases where both black and grey water can be treated together.

8.3.1 Advantages of Decentralized Sewerage

1. In general, prediction of sewage volumes is far easier in decentralized sewerage micro collection areas and to that extent the design becomes realistic.
2. Flows in a decentralized sewerage are relatively smaller than conventional sewerage and this implies that environmental damages from any mishaps are also minimal.
3. Given the smaller flows, the sewer sizes are also smaller and the depths of cut are also less thus, making it easy to construct and maintain.
4. Additions of new service areas which are independent of the existing system and the need to augment or enlarge the existing sewers and STPs are avoided.
5. The STPs are smaller and it is easier to find the reuse prospects nearby as compared to all the sewage being treated in one far corner.
6. It is also easier to layout return lines of treated sewage for use in medians, industrial supplies, flushing far flung head manholes, etc.
7. The ecology of rivers, streams and receiving waters are better managed by smaller volumes of discharges of treated sewage at multiple locations than one massive volume in a single location and also if the single STP is out of order, the entire stretch of the water course is polluted.

8.4 TECHNOLOGIES OF DECENTRALIZED SEWERAGE

8.4.1 Simplified Sewerage

Simplified sewerage is a technology widely known in Latin America, but much less known in Africa and Asia. It has been successfully demonstrated in the Orangi habitation of Pakistan (having a population of about 7.50 lakh, where per capita water supply is about 27 lpd) and since adopted there in situations similar to the status in the preamble here. Duncan Mara defines simplified sewerage as “An off-site sanitation technology that removes all wastewater from the household environment.” Conceptually it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local situation. The simplified sewerage approach is now widely used. Figure 8.4 is one such example at Brazil as an in-block system rather than – as with conventional sewerage – an in-road system.

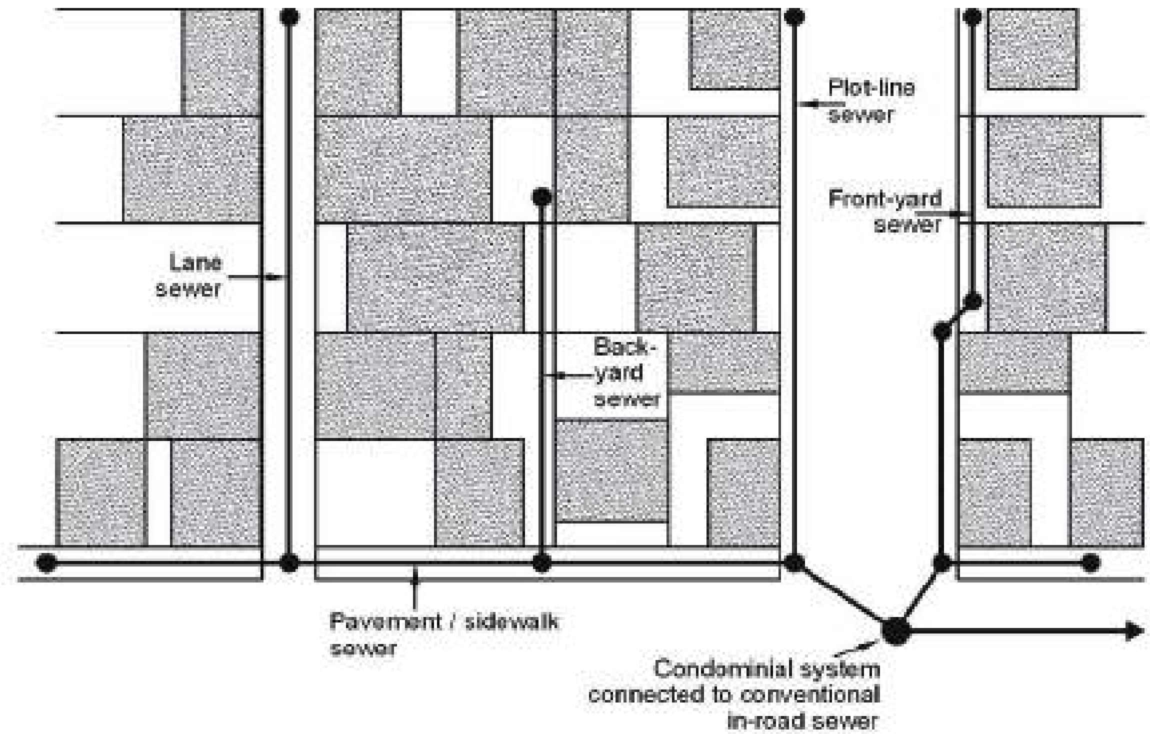


Figure 8.4 Simplified sewerage as avoiding public roads unless actually required

The key feature of an in-block system is that sewers are routed in private land, through either back or front yards. This in-block or back-yard system of simplified sewerage is often termed condominial sewerage in recognition of the fact that tertiary sewers are located in private or semi-private space within the boundaries of the ‘condominium’. These simplified sewers are laid at shallow depths, often with covers of 400 mm or less. The minimum allowable sewer diameter is 100 mm, rather than the 150 mm or more that is normally required for conventional sewerage.

The relatively shallow depth allows small access chambers to be used rather than large expensive manholes as in Figure 8-5, Figure 8.6 and Figure 8.7 overleaf.

Source: Mara et al.

Figure 8.5 Pavement (sidewalk) simplified sewerage being installed in the high-income area of Lago Sul in Brasília in 1999

Figure 8.6 Simplified sewerage in the footpath and main road free of manholes in French Puducherry - early 20[th] century

Source:Mara et al., 2001

Figure 8.7 Junction chamber for simplified sewerage using larger diameter concrete pipes, used in Guatemala

8.4.1.1 Design Criteria

In regard to design, the basic procedures are the same as any hydraulics. For example, for serving 500 people with a water consumption of 80 lpcd and using a return factor of 0.85, the average daily sewage flow will be 0.85 × 500 × 80 / 24 / 3600 = 0.4 lps.

Depending on the peak factor as given in Chapter 3, the design flow will be its multiplication and the design will be as per Manning's formula as in Chapter 3. The design guidelines are available in *http://www.efm.leeds.ac.uk/CIVE/Sewerage/manual/pdf/simplified_sewerage_manual_full.pdf.*

The long-term sustainability of simplified sewer systems can be ensured by a good partnership between the community served by simplified sewerage and the sewerage authority along with the following key factors:

1. Good design
2. Good construction
3. Good maintenance
4. An adequate, but affordable, tariff structure.

It is in item 4 that the success of the system resides and requires a public hearing and acceptance, instead of taking the public acceptance for granted. Eventually, when the habitation becomes fully developed, of course the conventional sewerage can still be incorporated in lieu of the simplified sewerage.

8.4.2 Small Bore Sewer System

Small bore sewer system is designed to collect and transport only the liquid portion of the domestic sewage for off-site treatment and disposal. The solids are separated from the sewage in septic tanks or aqua privies installed upstream of every connection to the small bore sewers. Where conventional sewers would be inappropriate or infeasible, this system provides an alternative. This system also provides an economical way to upgrade the existing on-site sanitation facilities to a level of service comparable to conventional sewers. Since the small bore sewer collects only settled sewage, it needs reduced water requirements and reduced velocities of flow. This in turn reduces the cost of excavation, material and treatment. This is also called as settled sewerage.

8.4.2.1 Components of the System

The small bore sewer systems consist of house connections, interceptor tanks, sewers, cleanouts and manholes, vents and in some cases lift stations.

8.4.2.2 Suitability of the System

This system is suitable under the following conditions, where

1. Effluent from pour-flush toilets and household sullage cannot be disposed off on-site
2. Installation of new schemes is taken up, especially for fringe areas

3. A planned sequence of incremental sanitation improvements with small bore sewers as a first stage is contemplated
4. Existing septic tank systems have failed or where there are a number of septic tanks requiring the effluent to be discharged, but soil and ground water conditions do not permit such a discharge.

8.4.2.3 Design Criteria

Each house sewer is usually connected to an interceptor tank, which is designed as a septic tank. The optimum number of house sewers to be connected to an interceptor tank can be worked out for each case. The effluent from the tank is discharged into the small bore sewer system, where flow occurs by gravity utilizing the head resulting from the difference in elevation of its upstream and downstream ends. The sewer should be set deep enough to carry these flows.

The diameter of sewer pipe shall be designed for incremental flows between successive sections. First consider the available ground slope and choose a minimum of 100 mm sewer pipe and use Manning's formula for pipes flowing full and find out the flow carrying capacity. If this is lesser than the actual flow in that section, increase the pipe diameter in that section as needed. Velocity is not a criterion.

Design decisions regarding the location, depth, size and gradient of the sewer must be carefully made to hold hydraulic losses within the limits of available head. Minimum pipe diameter of 100 mm is recommended. Maintenance of strict sewer gradients to ensure minimum self-cleansing velocities is not necessary. The sewer may be constructed with any profile as long as the hydraulic gradient remains below all interceptor tank outlet inverts. Ventilation is not necessary for small bore sewers, if they are laid on a falling gradient. A vent cleanout to release air may be provided at every hump. Profiles are shown in Figure 8.8 (overleaf). An example on design calculation is also presented in Appendix A.8.1.

8.4.3 Shallow Sewers

8.4.3.1 System Description

Shallow sewers are designed to receive domestic sewage for off-site treatment and disposal. They are a modification of the surface drain with covers and consist of a network of pipes laid at flat gradients in locations away from heavy imposed loads (usually in backyards, sidewalks and lanes of planned and unplanned settlements). They are usually laid at a minimum depth of 0.4 m. Where vehicular loading is present and the invert depth of sewer is less than 0.8 m, a concrete encasement is provided for the sewer.

8.4.3.2 Components of the System

The shallow sewer system, like the conventional sewer system consists of house connections, inspection chambers, laterals, street collector sewers, pumping stations where necessary and treatment plants. Low volume pour flush or cistern-flush water seal toilets are connected to the inspection chamber by means of a 75 mm diameter sewer.

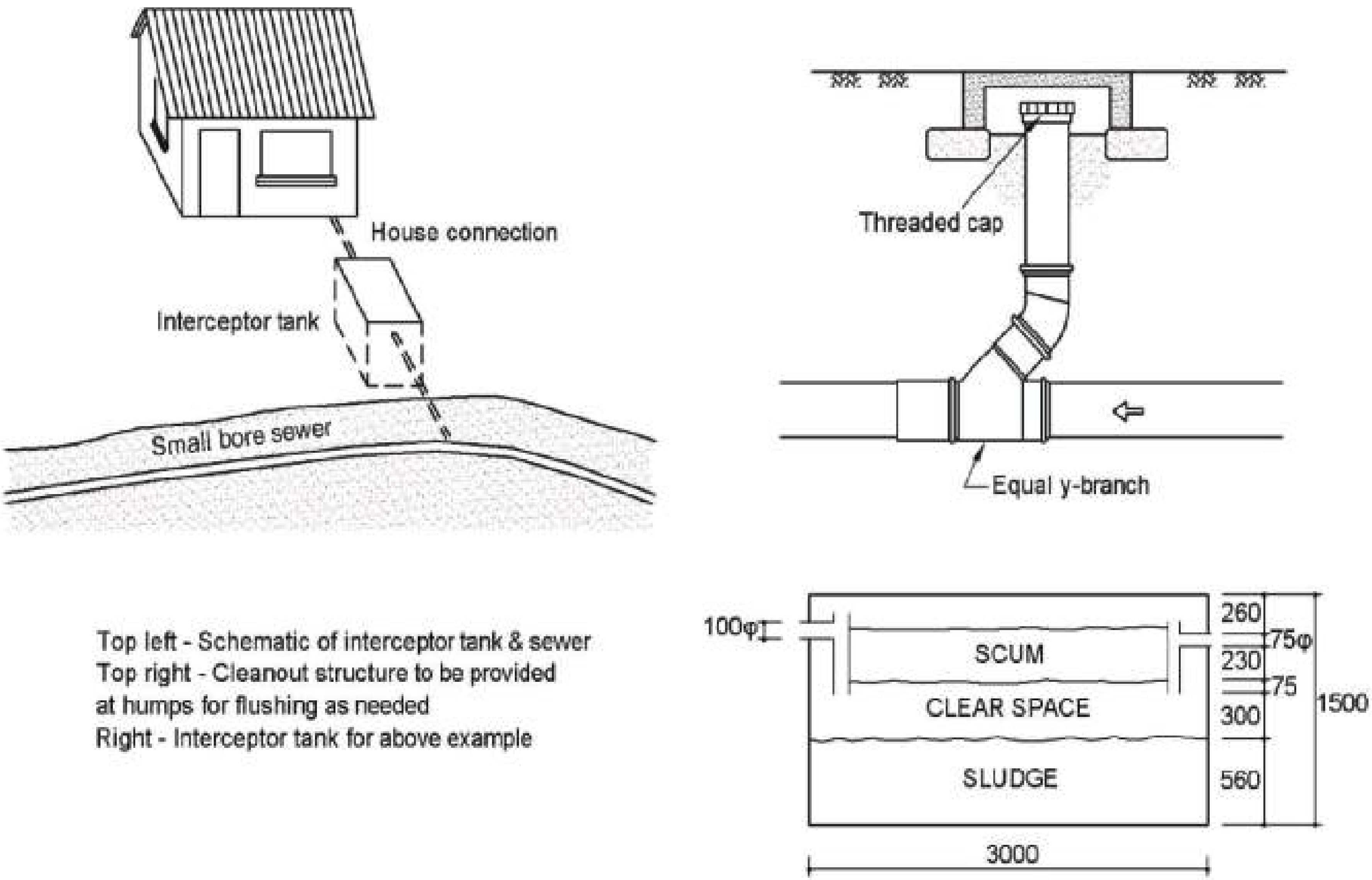


Figure 8.8 Schematics of small bore sewer system

A vertical ventilation column of the same diameter is provided on the house connection. The sullage water generated in the house is also connected to the inspection chamber directly when water consumption is more than 75 lpcd. Where the water consumption is lesser and where grit is used for cleaning purpose, it is connected through a grit/grease trap. Inspection chambers are provided along the street collector sewers and along the length of the laterals at intervals not exceeding 40 m. Usually one chamber is provided for each house. However two or more houses may share a single inspection chamber. The chamber is provided with a tight-fitting RCC cover. The laterals are of small diameter (minimum 100 mm) and of stoneware or concrete, which are buried in a shallow trench. The minimum depth of pipe invert is 0.4 m. In general, they have a straight alignment between inspection chambers and are suitably aligned around existing buildings. They may even pass under property boundary walls and also under future building areas. The inspection chamber however, is located is an open area. The street collector sewer has a usual minimum diameter of 150 mm, however, 100 mm sewers may also be used if hydraulic capacities permit. Where community septic tanks are provided at the exit of the lateral sewers, the street sewers should be designed as small-bore sewers. The pumping stations should, as far as possible, be avoided in such cases.

8.4.3.3 Design

The design procedure is as much the same as that of gravity sewer design in Chapter 3.

8.4.3.4 Applicability

Shallow sewers are suitable where

1. high density, weaker sections, squatter settlements (100 to 160 persons per hectare) exist
2. adverse ground conditions exist and on-site disposal is not possible
3. sludge also has to be disposed off and where the minimum water consumption rate is 25 lpcd.

8.4.3.5 Limitations

Shallow sewerage system is suitable where adequate ground slopes are available. Since these sewers are laid at flat gradients the solids are likely to get deposited unless flushed at peak flow conditions. Otherwise, these sewers may get clogged and require frequent cleaning.

8.4.4 Twin Drain System

This is an integral twin drain on both sides of the road. The drain on house side receives the sewage. The drain on road side is the storm water drain. It is in use in coastal areas of Tamilnadu particularly in Tsunami affected habitations. The advantage is that even if the per capita sewage falls to low quantities, say 28 lpcd as is still there in some cases, where water is scarce like in coastal fishermen communities this can be adopted. The design of the drain with removable cover slabs permits the daily scraping forward of sediments progressively by each house owner in the portion of the drain before his premises to the destination treatment site, something that the other options do not permit that easily.

8.4.4.1 Installation at Kolachel, Tamil Nadu

This is a decentralized sewerage and sewage treatment for Tsunami Rehabilitation for a population of 2,000 and 350 dwellings and it is in use since July 2007 at Kolachel which is located near the backwaters of Bay of Bengal in Tamilnadu. A schematic drawing of the self-contained system is shown in Figure 8.9. The photographs are in Figure 8.10 and Figure 8.11

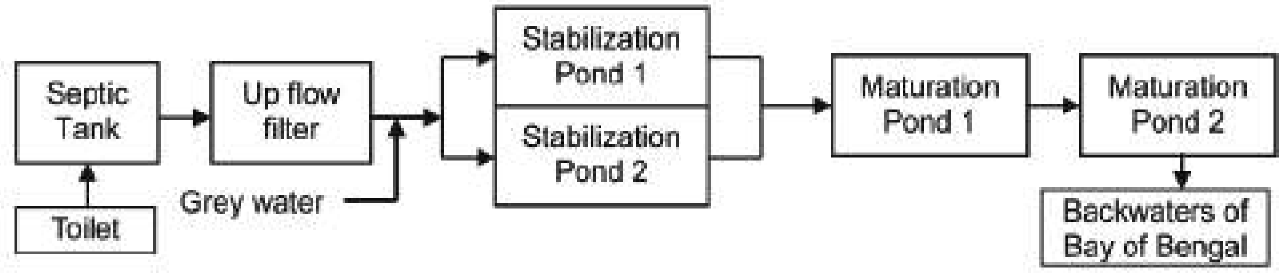


Figure 8.9 Schematic of the twin drain decentralized sewerage

8.4.4.2 Design Adopted for the System

1) The daily flow is 1,89,000 litres (350 houses – 6 persons per dwelling – 90 litres per head).
2) The septic tank was sized at 2 m × 0.9 m × 1.4 m liquid depth for cleaning at 3 year interval.
3) The up flow filter was sized at 0.7 m × 0.9 m × with floor depressed by 0.5 m.
4) The ablution water is about 6 litres/person/day.
5) The drains were designed for a velocity of 0.3 m/s to conserve depth of excavation.
6) The loading for oxidation pond design was 275 kg BOD/hectare/day.
7) The pond was designed for liquid depth of 1.5 m and sludge accumulation of 0.5 m.
8) The detention in maturation ponds was 3 days and was 1.5 m deep.
9) The facultative ponds were provided in two parallel modules of each 50% capacity.
10) The maturation ponds were provided in series with two modules of each 50% capacity.
11) The pond bottom was dense clay for 1.5 m and hence, lining was not needed.
12) The treated sewage was flowing out into the backwaters of the Bay of Bengal.

8.4.4.3 Performance of the System

The biochemical parameters of performance are given in Table 8-1.

8.4.4.4 Financial Aspects of the System

By way of comparison, the cost of the collection system starting from septic tank and up to the ponds was only 38% of what would have been the cost for a conventional underground sewer system. In respect of the O&M costs, the twin drain system is only 8% of that for the conventional system. This illustrates the relative sustainability of this system.

Top: Septic tank up flow filter under construction in brickwork and as completed
Middle: Terminal chamber & twin drain under construction and as completed
Bottom: Stabilization pond and maturation pond and safe walkway all-round.

Source: M/s Kottar Social Service Society, Nagercoil and M/s Caritas India and M/s Caritas Germany

Figure 8.10 Twin drain system

Twin Drain with top perforated cover slabs for storm drain. Salt pan totally spoiled by Tsunami waves overflowing on it and since reclaimed to accommodate the STP pond system.

Source:M/s Kottar Social Service Society, Nagercoil and M/s Caritas Swiss

Figure 8.11 Another set up of twin drain system at Kodimunai

Table 8.1 Physico-chemical characteristics of the Kolachel System (Mean values)

No	Location	BOD	COD	SS	TKN	Total P
1	Septic Tank entry	1294	2565	4142	170	30
2	Up flow filter entry	702	1509	1450	111	24
3	Up flow filter outlet	399	1003	628	88	14
4	Grey water	362	615	359	28	16
5	Stabilization pond inlet	51	212	57	14	11
6	Stabilization pond outlet	31	144	42	10	8
7	Maturation pond 1 outlet	32	144	42	10	8
8	Maturation pond 2 outlet	23	124	38	7	6

8.4.4.5 Applicability

In most of new layouts the septic tank and open drains on road sides for storm water are a matter of routine and invariably the septic tank effluent is discharged into the drain which complicates the environmental hazard in rainy seasons. The twin drain system can stall the pollution by containing the septic tank effluents, which can be collected and provided with treatment. For new layouts, it will be useful if the bye-laws can be strengthened to mandate the twin drain instead of the roads drain alone, which is anyway mandated by the Town and Country planning act.

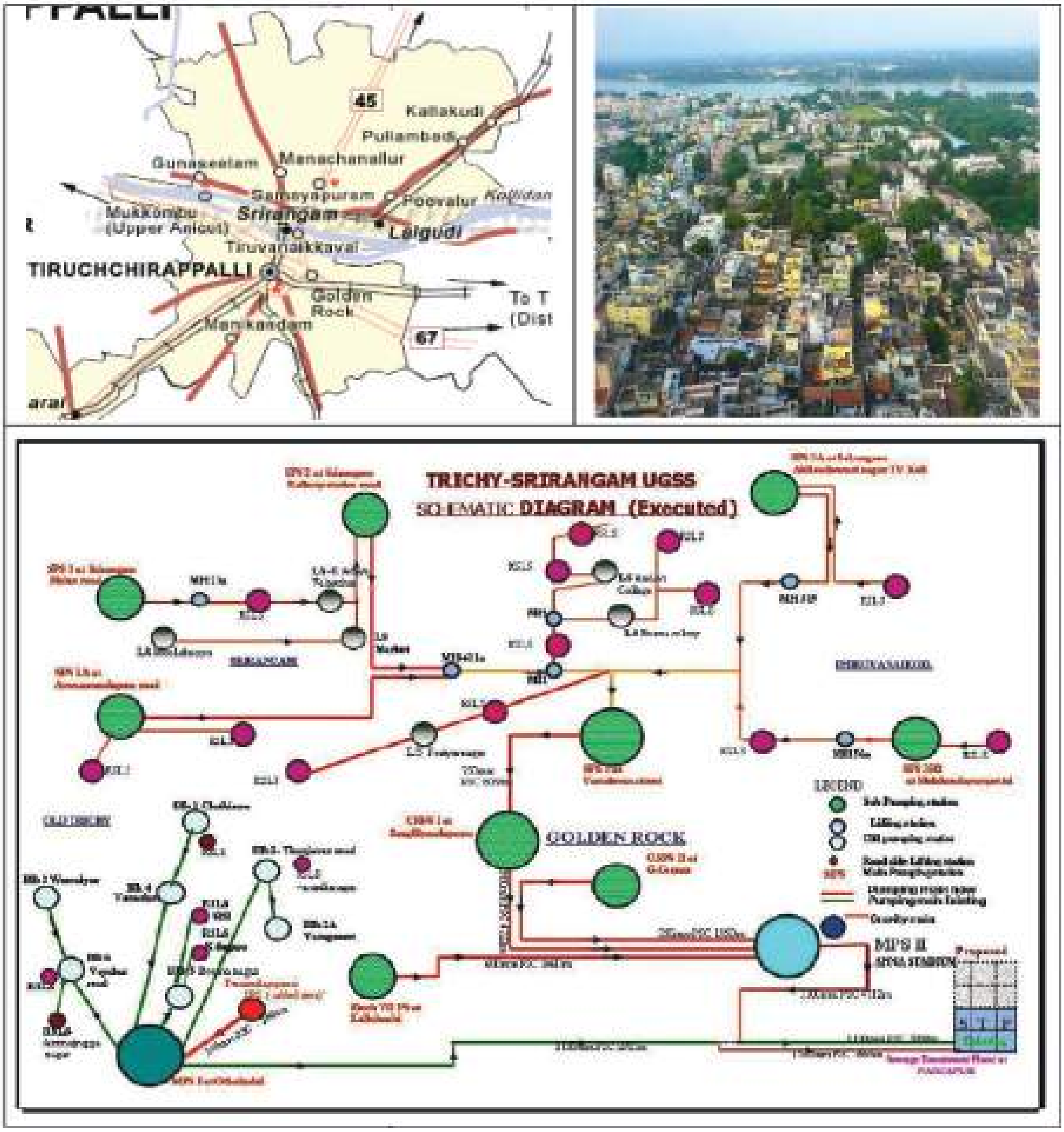
8.5 APPLICATION OF DECENTRALIZED SEWERAGE IN URBAN AREAS

It is not as though the decentralized sewerage is meant for peri-urban and rural settings alone. In fact, it is as much applicable to even metropolitan centres as in the case of our major State capitals like Chennai, Bangalore, Delhi, Ahmedabad, etc. except that the treatment is decentralized, but the sewerage is conventional sewerage. This is understandable in these locations, but Trichy for example where over the decades, a decentralized sewerage could have been evolved, but it has been a case of the entire sewage going down to one far corner and the sewage from one side of the river Cauvery being pumped to the other side 330 m across. While apparently this may appear paradoxical, it is not so because the tiny Srirangam habitation from where the sewage is pumped across the major river, is a highly revered and very densely populated religious centre and positioning a STP at Srirangam was ruled out from public acceptance point of view. Hence, it has not been possible to decentralize as shown in Figure 8.12 (overleaf).

The Cauvery and Coleroon (Kollidam) are, perhaps, the biggest rivers in South India.

Trichy (Tiruchirapalli) is situated on the banks of river Cauvery and the STP of the city is located on the South Eastern corner of the city.

The sewage of the entire habitation between Cauvery and Coleroon rivers could have been routed towards Coleroon river.



Top left - Trichy town of 150 km^2 area lying on both sides of the perennial river Cauvery

Top right - Panorama of the town illustrating the efficacy of decentralized sewerage

Bottom - Paradox of centralized sewerage over a century

Source: TWAD Board webpage

Figure 8.12 Historical sewerage system of an Indian city

However, the sanctity of the Srirangam town in this location is revered by the followers so highly that STP was not acceptable by the public and hence, the entire sewage is pumped across the Cauvery river to the existing STP. This is a case where decentralized sewerage, though technically justifiable has to give way to public acceptance.

8.6 PUBLIC TOILETS AS DECENTRALIZED SEWERAGE

In effect public toilets are a further decentralization within decentralized sewerage in that it answers the needs of the floating population in locations as market places, bus terminals and super markets. These are a compelling necessity as the user is a stranger to the location, who may be passing through it and may not know what to do and is unable to control the urge to relieve him or herself.

8.6.1 Norms for Public Toilet facilities with focus on attention to gender issues

Provision of public toilet facilities to meet the demands of opposite genders entering a toilet which is not designated for the accompanying child should be given importance while designing these kinds of facilities. Mostly the child caring facilities are provided in Women's toilet section and not generally in Men's toilets section. Absence of such facility would put the men with difficulties, when they required caring for the accompanying children and kids. Provision of dedicated toilets for differently abled persons and transgender is also need to be taken care of while designing the public toilet facilities. The General Norms for Public Toilets and norms for provision of sanitary facilities as recommended by the Town and Country Planning Organisation (TCPO) in the Model Building Bye-laws are given in Table 8-2 and Table 8-3 respectively.

Table 8.2 Norms for Toilets in Public spaces

Public Toilet	On roads and for open areas @every 1 km, including in parks, plazas, open air theatre, swimming area, car parks, fuel stations. Toilets shall be disabled-friendly and in 50-50 ratio (M/F). Provision may be made as for Public Rooms (Table 8.3).
Signage	Signboards on main streets shall give directions and mention the distance to reach the nearest public convenience. Toilets shall have multi-lingual signage for the convenience of visitors. Helpline number shall be pasted on all toilets for complaints/queries.
Modes	Pay & use or free. In pay and use toilets entry is allowed on payment to the attendant or by inserting coin and user gets 15 minutes.
Maintenance/ Cleaning	The toilet should have both men and women attendants. Alternatively automatic cleaning cycle covering flush, toilet bowl, seat, hand wash basin, disinfecting of floor and complete drying after each use can be adopted, which takes 40 seconds. Public toilet shall be open 24 hours.

Table 8.3 may well ipso facto apply for transit stations like bus stations, markets and most importantly road side users. The determination of the numbers for roadside toilet users can be computed by considering the number of people transiting that road in the day time and providing the toilets at strategic locations.

Toilets for transgenders can also be appropriately allocated as stand alone without clubbing with gender based toilets and the doors opening directly into the vastness of the hall instead of a narrow passage. Toilets for differently abled person's are easily constructed and identified and will almost invariably have a western toilet, guide rails on both walls, water faucet for ablution and wash basins at chair level.

Table 8.3 Norms for sanitary facilities in Public Toilets

No.	Sanitary Unit	For Male	For Female (A)
1.	Water Closet	One per 100 persons up to 400 persons; for over 400 add at the rate of one per 250 persons or part thereof.	Two for 100 persons up to 200 persons; over 200 add at the rate of one per 100 persons or part thereof.
2.	Ablution Taps	One in each W.C.	One in each W.C.
3.	Urinals	One for 50 persons or part thereof.	Nil
4.	Wash Basins	One per W.C. and urinal provided	One per W.C. provided

Note:

i) It may be assumed that two-thirds of the number are males and one- third females

ii) One water tap with drainage arrangements shall be provided for every 50 persons or part thereof in the vicinity of water closet and urinals.

* At least 50% of female WCs may be Indian pan and 50% EWC

8.6.2 Off-Site Treatment

As these locations are amidst habitation, it should be possible to connect them to the existing collection system, whether it is conventional sewerage or a decentralized sewerage or in its absence, provide a collection tank duly covered and transfer the contents by a sewer lorry to the existing disposal site/sites. In any case, on-site disposal of these public toilets shall be totally banned.

8.6.3 One-way See through Public Toilets

A key issue of public toilets amidst downtowns especially for women in software firms, etc. brings up the security concerns which may be possible to be got over by the pay and use type and see through mirror wall toilets reported to be in use abroad as in Figure 8.13 (overleaf). It shows the view of the roadside from inside the toilet, thereby facilitating a much needed security for the lone user in metros at odd hours.

8.7 COMMUNITY TOILETS AS DECENTRALIZED SEWERAGE

The community toilet is to be defined as a facility to be continuously used day in and day out by a fixed number of users at public locations or residential locations, and where a reasonable control over the number of users is possible.

Examples are those in economically weaker sections, educational institutions, sites of religious centres situated away from the main habitation, whether used daily or seasonally or for clusters of dwellings far away from sewerage and most important meeting locations, which are used in high numbers by the population though infrequently.

Source: http://www.toxel.com/tech/2009/05/27/transparent-public-toilets-from-switzerland/

Figure 8.13 A toilet reportedly in Switzerland affording security to the user to be aware of the surroundings through the one way mirror viewable from inside only

8.7.1 Norms

The norms for the number of seats, wash basins, etc., can be appropriated from the nearest category as in the NBC for railway stations, hostels, educational institutions, which border on community facilities. In respect of economically weaker sections, the design approach in Chapter 3 shall be followed to assess the volume of sewage. The issues already discussed under public toilets in respect to gender related and differently abled persons shall be considered here also. The designs for community toilets, which also include a washing section and bathing section have been developed for easy utility by the variety of users in common domain by the National Institute of Design and needs to be considered for suitable adoption. Their designs of pre-fabricated toilets and their networking, both in horizontal plane and vertical plane are worthy of adoption. The problem arises in assessing the needs for fairs, festivals and public meetings in locations where large number of people congregate though infrequently, like for example the foregrounds of Ana Sagar in Ajmer. The Norms for Toilet facilities for infrequent events is given in Table 8.4 herein.

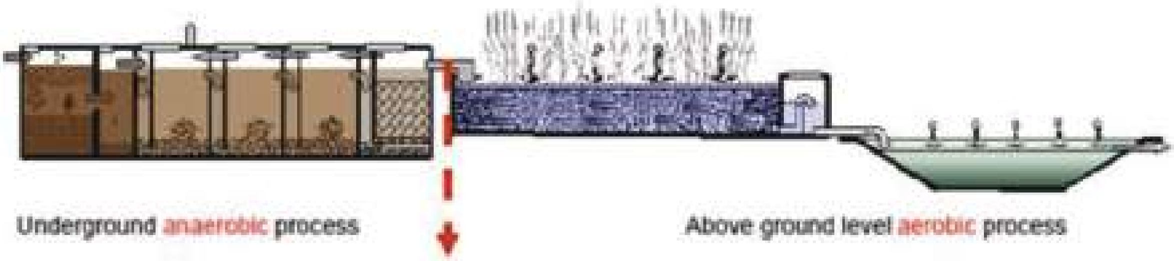
Table 8.4 Norms for Toilet facilities for infrequent events

Patrons	Male						Female			
	Toilets		Urinals		Sinks		Toilets		Sinks	
	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
<500	1	3	2	8	2	2	6	13	2	2
<1000	2	5	4	10	4	4	9	16	4	4
<2000	4	9	8	15	6	7	12	18	6	7
<3000	6	10	15	20	10	14	18	22	10	18
<5000	8	12	25	30	17	20	30	40	17	20

(a)- Where alcohol is not available; (b)-Where alcohol is available

8.8 DEWATS

This is an abbreviation of Decentralized Wastewater Treatment System (DEWATS) and has been assigned to a typical system of sewage treatment and resource utilization for greening in isolated habitations. The generalized treatment sequence is shown in Figure 8.14.



Source: ISPIRATION webpage

Figure 8.14 Schematic treatment process of DEWATS technology

This system is addressed to isolated habitations, where there is a need for non-mechanized and self-operating treatment technology given the premise that adequate land area is available and at reasonable distance from the habitation itself. Another aspect will be to group the toilets or at least bring the sewage from the various centres to the Dewats facility. The typical treatment units are:

a) Pre-treatment settler: retention time of about 2 hours; BOD reduction by about 30%
b) Anaerobic Baffled Tank Reactor: retention time of about 24 hours; BOD reduction by about 80%
c) Anaerobic filter: retention time of about 8 hours; BOD reduction by about 90%
d) Planted gravel filter: retention time of about 36 hours; BOD reduction by about 90%
e) Polishing pond.

These have been installed and commissioned in quite a few habitations in India and a compilation of the facility at the earthquake ravaged place of Bhuj in Gujarat is shown in Figure 8.15. The treatment process has its advantage of not dependant on mechanized units but requires relatively large areas away from the habitation and vector propagation control in the planted gravel filters and ponds.

Source: ISPIRATION webpage

Figure 8.15 Typical DEWATS treatment plant components

8.9 RECOMMENDATIONS

The decentralization concepts and technologies in sewage management need to be systematically investigated, with focus on its development and practical implementation in India. It may be borne in mind that the approach adopted for decentralized sewage management system (DSMS) is area specific and governed by number of issues and conditions prevailing, and also the methodology adopted and is influenced by (i) technical aspects as covered in this chapter and (ii) financial aspects, (iii) social aspects, (iv) environmental aspects, and (v) legal aspects that will be covered in the part C of the manual. It needs to be realized that this aspect and programme of decentralized sewerage is what the country needs urgently if the MDG is to be achieved especially in the peri-urban, rural and outlying areas and habitations. Accordingly, the following recommendations are brought up in deciding on implementing this.

1) As Incremental Sewerage - Decentralized Sewerage has an enormous significance by way of incremental sewerage and sanitation especially in newly developing peri-urban and rural settings, where conventional sewerage needs time to qualify itself physically and financially.

2) As a Combination of Collection System Options - It is the interim period from start of the layout to such time that underground conventional sewerage will qualify itself that is the bane of all environmental hazards of indiscriminate pollution. Ingenuity of a combination of decentralized collection systems and incremented treatment capacity of the STP are the remediations for the country as a whole.

3) Public Acceptance is the Key - However, with the mindset of the people that sewerage de facto implies only to the underground conventional sewerage, any deviation from a conventional system will require a public acceptance before implementation and as such, decentralized sewerage is not an exception. The Srirangam case study is an ideal example. Any attempt in starting a decentralized treatment there would have never seen the light of the day. This aspect must not be underestimated and hence, the public consultation process shall be announced well in advance in local media and repeated one more time giving notice of at least two weeks and making the venue as local marriage hall or public hall with adequate space and hired chairs and expenses being met by the local body. The technicalities are to be toned down and the benefits and costs alone need to be cited elaborately and the opinion elicited. Understandably, it will not be a full acceptance by all the habitation and there will be various cost recovery models thrown up for example, built-up area based on; number of families based, history of residence in terms of years, economically weaker sections, clusters, non-commercial Vs. commercial occupancies, etc., and these are to be debated to bring the issues on hand to a reasonable level of acceptance. The exercise needs to be repeated for a second time. At the end, if a consensus is reached, the project can be considered forward and if it still eludes, the best is pose a conventional sewerage system to JnNURM and await its turn.

4) Design of Collection System - With regard to design procedures of the collection systems, the Manning's formula holds good whether it be a circular conduit or a drain.

5) Design of Treatment Plants - With regard to treatment, the guidelines in Chapter 5 will however, apply as it becomes appropriate to each location.

CHAPTER 9: ON-SITE SANITATION

9.1 OVERVIEW OF ON-SITE SANITATION

The areas that are not served by piped sewer systems can adopt on-site systems. The treatment can be either on-site or off-site like in the case of septage management. These are interim measures till a decentralised or a full sewerage system is implemented.

It is strongly recommended that the town planning agencies / authorities / ULB / metropolitan development authorities earmark adequate spaces for laying of sewer lines, construction of SPS and STP.

9.1.1 On-site Sewage Treatment System

Unlike off-site centralized treatment (sewerage), on-site sewage treatment features individual and distributed treatment. The on-site treatment system includes a wide range of facilities, such as a basic sanitation facility like a pit latrine, a simple sewage treatment system that consists of a septic tank and a soak pit for anaerobic treatment, and an advanced facility like Johkasou that treats sewage by sophisticated methods.

In an urban area with high population density, an STP intensively treats sewage collected by pipes laid over a wide area. The on-site system treats sewage near the source.

Accordingly, the latter uses various kinds of treatment technologies according to treatment scale and the surrounding conditions. Sludge generated in each on-site treatment facility is collected and treated separately.

9.1.2 On-site Classification

This subsection summarizes the classification of toilets and on-site treatment methods as well as their features.

9.1.2.1 Historical

The historical pit latrines are rather rudimentary sanitation facilities atleast serving to contain the spread of faecal organisms from the night soil and bringing about interactions between soil organisms and feacal organisms in the pit. These have since been upgraded to various types as in Figure 8.3. In respect of community toilets, installations such as Dewats have also come up.

9.1.2.2 Simple Treatment Method

A septic tank system is a typical on-site treatment facility that consists of a septic tank and a soak pit and employs two technologies: the first is anaerobic treatment and the second is the methods of letting treated sewage penetrate the ground.

It shows stable performance, provided that the water temperature is kept suitable to digestion and the soil has good permeability.

However, the septic tank reduces BOD up to 50%, so if underground penetration is impossible due to high groundwater levels, rocky strata, non-availability of land for soak-pit, another method must be employed to hygienically treat sewage passing through the septic tank such as anaerobic filter and contact aeration. When this system is applied to an urban area with high population density, care must be taken not to have a negative effect on the surrounding environment.

9.1.2.3 Advanced Treatment System

Conventional septic tanks system, if properly designed and with proper septage removal frequency can effectively remove about 40-50% BOD and 50-70% TSS. However, due to partial treatment and associated health hazards the effluent can only be discharged into soak pits. Due to recent groundwater pollution related episodes, unavailability of space for soak pits and under rocky strata, soak pits are avoided and the effluent is commonly discharged to open stormwater drains. Hence, it is causing another type of pollution menace such as unsightly conditions, eutrophication, odour, vector and water related diseases.

Some of the interim solutions are the improved design of septic tanks such as anaerobic baffled reactor or the post treatment of septic tank effluents by anaerobic filters. Both configurations can partially solve the pollution related problems by increasing the overall BOD removal to more than 70%. These systems can lessen the burden of organic pollution without any extra energy cost. The capital cost of these systems may not be more than 20-30% of the conventional septic tank cost. Nevertheless, due to the limitation of anaerobic sewage treatment, these systems cannot bring down the BOD and TSS levels up to the national effluent discharge standards. Hence, alternate solution could be the aerobic type post treatment such as contact aeration. This system can bring down effluent BOD to less than 30 mg/l and TSS to less than 50 mg/l but at the expense of electrical power requirement for 24×7 operating air blower with standby equipment and standby power.

One such system is the Japanese type Johkasou system. This system is an integrated septic tank-anaerobic filter-contact aeration-final settling tank and effluent disinfection facility. However, due to higher cost considerations, these systems may be affordable only in very fragile environment. These systems have also been upgraded for even nitrogen removal by providing internal recirculation. The detail of these systems is provided in the following sections. There are many other similar package treatment systems elsewhere that can also be used.

9.2 THE PROHIBITION OF EMPLOYMENT AS MANUAL SCAVENGERS AND THEIR REHABILITATION ACT, 2013

The aforesaid act was notified by the GOI in September 2013. The act shall come into force from 6th December 2013. The text of the act as in the Gazette is in Appendix A 1.1. The time frame specified under the Act for the fulfilment of responsibilities and carrying out certain activities are mentioned in Appendix A 1.2.

9.3 INTERIM MEASURES

There are various on-site systems which can be used but with a caution to prevent ground water and surface water pollution due to indiscriminate disposal of sewage from these on-site systems.

9.3.1 Public and Community Toilets

A public toilet, a kind of common toilet installed in stations and on streets, is open to everyone rather than specified users. In contrast, a community toilet has limited users such as residents. These common toilets are controlled by local governments, residents, or private sector organizations. A common toilet normally has two sections: one is for males and the other is for females. In addition, another section special to persons in a wheelchair (unisex) is sometimes provided.

In general, an on-site common toilet includes a special sewage treatment facility such as a septic tank. The flow rate of sewage to be treated is derived from the total number of users based on how many toilet bowls are installed and how frequently they are used.

The toilet is equipped with a water supply unit, a ventilator, and a lighting device. Figure 9.1 shows example arrangements of faeces, urine, and hand-washing units.

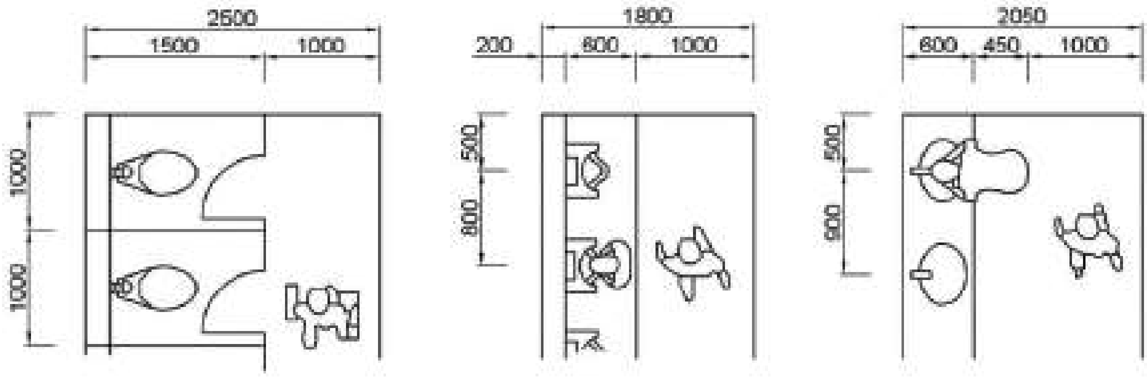


Figure 9.1 Examples of common toilet arrangements

Example of design

The following shows an example of estimating the number of public toilet users.

Basic Setting

Number of toilet bowls [c]: 10 (in total)

Total number of users [n]

$$n = 16c$$
$$= 16 \times 10 = 160$$

9.3.2 Mobile Toilet

Mobile toilets are temporarily installed in places where there is no toilet, such as shelters during natural disaster, venues for events, and construction sites, or where the number of existing toilets is short. A mobile toilet box has a tank for storing excreta in its lower part. If the tank is full, a vacuum tanker collects the stored sewage. Each toilet has a single room or multiple rooms with a hand washing unit, which is selected according to the flexibility of installation sites and ease of transport by a truck. In addition, there is a mobile flush toilet that is equipped with a water tank and a pedal.

Stepping on the latter activates a manual pump to cause washing water to flow. The box is made by assembling fiberglass-reinforced plastic (FRP) side panels, so its weight is light. Local governments keep these toilets to prepare for disasters and events, or rental companies lease them. The mobile toilet features easy installation work on the ground. Figure 9.2 shows a mobile toilet having faeces, urine, and hand washing units.

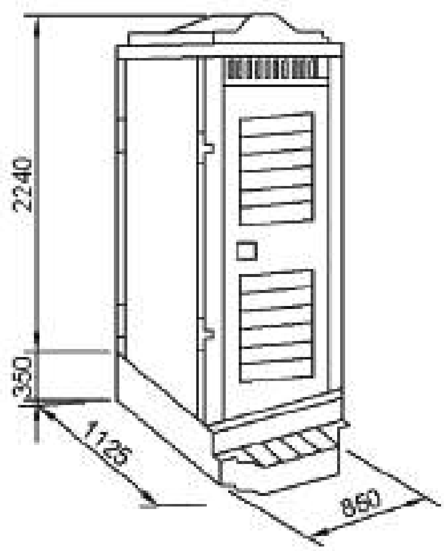


Figure 9.2 Mobile toilet

9.3.3 Pour Flush Water Seal Latrine

In a conventional water flush latrine, the excreta is normally flushed with 10 to 14 litres of water from a cistern. In a pour flush latrine, as the name suggests, excreta is hand flushed by pouring about 1.5 to 2.0 litres of water. These pour-flush leaching pit latrines were first developed in India in mid-forties with a single leach pit and squatting pan placed over it. When the pit in use gets filled up another pit is dug and the squatting slab is removed and placed over the new pit. The first pit is covered with earth and the excreta is allowed to digest. After one or two years, the digested excreta is used as manure.

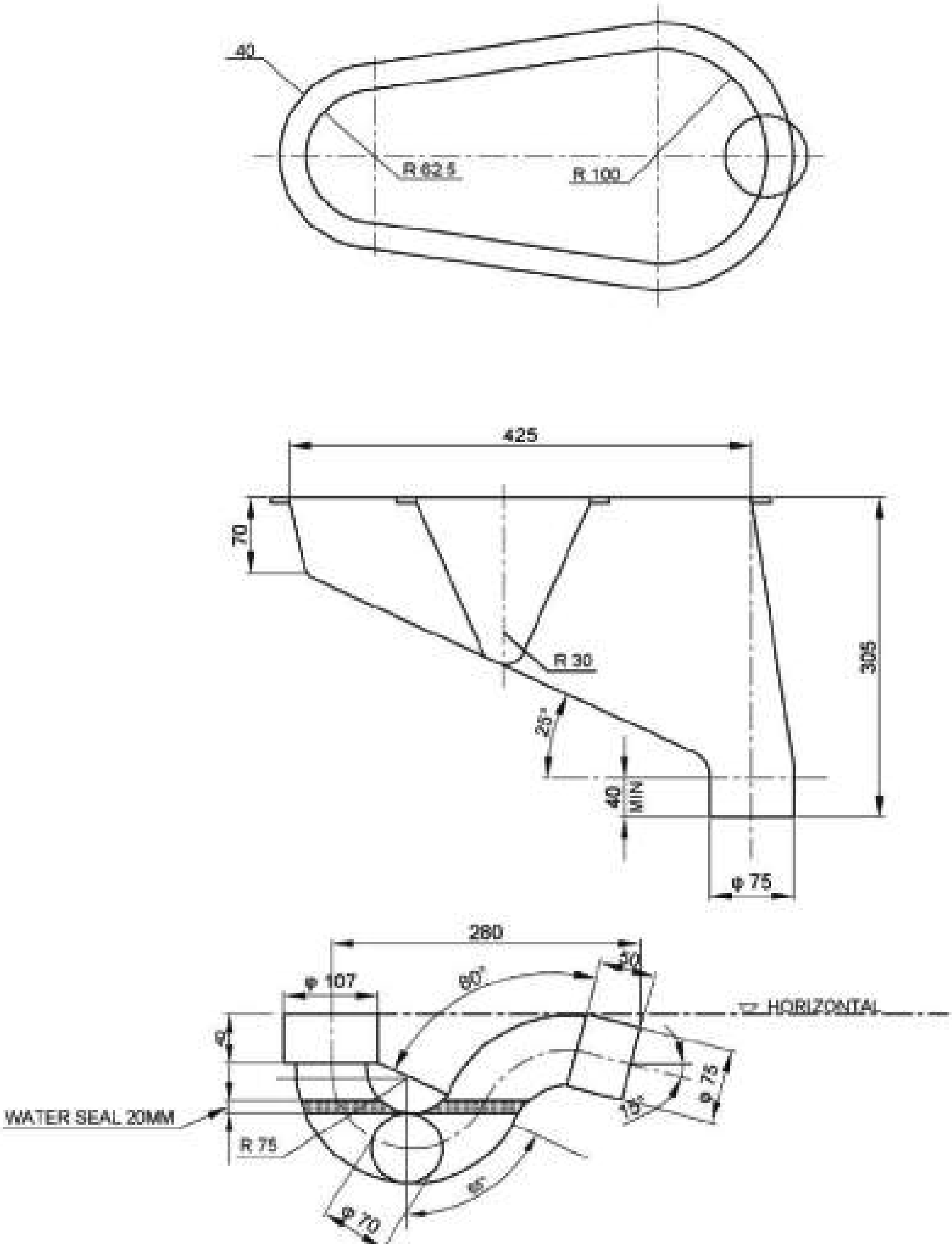
In the late fifties, a modified design of the system was developed. In this system the leach pit is kept away from the seat instead of placing it underneath the pan. In a single pit system, desludging has to be done almost immediately after the pit has been filled up to enable its re-use; this involves handling of fresh and undigested excreta containing pathogens which is a health hazard. Single leach pit is appropriate only if it is desludged mechanically by a vacuum tanker. To overcome this shortcoming, the twin-pit design was introduced and in this case when one pit is full, the excreta is diverted to the second pit. The filled up pit can be conveniently emptied after 1.5 to 2 years, when most of the pathogens die off. The sludge can safely be used as manure. Thus the two pits can be used alternately and perpetually.

With simple care, pour-flush water-seal latrine is a very satisfactory and hygienic sanitation system and hence it can be located inside the house since the water-seal prevents odour and insect nuisance from the pit.

9.3.3.1 Design and Materials

9.3.3.1.1 Squatting Pan, Trap, Footrests, and Connecting Drain

The squatting pan is of special design with steep bottom slope 25 - 28° and a trap having 20 mm water seal set on a cement concrete floor. The hydraulic design of the pan is such that the human excreta can be flushed by pouring only 1.5 to 2 litres of water. The squatting pan and trap design details are shown in Figure 9.3.



Source: CPHEEO, 1993

Figure 9.3 Squatting pan and trap

The squatting pan can be of ceramic or glass reinforced plastic (GRP), High Density Polyethylene (HDPE) or Poly Vinyl Chloride (PVC), Polypropylene (PP), Cement mosaic or even concrete. The squatting pan is connected to the leaching pit through a trap and a pipe or covered drain. The design and material details for latrine units squatting pan, trap, footrest and the connecting drain are summarised below in Table 9.1.

Table 9.1 Material and other details for latrine unit

No.	Squatting Pan	Trap	Footrests	Connecting Drain
1	Horizontal length of pan should be at least 425 mm and longitudinal bottom slope 25 -28°	It should be 70 to 75 mm with 20 mm water seal	It should be 250×125 mm with 15 to 20 mm height	May be non-pressure pipes of PVC minimum 75 mm dia
2	Material: Ceramic, FRP, PP, HDPE, PVC, Cement mosaic or Cement concrete	Fibre Glass, Ceramic, HDPE or CC traps	Ceramic or concrete with mosaic finish brick or stone	Bricks or stone semi-circular bottom
3	Should conform to IS: 2556 (Pt. III), IS: 11246, GRP Sq. Pan	Should conform to IS: 2556 (Pt. XIII)	Should conform to IS: 2556 (Pt. X)	Slope should be 1 in 5 to 1 in 15 as per site conditions
4	(A)			(B)

(A)- Ceramic, FRP, PP are smooth and require less water for flushing. FRP cheaper, lighter and easier to transport than the other

(B)- The inlet pipe should project 100 mm in to the leach pit.A junction chamber of 250×250 mm should be provided in case of pipe

Source: CPHEEO, 1993

9.3.3.1.2 Leach Pits/Twin Pit Latrine

Leach pits serve a dual function of (a) storage and digestion of excreted solids and b) infiltration of the waste liquids and are therefore, to be designed on the basis of the following parameters:

- Sludge accumulation rate
- Long term infiltration rate of the liquid fraction across the pit soil interface
- Hydraulic loading on the pit
- Minimum period required for effective pathogen destruction
- Optimal pit emptying frequency.

9.3.3.1.2.1 Sludge Accumulation Rate

The sludge accumulation rate is a function of a wide range of variables including water table level, pit age, water and excreta loading rates, microbial conditions in the pit, temperature and local soil conditions and the type of material used for anal cleansing.

The leach pit is classified as wet or dry depending on whether the ground water table is above the bottom of pit or below. In dry pits, the pit volume needed is calculated on the basis of solids accumulation rate, but in wet pits though the sludge accumulation rate is lower - the sludge digestion rate is high in the presence of water, yet volume of pit has to be increased to prevent flooding due to surcharge of pits. The sludge accumulation rates given below in Table 9.2 may be used to calculate the pit volume.

Table 9.2 Sludge accumulation rates

Material used for anal cleansing	Effective Volume in m^3 per Capita per Year (A)		
	Pit under dry conditions	Pit under Wet conditions	
		With successive desludging intervals	
		2 years	3 years
Water	0.04	0.095	0.067
Soft Paper	0.053	0.114	0.080

(A) Effective Volume is the volume of the pit below the invert level of pipe or drain.

Source:CPHEEO, 1993

9.3.3.1.2.2 Long Term Infiltration Rate

On account of clogging of soil pores around the leach pits, the long term infiltration capacity (after clogging) of the soil is always less than the natural percolative capacity. The recommended design values of the long term infiltrative capacity can be derived for the typical soil conditions as given below in Table 9.3.

Table 9.3 Long term infiltration rates of different types of soils

No.	Soil type	litres / sqm / day
1	Sand	50
2	Sandy loam, loams	30
3	Porous silty loams, porous silty, silty clay loams	20
4	Compact silty loams, compact silty clay loams, clay	10

Source: CPHEEO, 1993

9.3.3.1.2.3 Hydraulic Loading

The hydraulic loading rate is the total volume of liquids entering the leach pit and is expressed in litres per day although it is often more convenient to consider per capita loadings (litres per capita per day). For computing the pit hydraulic loading, sewage contribution of 9.5 litres per day per person, including water used for ablutions and flushing, urine, excreta, etc., can be taken as the basis.

The outer surface area (perimeter) of the pit from pit bottom to invert level of pipe or drain is to be considered for infiltration. The pit bottom is not taken into account as it gets clogged in course of time. The infiltration area required is the total flow in the pit per day divided by the long term infiltrative rate of the soil where pits will be located. The infiltrative area of leach pits, sized on the basis of sludge accumulation rate should conform to the computed infiltrative area.

9.3.3.1.2.4 Pathogen Destruction

After a period of almost all pathogens viruses, bacteria, protozoa and helminths die off in the leach pit or in the surrounding soil, but not Ascaris Lumbricoides (the large human round-worm) particularly if the leach pit is wet. After about one or one and a half years of storage in the pit, it may not be hazardous to handle the contents of the pit for use as manure.

9.3.3.1.2.5 Optimal Pit Emptying Frequency

The minimum acceptable design interval between successive manual desludging of each twin leach pit could be one-and-a-half-years. However, to provide a reasonable degree of operational flexibility, it is desirable to provide three years storage volume in urban areas and two years in rural areas.

9.3.3.1.2.6 Size of Pits

Sizes of leach pits, [designed as above for different number of users, using water ablution and for different subsoil water levels], with 3 years sludge storage volume, are in Table 9.4.

Table 9.4 Size of leach pits

	5 Users		10 Users		15 Users	
	Dia	Depth(A)	Dia	Depth(A)	Dia	Depth(A)
Dry Pits	900	1,000	1,100	1,300	1,300	1,400
Wet Pits	1,000	1,300	1,400	1,400	1,600	1,500

Note: (A) Depth from bottom of pit to invert level of incoming pipe or drain (all dimensions in mm)

Source: CPHEEO, 1993

The surface area of these is adequate for soils with long term infiltrative rate down to 20 l/m^2/day. The above depths should be increased by 300 mm to provide a free board depth of pit from invert level of pipe or drain to bottom of pit cover.

9.3.3.1.2.7 Design of Pits under Different Conditions

A typical pour flush latrine with circular pits is shown in Figure 9.4.

In water logged area: The pit top should be raised by 300 mm above the likely level of water above ground level at the time of water logging. Earth should then be filled well compacted all round the pits up to 1.0 m distance from the pit and up to its top. The raising of the pit will necessitate raising of latrine floor also. A typical pour flush latrine in water logged areas is shown in Figure 9.5.

In high subsoil water level: Where the subsoil water level rises to less than 300 mm below ground level, the top of the pits should be raised by 300 mm above the likely subsoil water level and earth should be filled all round the pits and latrine floor raised as stated above. A typical pour flush latrine with leach pits in high subsoil water level is shown in Figure 9.6

In rocky strata: In rocky strata with soil layer in between, the leach pits can be designed on the same principle as those for low subsoil water level and taking the long term infiltrative capacity as 20 $l/m^2/d$. However, in rocks with fissures, chalk formations, old root channels, pollution can flow to very long distances; hence these conditions demand careful investigation and adoption of pollution safeguards as stated in paragraph below.

In black cotton soil: Pits in black cotton soil should be designed taking infiltrative rate of 10 $l/m^2/d$. However a vertical fill (envelope) 300 mm in width with sand, gravel or ballast of small sizes should be provided all round the pit outside the pit lining.

Where space is a constraint: Where circular pits of standard sizes cannot be constructed due to space constraints, deeper pit with small diameter (not less than 750 mm), or combined oval, square or rectangular pits divided into two equal compartments by a partition wall may be provided. In case of combined pits and the partition wall should not have holes. The partition wall should go 225 mm deeper than the pit lining and plastered on both sides with cement mortar. A typical pour flush latrine with combined pits is shown in Figure 9.7

Design example of leach pit is given in Appendix A.9.1.

9.3.3.2 Construction of Pour Flush Latrine

9.3.3.2.1 Squatting Pan and Trap

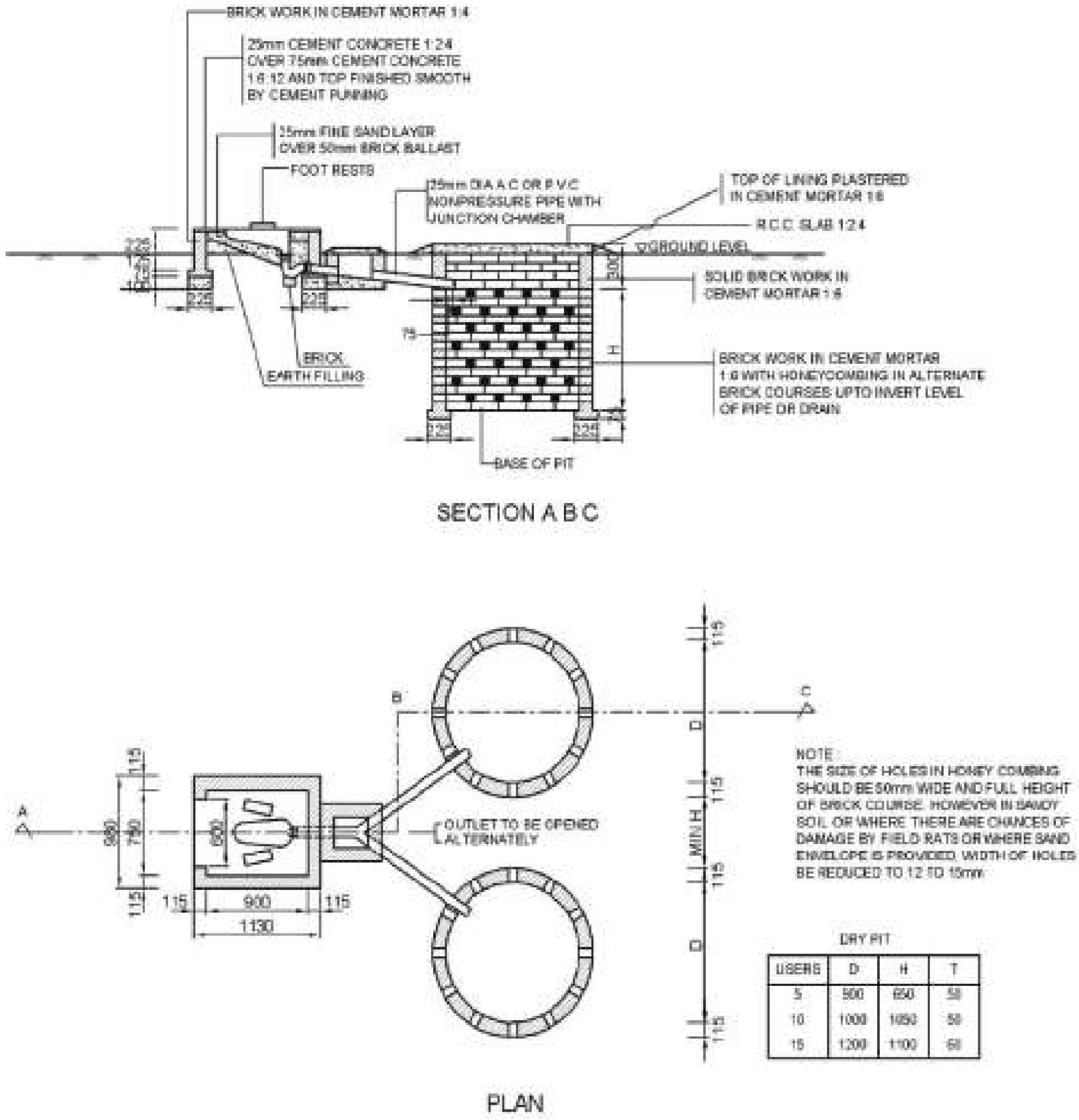
The pan could be ceramic, GRP, PVC, PP, Cement Concrete or Cement Mosaic. Ceramic are the best but costliest. Mosaic or cement concrete pans have the advantage that these can be manufactured locally by trained masons but the surface tends to become rough after long use. Their acceptance is less compared to other types. Traps for ceramic pans are made of the same material but in case of GRP pans, HDPE traps are used. For mosaic pans, traps are of cement concrete.

9.3.3.2.2 Foot Rests

These can be of ceramic, cement concrete, cement mosaics or brick plastered. The top of the footrest should be about 20 mm above the floor level and inclined slightly outwards in the front.

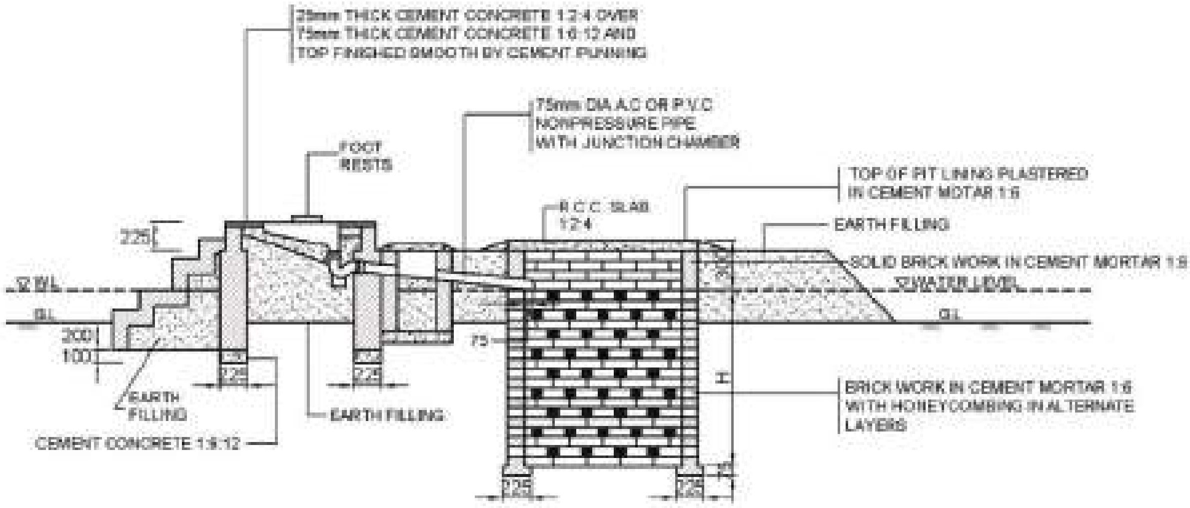
9.3.3.2.3 Pit Lining

The pits should be lined to avoid collapsing. Bricks joined in 1:6 cement mortar are most commonly used for lining. Locally manufactured bricks should be used wherever available. Stones or laterite bricks cement concrete rings could also be used depending upon their availability and cost. However, for ease of construction, use of concrete rings will be advantageous where the subsoil water level is above the pit bottom.

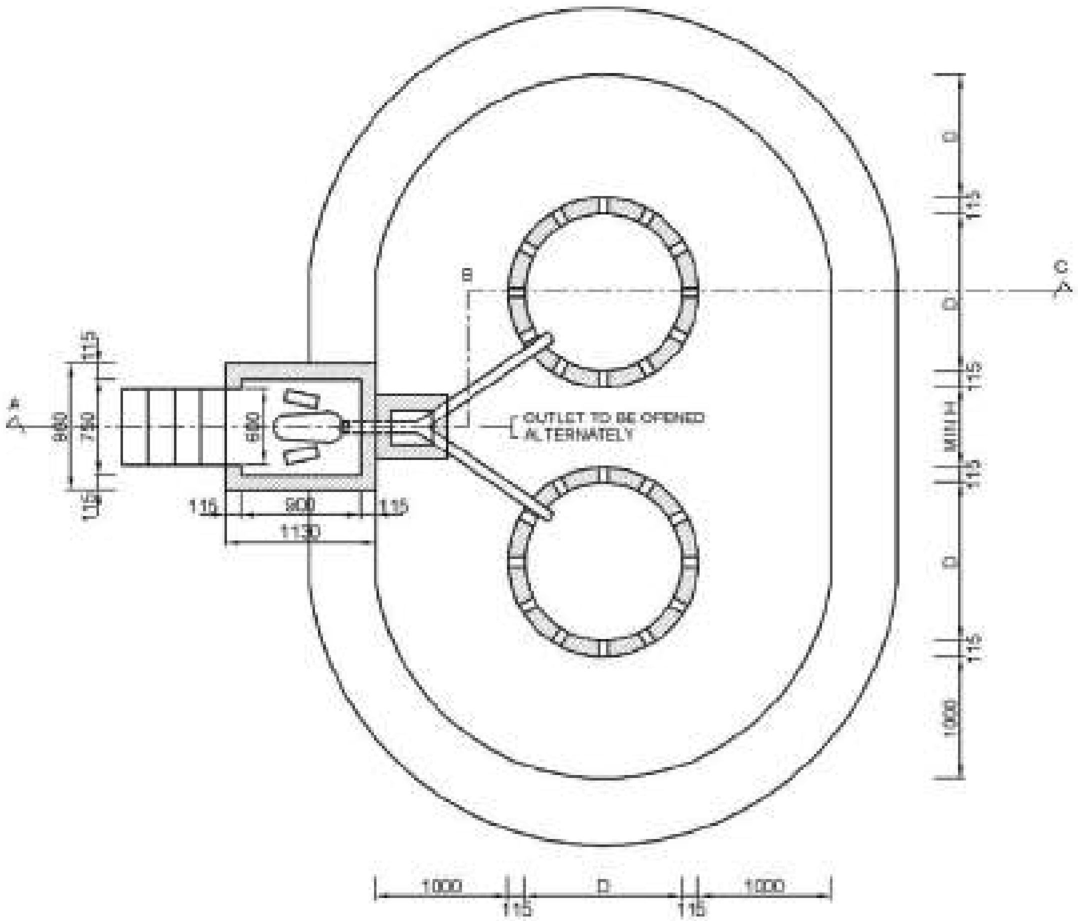


Source: CPHEEO, 1993

Figure 9.4 Pour flush latrine with circular pits



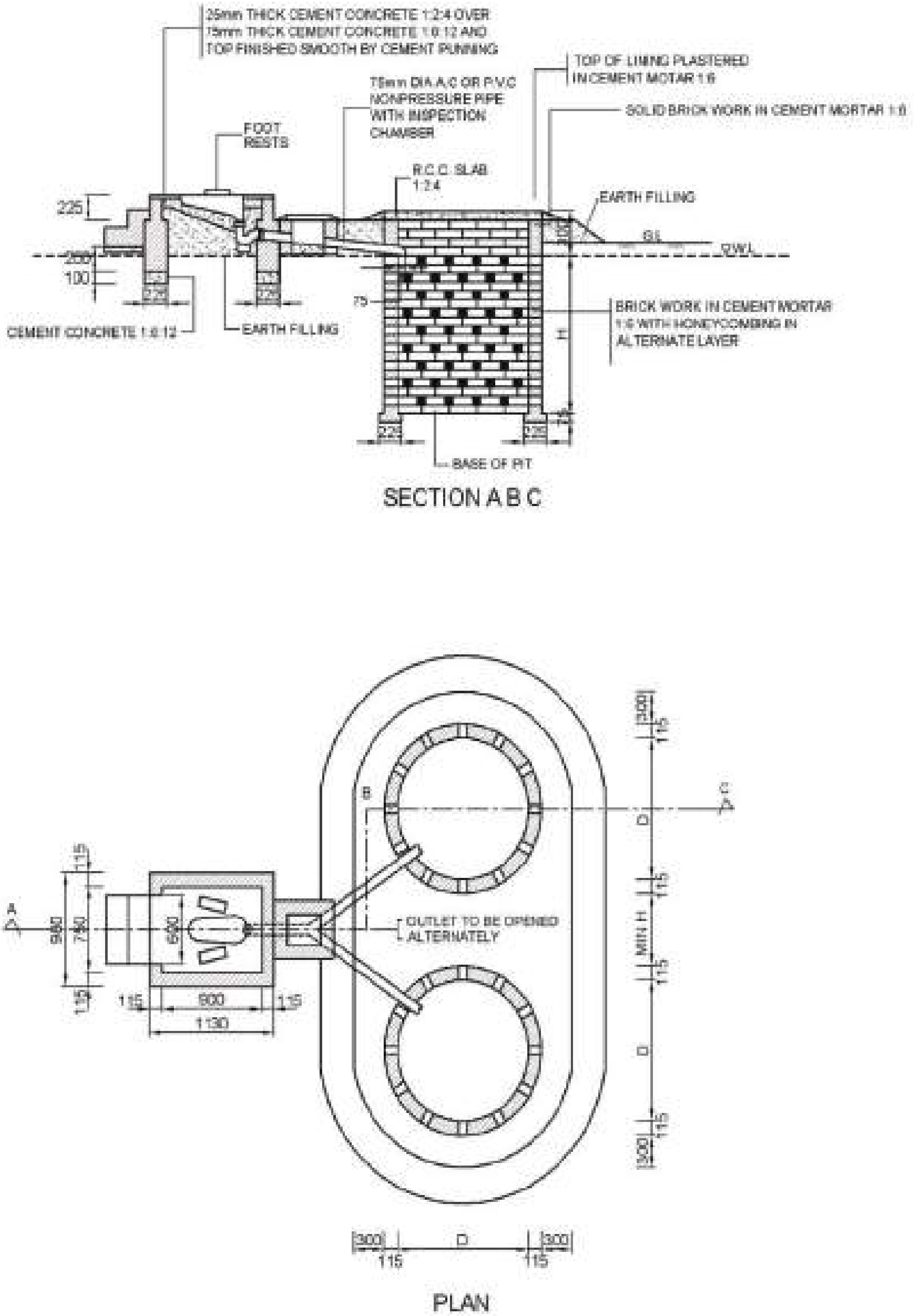
SECTION A B C



PLAN

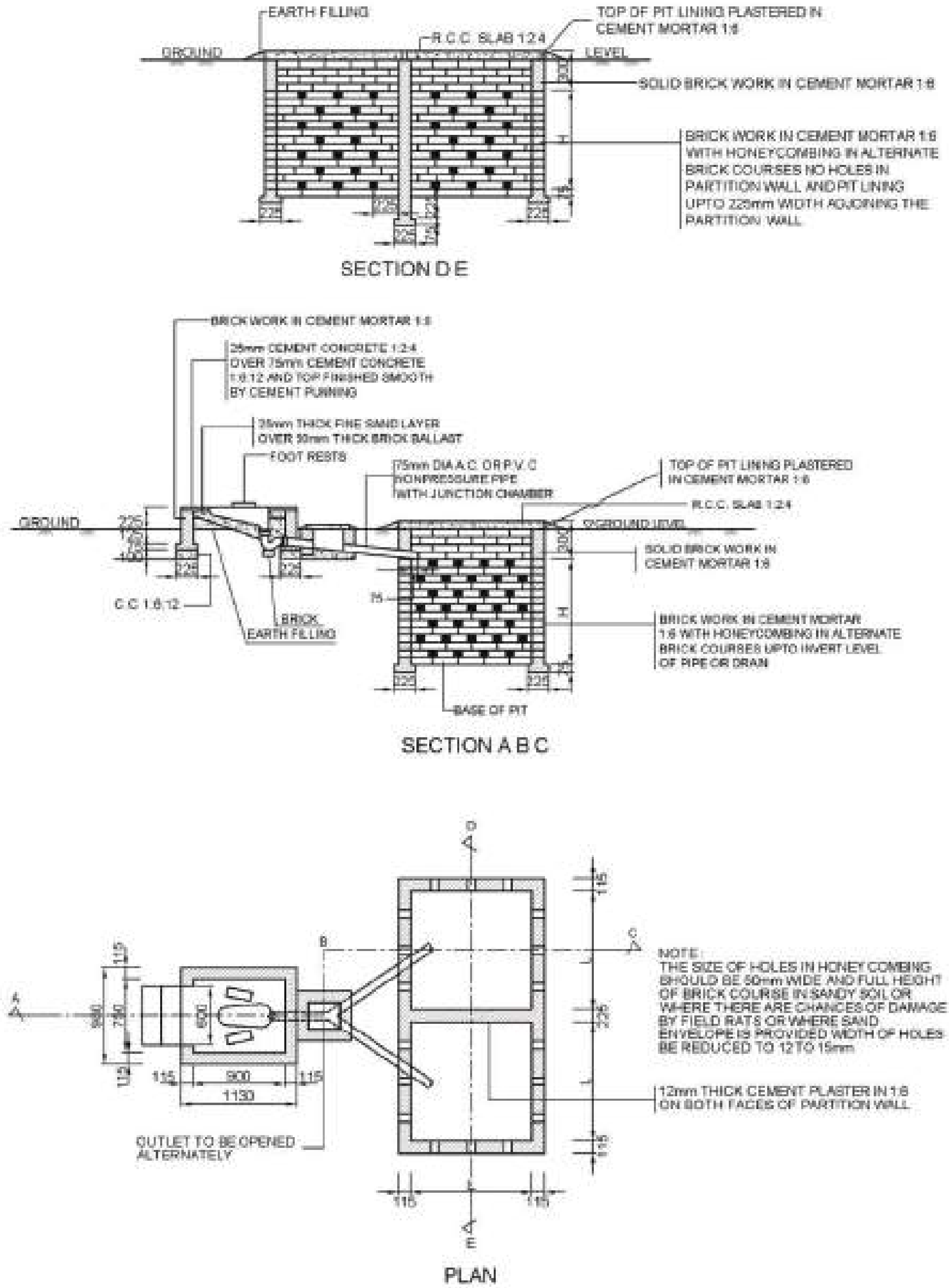
Source:CPHEEO, 1993

Figure 9.5 Pour flush latrine in water logged areas



Source:CPHEEO, 1993

Figure 9.6 Leach pits in high subsoil water level



Source:CPHEEO, 1993

Figure 9.7 Pour flush latrine with combined pits

The lining in brick work should be 115 mm thick (half brick) with honey combing up to the invert level of incoming pipe or drain; the size of holes should be about 50 mm wide up to the height of the brick course. For ease of construction, holes should be provided in alternate brick courses. In case the soil is sandy and sand envelope is provided, the width of openings should be reduced to 12 to 15 mm. Where foundation of building is close to the pit, no holes should be provided in the portion of lining facing the foundation and in rest of the lining 12 to 15 mm wide holes should be provided. The lining above the invert level of pipe or drain up to the bottom of pit's cover should be in solid brick work, i.e., with no openings.

The concrete rings used for lining should be 50 mm thick, about 450 mm in height and of required diameter in 1:3:6 cement concrete and have 40 mm circular holes staggered about 200 mm apart. The rings are not jointed with mortar, but are put one over the other. The rings above the invert level of pipe or drain should not have holes and are jointed with cement mortar.

9.3.3.2.4 Pit Bottom

Except where precautions are to be taken to prevent pollution of water sources, the pit bottom should be left in natural condition.

9.3.3.2.5 Pit Cover

Usually RCC slabs are used for covering the pits, but depending upon the availability and cost, flag stones can also be used. The RCC Slab may be centrally cast in pieces for convenience of handling.

9.3.3.2.6 Leach Pit Connection

The toilet pan is connected to the pit through a 75 mm brick channel of 'U' shape covered with loosely jointed bricks or 75 mm dia AC or PVC non-pressure pipe laid in 1:15 gradient. In case pipes are used, a chamber of minimum size 225 × 225 mm is provided at the bifurcation point to facilitate cleaning and allowing flow to one pit at a time. In case of drain, 'Y' portion of the drain serves the purpose by taking out the brick cover.

9.3.3.3 Pollution Safeguards

In order to reduce the pollution risk of ground water and water sources, the following safeguards should be taken while locating the pits.

9.3.3.3.1 Safe Distance from Drinking Water Sources

In dry pits or unsaturated soil conditions, i.e. where the height between the bottom of the pit and the maximum ground water level throughout the year is 2 m and more.

a) The pits can be located at a minimum distance of 3 m from the water sources such as tube wells and dug wells if the effective size (ES) of the soil is 0.2 mm or less, and

b. For coarser soils (with ES greater than 0.2 mm) the same distance can be maintained if the bottom of the pit is sealed off by an impervious material such as puddle clay or plastic sheet and 500 mm thick envelope of fine sand of 0.2 mm effective size is provided around the pit.

In wet pit saturated soil conditions, i.e. where the distance between the bottom of the pit and the maximum ground water level during any part of the year is less than 2 m,

a. The pits can be located at a minimum distance of 10 m from the water sources such as tube wells and dug wells if the ES of the soil is 0.2 mm or less, and

b. For coarser soils (with ES more than 0.2 mm), minimum distance of 10 m can be maintained if the pit is sealed off by an impervious material such as puddle clay or plastic sheet with 500 mm thick envelope of fine sand of 0.2 mm, effective size provided all round the pit.

9.3.3.3.2 Safe Distance from Water Supply Mains

Lateral distance between the leach pit and the water mains should be at least 3 m provided the water table does not rise during any part of the year above the pit bottom and the inlet of the pipe or drain to the leach pit is below the level of water main. If the water table rises above the bottom of the pit, then the safe lateral distance should be kept as 8 m. If this cannot be achieved, then the pipes should be completely encased to a length of at least 3 m on either side of the pit.

When the pits are located either under the foot path or under the road, or the water supply main is within a distance of 3 m from the pits, the invert of the inlet pipe should be kept at least 1 m below the ground level. This would ensure that the liquid level in the pits does not reach the level of the water main as the water mains are generally laid at 0.9 m depth.

The water pipe should not cut across the pit, but where this is unavoidable; the water pipe should be completely encased for length of 3 m on either side of the pit including the portion across the pit to prevent infiltration or exfiltration.

A study is reported by National Institute of Technology, Calicut, Kerala in respect of safe distance in laterite type of soils (Biju.et.al.2011)

The study area had houses with either the septic tank-soak pit system or pit latrines, the latter being more common with open wells as the source of water at 1.2 m to 2.4 m below ground in laterite soil. The horizontal distance between well and the soak pit / pit latrines varied from 5 m to 31 m. The MPN of total coliform from nearly 35 wells was studied and it was found that the number of total coliform correlated with the length of a specific parabolic curve connecting the soak pit / pit latrine and the well. This relationship was used to calculate the safe distance between the soak pit / pit latrine and open well so that the total coliform was not exceeding the MoEF classification of class “A” water in the well water and which is “Drinking water source without conventional treatment but after disinfection” at total coliform of not exceeding 50/100 ml. The distance evaluated was 21 m, where the water table rises to the level of soak-pit / pit latrine and the well.

9.3.3.3.3 Location of Pits

The ideal position for locating the pits is that the pits are placed symmetrically at the backside of pan. The pits may be located within the premises, under footpath or narrow lanes or under the road.

The minimum space between two pits should be equivalent to at least the effective depth (distance between the bottom of the pit and invert level of pipe or drain. Spacing can be reduced by providing an impervious barrier like cut off screen or puddle wall.

In many cases, the space available for constructing leach pits may be small and placement of pits near existing structure may be unavoidable. The digging of pits and subsequent seepage may disturb the soil around the pits.

The safe distance of the leach pits from the foundations of existing building depends upon the soil characteristics, depth as well as type of foundation of the structure, depth of the leaching pits etc., and varies from 0.2 to 1.3 m.

However, in cases where the leach pits are quite close to the existing building foundation, the opening in the brick work lining of the leach pit may be reduced to 12 to 15 mm.

Where the bottom of the pit is submerged below the maximum ground water level:

i. The top of the pits should be raised above the ground level, if necessary, so that the pipe into the pit is at least 0.75 m above the maximum ground water level.

ii) The sand envelope is taken up to 0.3 m above the top of the inlet pipe and confined suitably to exclude any surface drainage including rain water directly entering the sand envelope.

iii) In mound type latrines, 1 m high earth filling be provided at least 0.25 m beyond the sand envelope with the edges chamfered to lead away the rain or surface water, and

iv) The honeycomb brick work for the pit lining should be substituted by brick work in cement mortar 1:6 with open vertical joints, i.e. without mortar. Where sand is not available economically, local soil of effective size of 0.2 mm can also be used.

9.3.3.3.4 Subsoil Conditions

In depression and waterlogged areas, location of pits should be avoided, as far as possible, in depression where sewage or rain water is likely to remain collected all round and over the pits. If it cannot be avoided or the pits are to be constructed adjacent to ponds or tanks, then the top of pits should be raised to 0.6 m to 0.8 m above the ground level and earth filling should be done all around the pits up to a distance of 1.5 m right up to the pit top.

The raising of pit may necessitate raising the latrine floor also.

9.3.3.4 Night Soil Digesters

The night soil can be anaerobically digested either alone or in combination with cattle dung. It is rich in nitrogen and phosphorus in comparison to cow dung.

The characteristics of night soil are different from the cow dung and are mentioned in Table 9.5 (overleaf).

Table 9.5 Characteristics of night-soil and cow-dung

No.	Characteristics	Night Soil	Cow Dung
1	Moisture content, %	85 - 90	74 - 82
2	Volatile solids as % of Total Solids	80 - 88	70 - 80
3	Total Nitrogen as N, % on dry basis	3 - 5	1.4 - 1.8
4	Total Phosphorus as P_2O_5, % on dry basis	2.5 - 4.4	1.1 - 2.0
5	Potassium as K_2O, % on dry basis	0.7 - 1.9	0.8 - 1.2

Source: CPHEEO, 1993

9.3.3.4.1 Design Criteria

The design criteria for night soil digester are listed in Table 9.6.

Table 9.6 Design criteria and performance parameters for digester

No.	Item	Magnitude
1	Volumetric Organic loading, kg VS/m^3 d	1.6
2	Hydraulic residence time, d	25-30
3	Solids concentration of slurry fed to digester, %	5
4	Volatile solids destroyed during digestion, %	45-55
5	Gas yield, m^3/kg of VS added in m^3/capita/d	0.5
6	m^3/capita/d	0.034

Source: CPHEEO, 1993

The night soil digesters are constructed in a similar manner as anaerobic digesters and essentially consist of the following components:

i. Inlet tank with a feed pipe leading to digester

ii. Digester tank with fixed or floating dome for gas collection

iii. Outlet pipe from digester discharging digested slurry into a masonry chamber.

9.3.4 Conventional Septic Tank

A septic tank is a combined sedimentation and digestion tank where the sewage is held for one to two days. During this period, the suspended solids settle down to the bottom. This is accompanied by anaerobic digestion of settled solids (sludge) and liquid, resulting in reasonable reduction in the volume of sludge, reduction in biodegradable organic matter and release of gases like carbon dioxide, methane and hydrogen sulphide. The effluent although clarified to a large extent, will still contain appreciable amount of dissolved and suspended putrescible organic solids and pathogens.

Therefore, the septic tank effluent disposal merits careful consideration. Due to unsatisfactory quality of the effluent and also the difficulty in providing a proper effluent disposal system, septic tanks are recommended only for individual homes and small communities and institutions, whose contributory population does not exceed 300. For larger communities, septic tanks may be adopted with appropriate effluent treatment and disposal facilities. However, in both cases the sewage from the septic tank should be discharged into a lined channel constructed along with storm water drain as an interim measure till a proper sewerage system is laid. The outfall from such drains should be connected to a decentralised or centralised sewage collection system.

9.3.4.1 Design

Several experiments and performance evaluation studies have established that only about 30% of the settled solids are anaerobically digested in a septic tank. In case of frequent desludging, which is necessary for satisfactory effluent quality, still lower digestion rates have been reported. All these studies have proved that when the septic tank is not desludged for a longer period i.e., more than the design period, substantial portion of solids escape with the effluent. Therefore, for the septic tank to be an efficient suspended solids remover, it should be of sufficient capacity with proper inlet and outlet arrangements. It should be designed in such a way that the sludge can settle at the bottom and scum accumulates at the surface, while enough space is left in between, for the sewage to flow through without dislocating either the scum or the settled sludge.

Normally, sufficient capacity is provided to the extent that the accumulated sludge and scum occupy only half or maximum two-thirds the tank capacity, at the end of the design storage period.

Experience has shown that in order to provide sufficiently quiescent conditions for effective sedimentation of the suspended solids, the minimum liquid retention time should be 24 hours. Therefore, considering the volume required for sludge and scum accumulation, the septic tank may be designed for 1 to 2 days of sewage retention.

The septic tanks are normally rectangular in shape and can either be a single tank or a double tank. In case of double tank, the effluent solids concentration is considerably lower and the first compartment is usually twice the size of the second. The liquid depth is 1-2 m and the length to breadth ratio is 2-3 to 1.

Recommended sizes of septic tanks for individual households (up to 20 users) and for housing colonies (up to 300 users) are given below in Table 9.7 and Table 9.8 respectively.

Table 9.7 Recommended size of septic tank up to 20 users

No. of Users	Length (m)	Breadth (m)	Liquid depth (m) (cleaning interval of)	
			2 years	3 years
5	1.5	0.75	1.0	1.05
10	2.0	0.90	1.0	1.40
15	2.0	0.90	1.3	2.00
20	2.3	1.10	1.3	1.80

Note 1: The capacities are recommended on the assumption that discharge from only WC will be treated in the septic tank

Note 2: A provision of 300 mm should be made for free broad.

Note 3: The sizes of septic tank are based on certain assumption on peak discharges, as estimated in IS: 2470 (part 1) and while choosing the size of septic tank exact calculations shall be made.

Source: CPHEEO, 1993

Table 9.8 Recommended size of septic tank for housing colony upto 300 users

No. of Users	Length (m)	Breadth (m)	Liquid depth (cleaning interval of)	
			2 years	3 years
50	5.0	2.00	1.0	1.24
100	7.5	2.65	1.0	1.24
150	10.0	3.00	1.0	1.24
200	12.0	3.30	1.0	1.24
300	15.0	4.00	1.0	1.24

Note 1: A provision of 300 mm should be made for free board.

Note 2: The sizes of septic tanks are based on certain assumptions on peak discharges, as estimated in IS: 2470 (Part 1) and while choosing the size of septic tank exact calculations shall be made.

Note 3: For population over 100, the tank may be divided into independent parallel chambers of maintenance and cleaning.

Source: CPHEEO, 1993

9.3.4.2 Construction Details

The inlet and outlet should not be located at such levels where the sludge or scum is formed as otherwise, the force of water entering or leaving the tank will unduly disturb the sludge or scum. Further, to avoid short-circuiting, the inlet and outlet should be located as far away as possible from each other and at different levels. Baffles are generally provided at both inlet and outlet and should dip 25 cm to 30 cm into and project 15 cm above the liquid. The baffles should be placed at a distance of one-fifth of the tank length from the mouth of the straight inlet pipe. The invert of the outlet pipe should be placed at a level 5 to 7 cm below the invert level of inlet pipe.

Baffled inlet will distribute the flow more evenly along the width of the tank and similarly a baffled outlet pipe will serve better than a tee-pipe.

For larger capacities, a two-compartment tank constructed with the partition wall at a distance of about two-thirds the length from the inlet gives a better performance than a single compartment tank. The two compartments should be interconnected above the sludge storage level by means of pipes or square openings of diameter or side length respectively of not less than 75 mm. Every septic tank should be provided with ventilation pipes, the top being covered with a suitable mosquito proof wire mesh. The height of the pipe should extend at least 2 m above the top of the highest building within a radius of 20 m. Septic tanks may either be constructed in brick work, stone masonry or concrete cast in situ or pre-cast materials. Pre-cast household tank made of materials such as asbestos cement / HDPE could also be used, provided they are watertight and possess adequate strength in handling and installing and bear the static earth and superimposed loads.

All septic tanks shall be provided with watertight covers of adequate strength. Access manholes (minimum two numbers one on opposite ends in the longer direction) of adequate size shall also be provided for purposes of inspection and desludging of tanks.

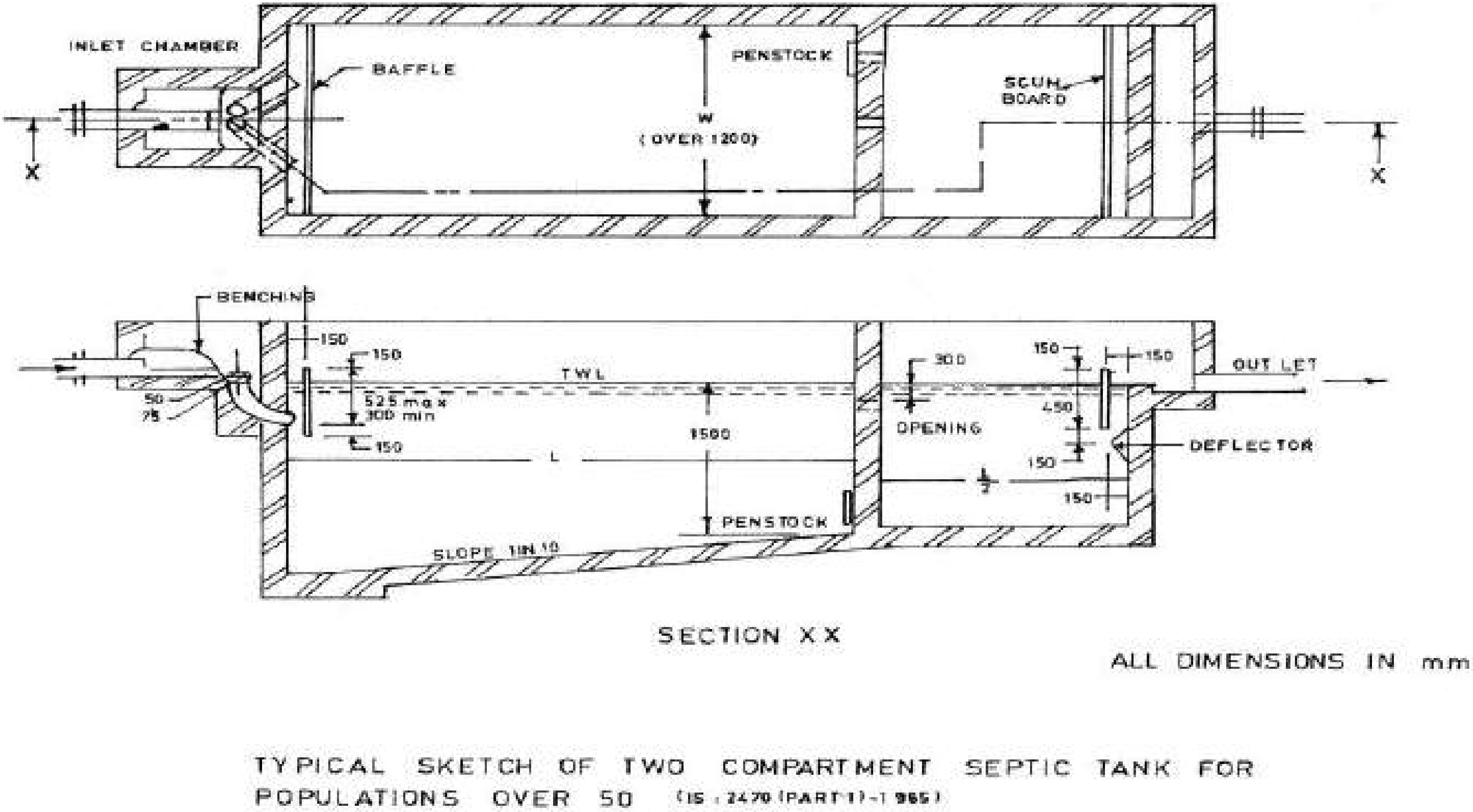
The floor of the tank should be of cement concrete and sloped towards the sludge outlet. Both the floor and side wall shall be plastered with cement mortar to render the surfaces smooth and to make them water tight. A typical two compartment septic tank is shown in Figure 9.8 (overleaf).

9.3.4.3 Sludge Withdrawal and Disposal

When sludge is drawn off from the bottom of the tank, at first the small quantity of sludge in the immediate vicinity of the outlet or suction pipe is withdrawn. This is followed by drawing off sewage, because the sludge, being only slightly heavier, but much more viscous than the sewage, lies away from the point of outlet and the scum remains floating on the surface. With continued draw-off more sewage is removed, until finally only sludge and scum remain in the tank. These come off last, and then only if there is sufficient slope on the floor of the tank, force them to gravitate to the outlet. This is the reason for the slow bleeding-off of sludge from steep bottomed pyramidal sedimentation tanks and for desludging by complete emptying. If septic tanks are desludged by only partial removal of the contents, then they become more and more full with sludge and scum, and the quality of the effluent deteriorates soon.

For certain reasons, desludging of septic tanks under hydrostatic head by means of a sludge pipe collecting of sludge from the lowest point in the tank and discharging at a higher level - should be discouraged. The manual handling of sludge should be avoided.

The mechanical vacuum tankers should be used by the municipal authorities to empty the septic tanks. Alternately, where space is not a constraint, a sludge-pipe with a delivery valve to draw the sludge as and when required, should be installed at the bottom of the tank to empty its contents into a sump, for subsequent disposal on land or sent for further treatment. Spreading of sludge on the ground in the vicinity should not be allowed. Portable pumps may also be used for desludging, in which case there will be no need for sludge pipe or sludge sump.



Source: CPHEEO, 1993

Figure 9.8 Structure of a septic tank

Yearly desludging of septic tank is desirable, but if it is not feasible or economical, then septic tanks should be cleaned at least once in two - three years, provided the tank is not overloaded due to use by more than the number of persons for which it is designed.

9.3.4.4 Secondary Treatment and Disposal of Effluent

The septic tank effluent will be malodorous, containing sizable portion of dissolved organic content and pathogenic organisms and hence, need to be treated before its final, safe disposal. Depending upon the situation — the size, treatment objective, resources available etc., the extent and type of secondary treatment facility can vary from the most conventional land disposal methods like soak pits or dispersion trenches to additional secondary biological treatment systems.

Normally, the land disposal methods are designed to achieve subsurface percolation or seepage into the soil. Satisfactory disposal therefore depends, to a great extent, on porosity and percolation characteristics of the soil.

In addition, other factors, such as level of subsoil water table, the climatic conditions, presence of vegetation, aeration of solid and concentration of suspended solids in the effluent also influence the application of these methods. Soak pits or dispersion trenches can be adopted in all porous soils, where soak percolation rate is below 25 minutes per cm and the depth of water table is 2 m or more from the ground level. Method of soil percolation test is described in Appendix A.9.2. Dispersion trenches should be preferred in soils with percolation rates between 12 and 25 minutes/cm, if adequate land is available. In areas with higher water table, dispersion trenches should be located partly or fully above ground level, in a mound.

The subsoil dispersion system shall be at least 20 m away from any source of drinking water. It should also be as far as possible from the nearest dwellings, but not closer than 7 m to avoid any corrosive effect due to tank gases vented into atmosphere. Subsoil dispersion system is not recommended in limestone or crevice rock formations, where there may be solution cavities that may convey the pollution to long distances and pollute water resources. In impervious soils such as dense clays and rocks, where percolation rate exceeds 25 minutes/cm, adoption of up flow or reverse filters, trickling filters, subsurface sand filters or open sand filters followed by chlorination should be considered, particularly for larger installations.

In the absence of information relating to ground water or subsoil, subsurface explorations are necessary. Percolation tests determine the acceptability of the site and serve as the basis of design for liquid absorption. The total subsurface soil area required for soak pits or dispersion trenches is given by the empirical relation:

$$Q = 130\sqrt{t} \tag{9.1}$$

where

Q = Maximum rate of effluent application in $l/d/m^2$ of leaching surface, and

t = Standard percolation rate for the soil in minutes.

In calculating the effective leaching area required, the area of trench bottom in case of dispersion trenches and effective side wall area below the inlet level for soak pits should be considered.

9.3.4.5 Soak Pits

Soak pits are cheap to construct and are extensively used. They need no media when lined or filled with rubble or brick bats. The pits may be of any regular shape, circular or square being more common. When water table is sufficiently below ground level, soak pits should be preferred only when land is limited or when a porous layer underlies an impervious layer at the top, which permits easier vertical downward flow than horizontal spread out as in the case of dispersion trenches.

Minimum horizontal dimension of soak pit should be 1 m, the depth below the invert level or inlet pipe being at 1 m. The pit should be covered and the top raised above the adjacent ground to prevent damage by flooding. It is being recommended that these are to be phased out in due course of time.

9.3.4.6 Dispersion Trenches

Dispersion trenches consist of relatively narrow and shallow trenches about 0.5 to 1 m deep and 0.3 to 1 m wide excavated to a slight gradient of about 0.25%. Open joined earthenware or concrete pipes of 80 to 100 mm size are laid in the trenches over a bed of 15 cm to 25 cm of washed gravel or crushed stone. The top of pipes shall be covered by coarse gravel and crushed stone to a minimum depth of 15 cm and the balance depth of trench filled with excavated earth and finished with a mound above the ground level to prevent direct flooding of trench during rains. The effluent from the septic tank is led into a small distribution box from which several such trenches could radiate out. The total length of trench required shall be calculated from the Eq. (9.1) and the number of trenches worked out on the basis of a maximum length of 30 m for each trench and spaced not closer than 2 m apart. Parallel distribution should be such that a distribution box should be provided for 3 to 4 trenches. It is being recommended that these are to be phased out in due course of time.

9.3.5 Improved Septic Tank

9.3.5.1 Up-Flow Anaerobic Filter

The up-flow filter can be successfully used for secondary treatment of septic tank effluent in areas where dense soil conditions, high water table and limited availability of land preclude soil absorption or the leaching system for effluent disposal. It is a submerged filter with stone media or half broken chamber well burnt bricks by hand and the septic tank effluent is introduced from the bottom.

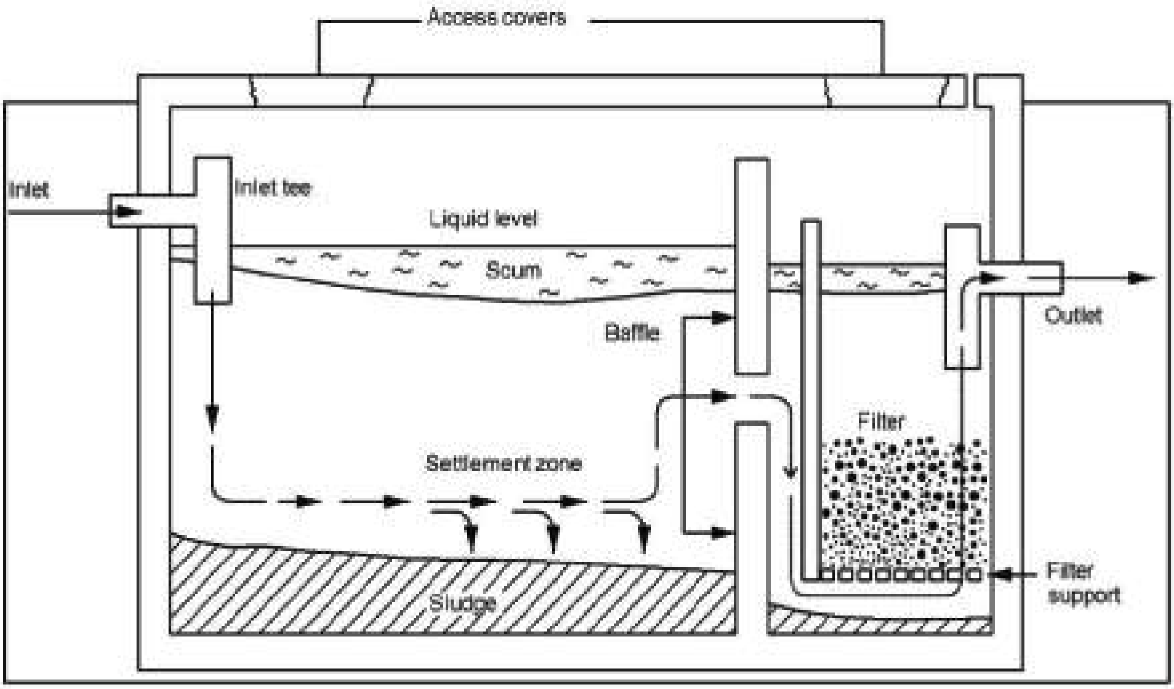
The microbial growth is retained on the stone media, making possible higher loading rates and efficient digestion. The capacity of the unit is 0.04 to 0.05 m3 per capita or 1/3 to 1/2 the liquid capacity of the septic tank it serves. BOD removals of 70% can be expected. The effluent is clear and free from odour. This unit has several advantages viz, (a) high degree of stabilization; (b) little sludge production; (c) low capital and operating cost; and (d) low loss of head in the filter (10 to 15 cm) in normal operation.

The up-flow anaerobic filter can either be a separate unit or constructed as an extended part of septic tanks. An anaerobic filter is a fixed-bed biological reactor.

Dissolved organic matter and non-settleable solids are filtered and anaerobically digested by the bacteria in the biofilm attached to the filter media.

Anaerobic filters are widely used as secondary treatment in household black or grey water systems and to improve the solid removal compared to septic tanks or anaerobic baffled reactors. Since anaerobic filters work by anaerobic digestion, they can be designed as anaerobic digesters allowing recovering the produced biogas.

Multi-chamber septic tank system prevents sludge carryover. The schematic diagrams of anaerobic filter, anaerobic baffled reactor, and multi chamber anaerobic filter is provided in Figure 9.9, Figure 9.10 and Figure 9.11 respectively.



Simple one unit anaerobic Filter integrated in the second chamber of a septic tank.
Gas is evacuated by the venting opening at the upper right.

Source: Tilley et al., 2008

Figure 9.9 Anaerobic filter

A typical septic tank up flow filter for 10 persons is shown in Figure 9.12. A typical septic tank - up flow filter evapotranspiration system is shown in Figure 9.13.

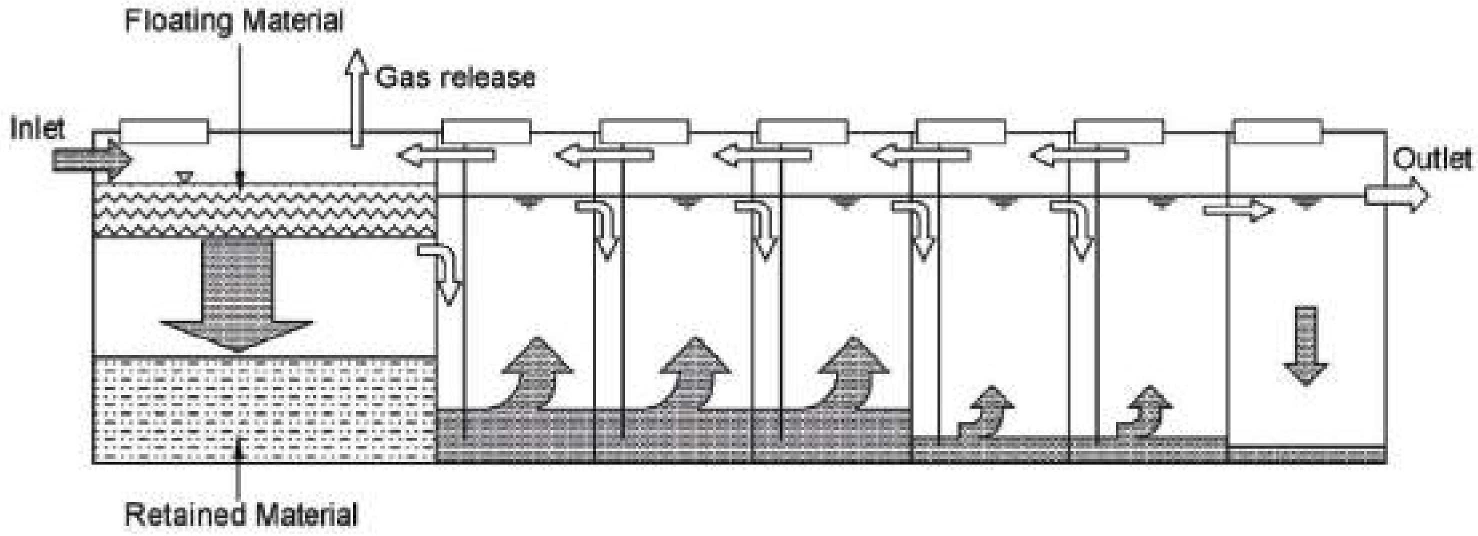


Figure 9.10 Anaerobic baffled reactor

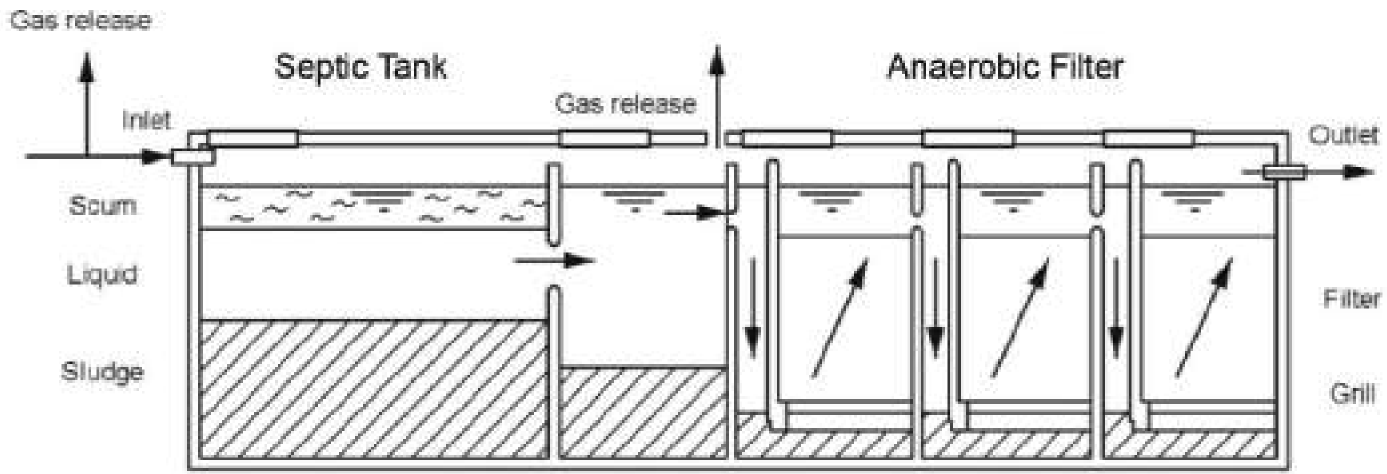


Figure 9.11 Multi chamber anaerobic filter

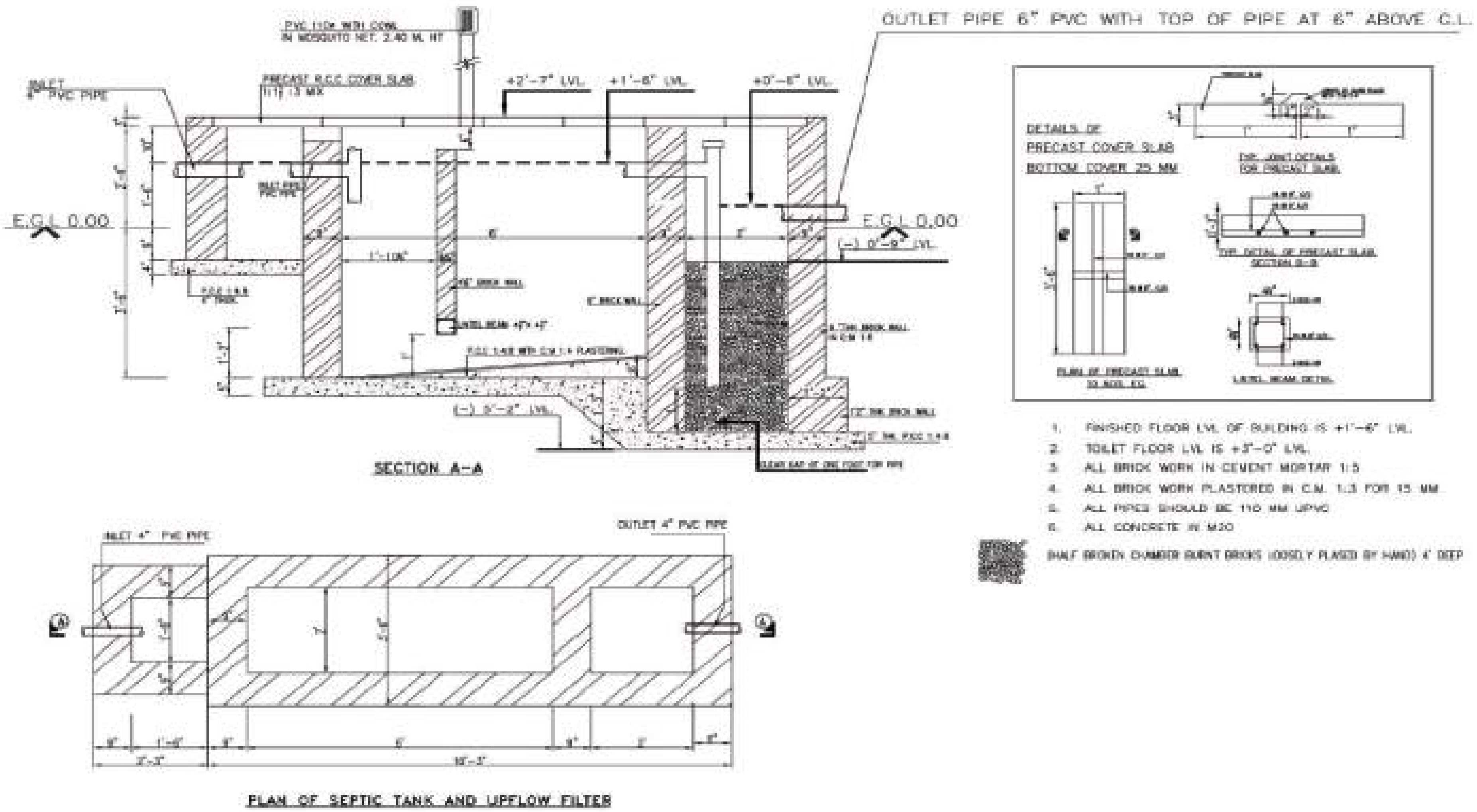
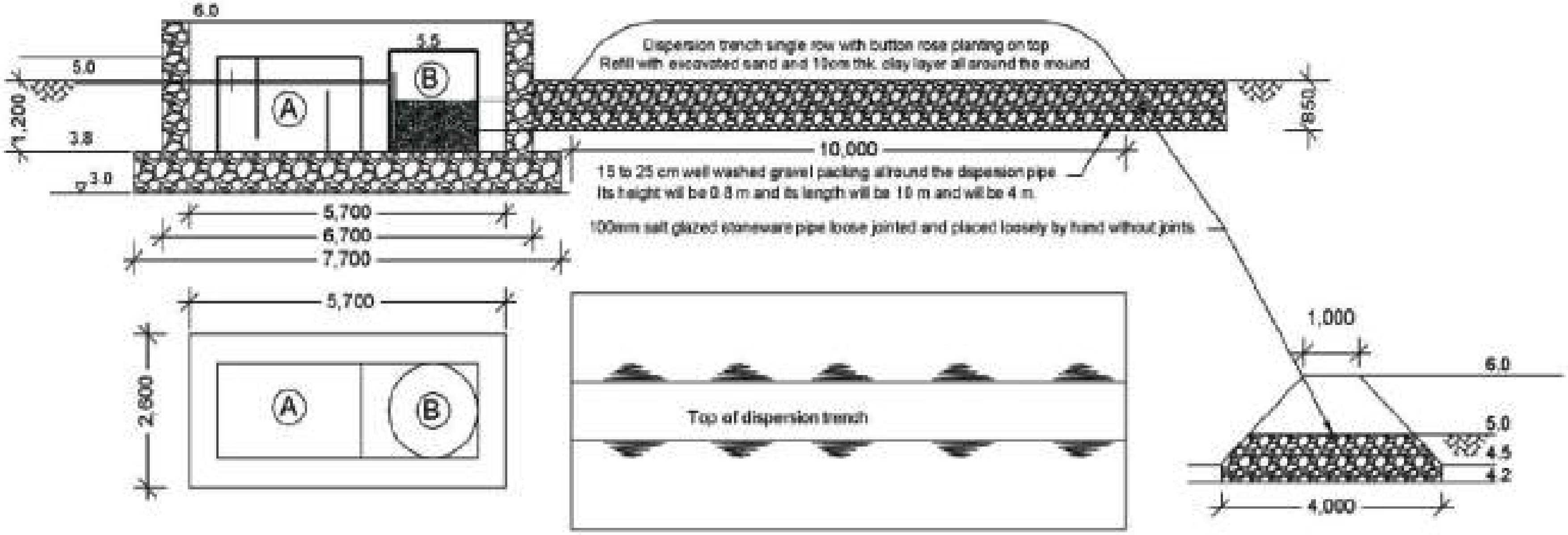


Figure 9.12 Typical septic tank up flow filter for ten persons



Location	Elevation	Location	Elevation
G.L.	5 m	Level of top of upflow filter media	4.7 m
Factory floor	5.3 m	Level of dispersion trench sewer	4.7 m
Seat of European water closet	5.7 m	Bottom level of dispersion trench	4.2 m
Level of sewer outlet	5.2 m	Top of gravel layer in dispersion trench	5.0 m
Level of sewer entering septic tank	5.0 m	Top of dispersion trench	6.0 m
Level of sewer leaving septic tank	5.0 m		

(A) Polyethlene septic tank model 5 of M/S. United sanitation devices, United building, Amala nagar, P.O., Thrissur Dt., Kerala, 680 555. Ph. 0847 - 2307791, 2308792, E-mail unitedseptictanks@gmail.com, enquiry@unitedseptictanks.com

(B) Chemical / acid storage HDPE tank model no. CCV-250-01 of M/S. Sintex of diameter 1506mm & height 1701mm with manhole diameter of 450mm from Sintex Industries Ltd, Plastic Division, 166, Anna Salai, Little Mount, Chennai-600015, Ph.: 22200302, 22200405, Fax: 044-22353225, E-mail: sintex06@dataone.in

Figure 9 .13 Septic tank - Up flow filter evapotranspiration system

The arrangement shown in Figure 9.13 is septic tank followed by up flow filter. The effluent of the up flow filter normally will discharge into nearby drains and will find its way to a public water course. However in some cases even the public drain may not be available and it becomes a challenge to dispose off the up flow filter effluent. In such a case the effluent will stagnate and will lead to propagation of flies, mosquitoes etc. leading to environmental problems. This can be avoided by a raising the elevation of the pan in the toilet so that effluent comes out of the septic tank at higher than the ground level. Further this effluent will go through the up flow filter before it finally comes out as treated effluent and this will involve additional drop of the sewage level. All these have to be considered so that the final effluent from the up flow filter will come out at least 30 to 45 cm above the ground level. At this location an elevated mound of sand can be constructed as a dispersion mound and flowering small plants can be grown for evapotranspiration. This system in Figure 9.13 is one such and meant for a factory with 25 persons working for whom the septic tank volume is 4.7 cum and is met by the above specified septic tank and the volume of up flow filter at one third volume of septic tank is also met with comfortably. The dispersion trench requirement is 13 sqm. The area provided is 28 x 0.8 = 22 sqm. The maximum uplift pressure can be as high as 5 – 3 = 2 m. This is countered by stone masonry floor of 0.8 m thickness which equates to 0.8 x 2.5 = 2 m of water column. On the same lines the top of the stone masonry side walls are increased to 6 m and thus the system is safe. The inter-space between the side wall and the filter and the septic tank will be filled with excavated sand and plastered in a chamber so that rain water flows away and does not get into the structure. The stone masonry itself will be random rubble using boulders available at site with base slab 0.8 m thick and sidewalls 0.5 m thick set in cement mortar 1.5 with only pointing. Later on, when the full-fledged sewerage system becomes functional, this on-site system can be dismantled and the entire stone masonry, septic tank, up flow filter are all reusable in other construction sites to advantage, The dispersion trench functions mainly by evapotranspiration due to the button rose plantation whose roots act like a pump is the capillary action. During times of rainfall, it will be necessary, to provide a temporary cover to prevent direct rainfall over the dispersion trench by simple arrangements like a tarpaulin sheet placed around it and stone boulders kept on the edges at GL.

9.3.6 Package Septic Tank – Anaerobic Filter Type System

The disadvantage of the septic tank is its low treatment efficiency (30-60% BOD and SS removal) and associated cost and space requirements for the construction of soak pit. Many situations such as presence of rocky ground, highly permeable soil and high groundwater table do not allow the construction of soak pits. In such cases, it is often a common practice to discharge effluent directly into an open drain causing surface water pollution. Another disadvantage of septic tank is its incapability to handle hydraulic shock loads, as peak flow disturbs the settling zone and causes high suspended solids in the effluent. One of the recommended solutions is the provision of anaerobic filter type system for the treatment of septic tank effluent (MoUD, 2008). Hence, package type septic tank- anaerobic filter system can be used to enhance the removals

Typically, this type of package on-site treatment system is made up of LLDPE (Linear Low Density Polyethylene) and can be installed easily in a very short time. It consists of two chambers, i.e., settling and anaerobic filter. The first chamber works as a septic tank, where settleable solids are settled down and further degraded anaerobically at the bottom zone.

The second chamber consists of up flow anaerobic filter, where further removal of organic matter takes place and made up of synthetic media with specific surface area of as high as 100 m^2/m^3. This provides additional surfaces for the growth of organisms that purify the sewage further. There are a couple of manufacturers in the country as also many others elsewhere, but published and documented performing data are not available. All the same, the relative performance as compared to mere septic tank alone is expected to be better. Precautions to be taken are the use of media from virgin material, their specific gravity being close to water and the percent volume of packing within the reactor so that the microbes do not overgrow, bridge up and eventually choke the entire filter. However, it should be noted that this effluent would still contain pathogens and nutrients that are capable of causing public health and environmental problems and there remains the ambiguity about the technology, its feasibility and technical robustness. Such systems can be easily modified and applied to India, where localized on-site treatment systems are most desirable. Lab scale testing has been carried out at IIT Roorkee and the test facility dimensions are shown in Figure 9-14.

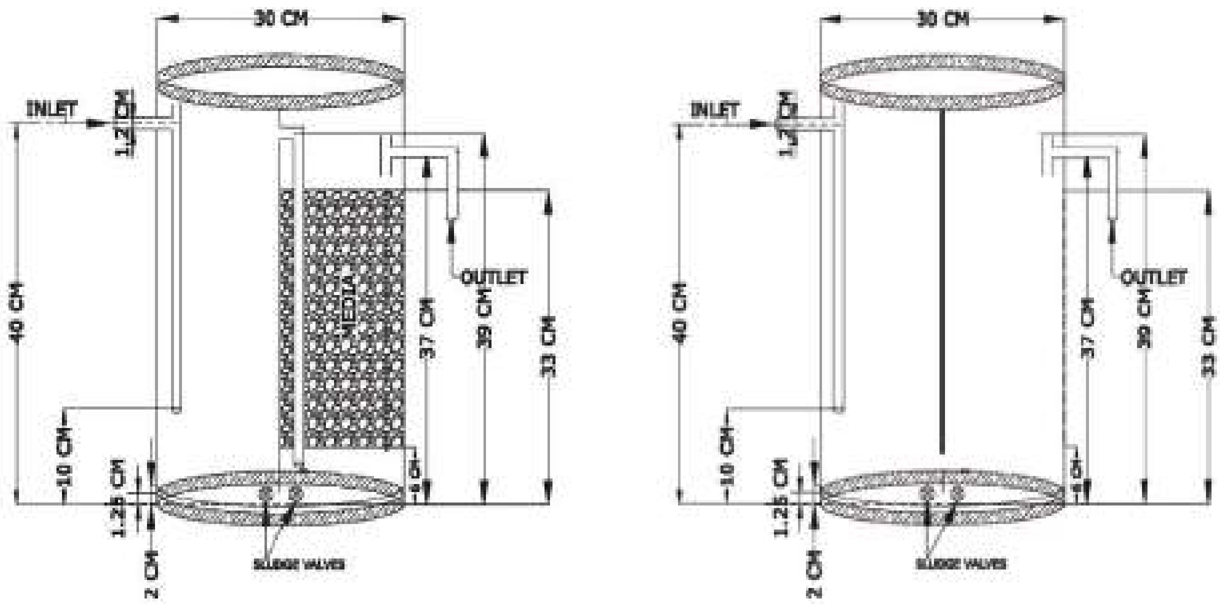


Figure 9.14 Lab Scale Facility Used for Testing the flow pattern and COD removal

The COD was in the range of 472 to 600 in raw sewage, 111 to 154 mg/l in septic tank effluent and 57 to 60 mg/l in the anaerobic filter effluent. Further studies are being pursued for pilot scale testing, followed by actual field units in a school campus and in a household and evaluate the parameters of hydraulic and organic loadings, performance results of removal of BOD, COD and coliforms and engineering modifications to bring out a design and O&M manual.

9.3.7 On-site Package Septic Tank - Contact Aeration Type System

Another improvement of the septic tank is to provide contact aeration tank after the septic tank. Hence, in package type septic tank - contact aeration system is developed in the line of well-established Japanese on-site treatment systems called Johkasou.

This type of package on-site treatment system is made up of LLDPE (Low Linear Density Polyethylene) and can be installed easily in a very short time. It consists of two chambers, i.e., settling and contact aeration with pall ring media.

The first chamber works as a septic tank, where settleable solids are settled down and further degraded anaerobically at the bottom zone. Second stage is high specific surface area (100 m^2/m^3), fixed film plastic media to retain high mass of aerobic microorganism to degrade the organic matter in the sewage aided by continuous diffusion of controlled air supply from a blower. The high specific surface area not only prevents clogging, but also provides intensive contact between the sewage and the fixed film aerobic bacteria for the fast degradation of organic matter. The treatment performance may be possible to be enhanced to 80-95% for BOD and SS removal. A possible section is illustrated in Figure 9.15.

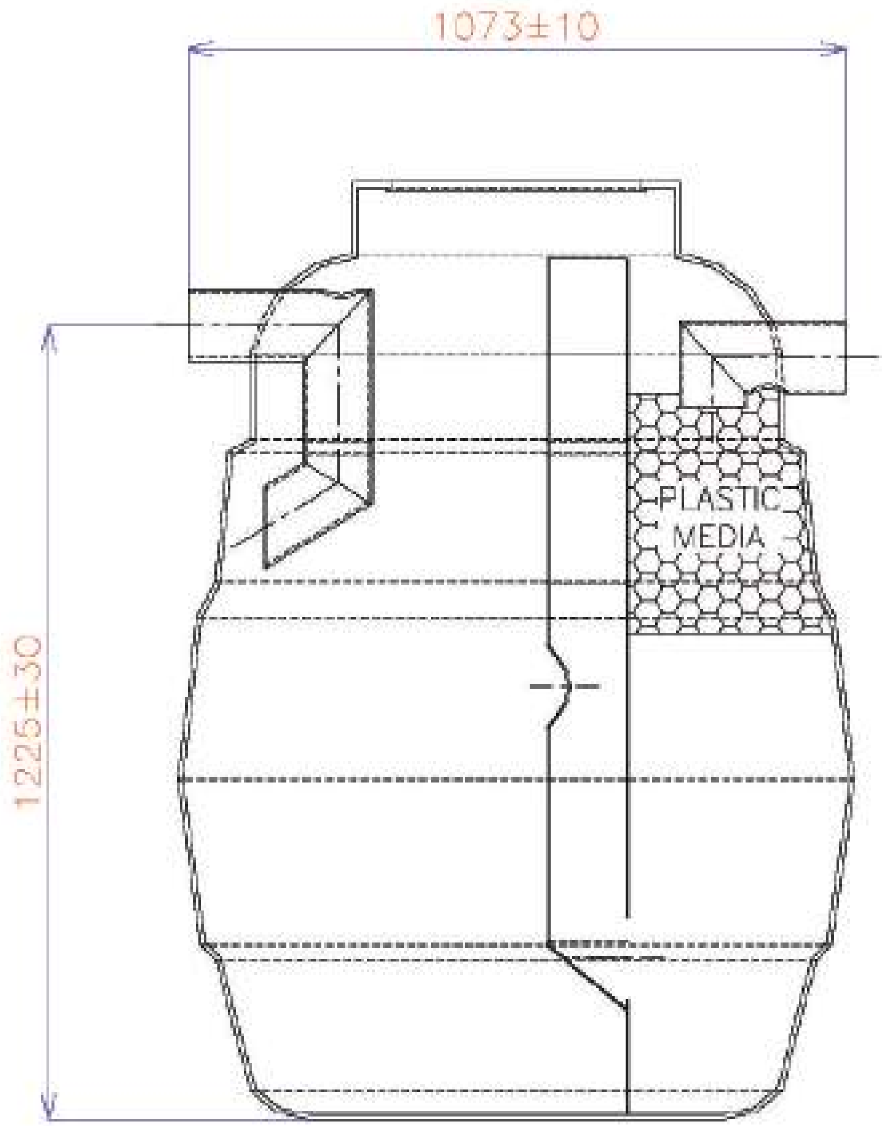


Figure 9.15 Illustrative configuration of an integral septic tank and contact aeration unit

9.3.8 Advanced Anaerobic - Aerobic Type On-site Treatment System (Johkasou)

There are various kinds of packaged treatment technologies. This subsection describes package type treatment plant, taking Japanese Johkasou as an example, and on-site construction type treatment plant.

9.3.8.1 Classification of Treatment Systems

Treatment systems are classified into various types according to capacity and performance.

i. Capacity

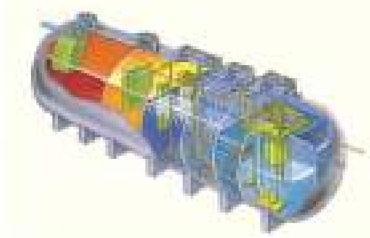
Treatment systems are classified into three types according to capacity: a small-scale unit is for several to more than a dozen people, who live in individual houses, a medium-scale system is for up to hundreds of people, who live in a condominium or small village and a large-scale system is for thousands of people in a large commercial building or factory.

Package-type is applied from small to large-scale systems. When unit is applied to large-scale, multiple tanks are connected. Package-type is made from plastics such as GFRP (Glass fibre reinforced plastics) or steel plates (that depends on the treatment method), so they can be manufactured in a factory.

The on-site construction type is made from RC and constructed on-site, so it looks nearly like a small-scale sewage treatment plant.

The classification according to the treatment capacity is mentioned in Table 9.9

Table 9.9 Classification according to treatment capacity (Example of Japan)

Package-type			On-site construction-type
Small-scale	Medium-scale	Large-scale	Medium/Large-scale
(About 5 to 50 people)	(About 51 to 500 people)	(Approx. 500 to 5,000 people)	(More than 500 people)
			

ii. Performance

Treatment processes are classified into three kinds according to performance:

1. Process that mainly removes BOD-related contaminants,
2. Process that removes BOD-related contaminants and nitrogen, and
3. Process that removes BOD-related contaminants, nitrogen, and phosphorus.

In addition, advanced treatment for better effluent quality is possible by applying membrane separator or flocculation separation or activated carbon adsorption, etc. Some package-types contain membrane separator unit in it. The classification according to the treatment performance is mentioned in Table 9.10.

Table 9.10 Classification according to treatment performance (Example of Japan)

Type	Treatment Method	Treated water quality, mg/L		
		BOD	T-N	T-P
Package type Small scale Medium scale Large-scale	BOD reduction	≤20	–	–
	Nitrogen removal	≤20	≤20	–
	Nitrogen and phosphorus removal	≤20	≤20	≤1
	Membrane separation	≤5	–	–
	Nitrogen and phosphorus removal	≤5	≤10	≤1
On-site construction Type Medium scale Large scale	Contact aeration	≤20	–	–
	Activated sludge			
	Flocculation separation (A)	≤10	–	≤1
	(A) and activated carbon absorption	≤10	–	≤1

9.3.8.2 System Configuration

A treatment system consists mainly of pre-treatment, main treatment, advanced treatment (if necessary), and disinfection processes.

i. Pre-treatment process

This process removes insoluble substances that are difficult to decompose biologically by means of sedimentation, floating, and screening. In the large-scale system, a flow equalizer is planned for stabilizing the biological treatment.

ii. Main treatment process

The main treatment process biologically removes BOD-related contaminants by aerobic treatment and removes nitrogen by combination of anoxic and aerobic treatment. The system employs a sedimentation tank for solid-liquid separation in most cases, but use of a membrane separator in place of the sedimentation tank makes it possible to downsize the system and to improve the quality of treated sewage further.

iii. Advanced treatment process (to be installed if necessary)

This process removes COD-related contaminants and phosphorus from the biologically treated sewage by means of flocculation sedimentation, sand filtration, activated carbon absorption, and dephosphorization.

iv. Disinfection process

This process disinfects E. coli and other bacteria to make effluent water safer.

9.3.8.3 Example Design in Japan

- Treatment flowchart and system configuration

Figure 9.16 shows the flowchart and configuration of a package-type treatment system based on the “anaerobic filter and contact aeration method (for BOD reduction)” as an example. This system consists of anaerobic filter, contact aeration, sedimentation, and disinfection tanks.

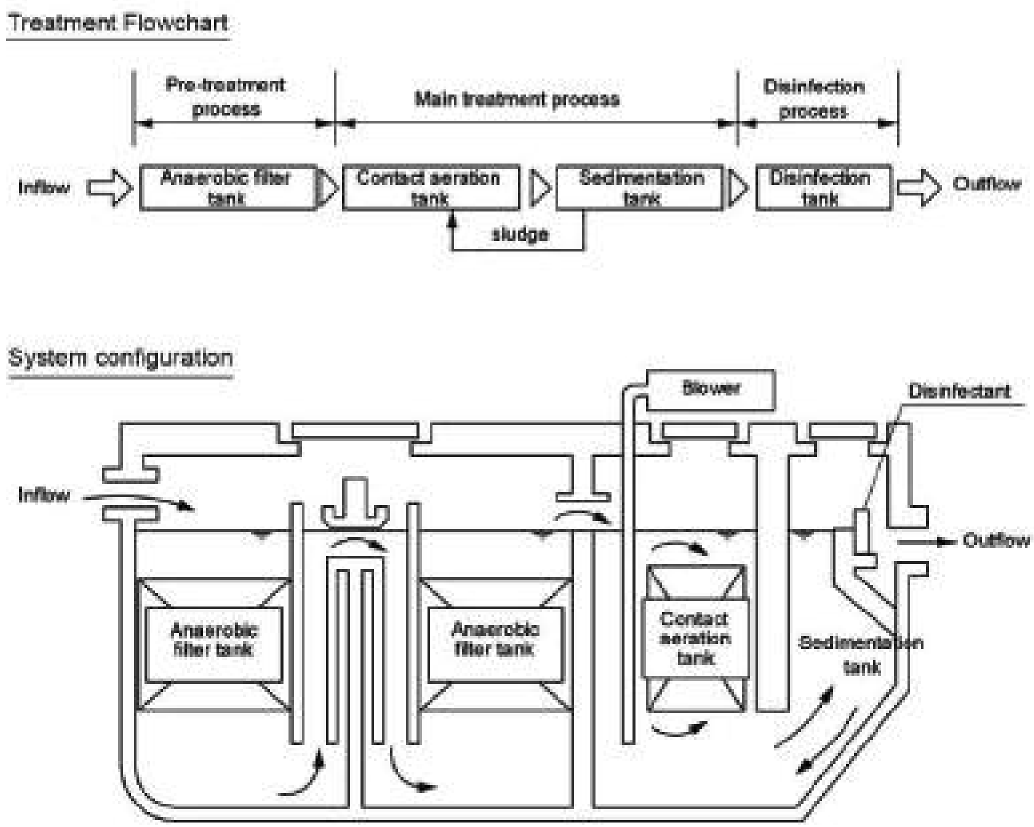


Figure 9.16 Treatment system based on the anaerobic filter and contact aeration method

- Outline of the system components

- Anaerobic filter tank

The main purpose of this tank is to remove solid matter that cannot be removed by biological treatment. In addition, anaerobic microorganisms adhering to the surface of the filter media submerged in this tank decompose part of BOD-related contaminants.

- Contact aeration tank

In this tank, the aerobic microorganisms are activated by the air supplied by blower and biodegradation takes place. That is, BOD-related contaminants are consumed and decomposed by the microorganisms. The contact media is installed in this tank and microorganisms are adhered on it to improve contact efficiency. Introduction of moving bed bioreactor (MBBR) contributes to reduce size of the package-type.

- Sedimentation tank

The purpose of this tank is solid-liquid separation. Supernatant and sludge contained in biologically treated sewage are separated by gravity sedimentation. Supernatant is transferred to subsequent process and the settled and separated sludge returns to the previous tank, resulting in a gradual rise in the sludge concentration of the aeration tank.

- Disinfection unit

This process disinfects E. coli and other bacteria contained in the supernatant from sedimentation tank to make effluent water safer. As the disinfectant, solid chlorine is used.

- Example specifications

Table 9.11 (overleaf) shows a package-type treatment system for 10 persons.

ii. On-site construction-type

- Treatment flowchart and system configuration

As an example of on-site construction-type treatment systems based on “the contact aeration method and the flocculation sedimentation method,” Figure 9.17 (overleaf) shows the flowchart and configuration. This system consists of a screen, a flow equalization tank, a contact aeration tank, a flocculation sedimentation tank, a disinfection unit, and a sludge treatment unit.

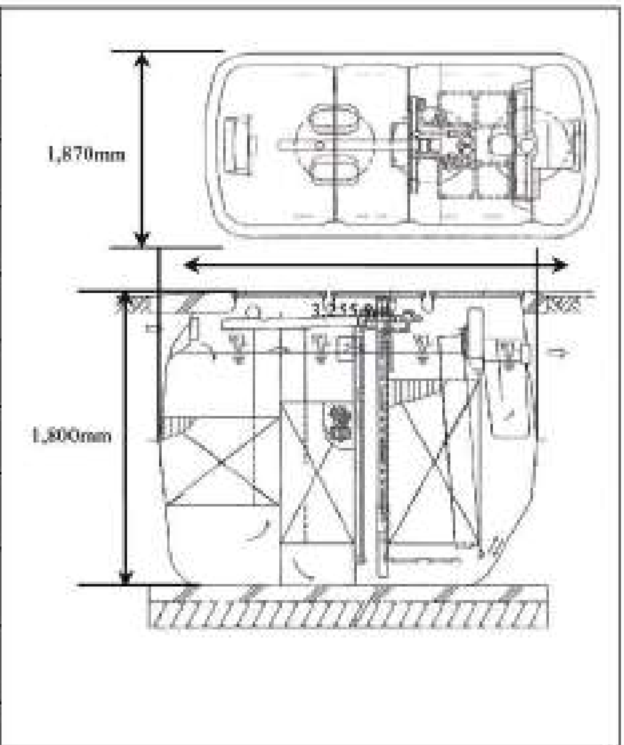
- Outline of the system components

- Screen

The purpose of this screen is to remove foreign matter. The screen is classified into three types according to mesh size: the coarse, fine and micro screens. A combination of them is planned according to the characteristics of sewage.

Table 9.11 Example specifications for a package-type treatment system in Japan

Capacity (A)	
10 Persons (2.0 m^3/day*)	
Weight (equipment only)	
470 kg	
Main body material	
FRP	
Tank volume, Equipment capacity	
Anaerobic filter tank	No. 1: 2.13 cum No. 2: 1.414 cum
Contact aeration tank	2.037 cum
Sedimentation tank	0.717 cum
Blower	120 L/min × 130 W



(A):The daily amount of sewage per person is 200 L.

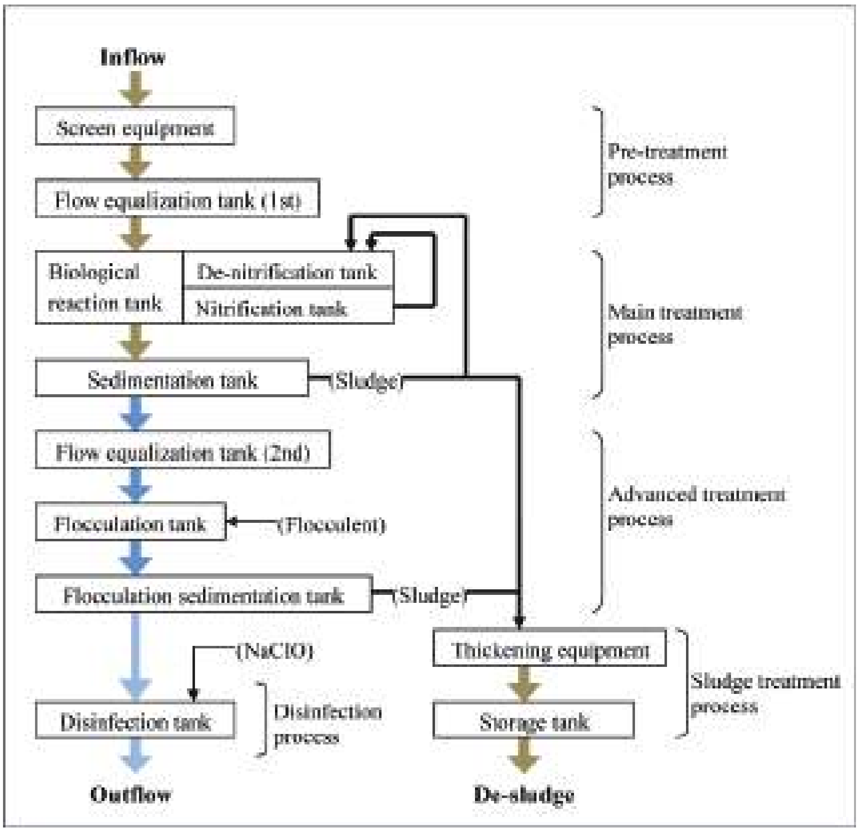


Figure 9.17 Flowchart of the contact aeration method and flocculation sedimentation method

- Flow equalization tank

  In the on-site small-scale treatment system, load changes due to rise or reduction in the flow rate of sewage and have a direct impact on the biological treatment function. This tank is installed before the biological reaction tank to have a stable load on it. The capacity of the flow equalization tank shall be specified according to changes in the flow rate of sewage.

- Contact aeration tank

  This unit consists of contact aeration and sedimentation tanks. The former is filled with a contact media to form and put biological film on the media surface and to biologically treat the sewage by letting it come into contact with the film under aerobic conditions. The contact aeration tank is equipped with an aerator that maintains the aerobic environment, and a back washing machine that removes biota generated excessively from the contact media.

- Flocculation sedimentation tank

  This tank removes COD-related contaminants and phosphorus by adding a flocculent to the sewage. The agent is classified into two types: one is an aluminium coagulant (e.g. aluminium sulphate) and the other is a ferric flocculent (e.g. polyferric sulphate). The unit consists of flow equalization, flocculation and flocculation sedimentation tanks.

- Disinfection unit

  This unit disinfects the treated effluent. Solid or liquid (sodium hypochlorite) chlorine is used as the disinfectant.

- Sludge treatment unit

  This unit receives and stores sludge generated in the biological reaction and flocculation sedimentation tanks. In certain circumstances, a sludge thickening or dehydrating unit may be planned. The stored sludge shall be regularly extracted and delivered to the outside. An example of designing an on-site construction-type sewage treatment system is shown in Appendix A.9.3.

9.3.8.4 Features

i. Advantages

- Since package-type treatment equipment can be fully manufactured in a factory, quality control of the product is easy and the price can be reduced due to a mass production effect.

- A treatment system for home use (5 to 10 persons) requires an area of 3 to 5 m^2; that is to say, it is a compact system. Moreover, when it is installed underground, the space above can be used for several purposes such as a garage.

- This system, being a product manufactured in a factory, does not require complicated work on site during installation, so the installation time is short (about one week). Accordingly, it can improve environmental sanitation quickly.

- The treatment system requires running costs, such as electric charges and chemical expenses, and the treated water is comparable to that of conventional treatment system. In addition, planning advanced treatment can result cleaner effluent and remove nitrogen and phosphorus.
- Where membrane separator is applied, BOD contained in the effluent is reduced to less than 5 mg/l, and the treated effluent can be reused for various purposes.
- The treatment system can be constructed more cost-effectively and faster, because sewer is shorter compared with conventional system, especially in areas with low-medium population density, areas that have not been covered by sewer and individual houses or buildings.

ii. Notes on application

Keeping the performance of a treatment system high requires proper maintenance, which varies depending on the scale and treatment method of the system. Common works to achieve this are listed below. Each work requires expertise, so it is necessary to build up an implementation system, to train inspectors (vendors), and to educate users to increase their awareness of the importance of maintenance. For more information about the maintenance of treatment systems, see Part B Operation and Maintenance.

- Maintenance and inspection

  Inspecting mechanical components including the blower, replenishing tanks with chemicals including disinfectants, etc.

- Water quality check

  Checking the aeration tank for DO testing the quality of the discharged water, etc.

- Cleaning

  Removing foreign matter from the screen and extracting generated sludge.

9.4 DECISION MAKING FOR ON-SITE TREATMENT TECHNOLOGY

9.4.1 General

Employing on-site sewage treatment technology requires an in-depth survey of requirements of the installation site, such as the volume and quality of treated sewage, the selection of a method based on the resulting data and the determination of the scale. After the determination of the basically required performance and scale, the treatment method shall be chosen in consideration of the following requirements:

i) The method shall be as simple as possible.

ii) The maintenance shall be easy.

iii) The construction and maintenance costs shall be low.

iv) The method shall contribute to environmental preservation and water quality improvement.

v) The quality characteristics of incoming sewage shall be understood.

vi) Changes in the quality and rate of incoming sewage shall be taken into consideration.

Any sewage treatment system is required to be made available always and display its function and performance, but the installer may have little knowledge about it. The selection of an appropriate treatment method requires consideration of preserving the water quality environment in receiving water bodies and fund necessary for construction and maintenance.

9.4.2 Problems with Existing On-site System

9.4.2.1 Natural Characteristics

A natural condition requiring caution is weather, such as temperature changes and precipitation. In addition, it is necessary to investigate geographical features and groundwater levels.

The effect of temperature on the sewage treatment function varies depending on the type of treatment facilities, the degree of load, and the kinds of contaminants to be removed. A combination of nitrification and denitrification is susceptible to temperature changes; the reaction rate at 23ºC is 2 to 2.5 times that at 13ºC, the higher the water temperature, the higher the reaction rate. Accordingly, if this technology is applied to cold areas, it is effective to set up BOD and nitrogen loads lower than the design values.

Precipitation is one of the local characteristics. Mixing a large amount of rainwater with sewage reduces the treatment function. Particularly in a housing estate where sewage is collected and treated, it is necessary to employ an advanced construction technology in consideration of the effect of the amount of rainwater.

9.4.2.2 Social Requirements

If the treated sewage receiving water body is a source of drinking water, it is essential to employ a treatment method that can remove nitrogen and phosphorus and faecal coliforms to preserve the water source. In addition, the features of site where a sewage treatment tank is installed shall be taken into consideration. For example, a region with low population density can provide a relatively extensive site, which makes it possible to employ a treatment method featuring easy maintenance, while an urban area is obliged to use a compact treatment method. In addition, the latter case has the risk of troubling the neighbouring people with noise and offensive odour generated by the sewage treatment system.

Accordingly, it is essential to select a treatment method that does not cause such problems or to take measures to mitigate them.

The extent of maintaining a sewage treatment system has an effect on the treatment function and performance. Therefore, a small-scale system shall employ a method featuring as easy maintenance as possible. In addition, it is necessary to select an installation site in consideration of the smooth extraction, transportation, and treatment of sludge.

9.4.3 Elements of Successful Programme

Planning an on-site system and making it successful require the following decision-making processes (Stages 1 to 5).

Stage 1: Outline survey of settlements and services

The objective of the first stage is to gather information about the coverage and quality of existing services to clarify the key problems to be addressed and priority locations for improvement.

This investigation might be done citywide or within areas of the town that have already been earmarked for attention. The information can be obtained from (a) maps and other secondary sources; (b) from a rapid physical inspection on the ground; and (c) from informal discussion with residents.

This preparatory work does not involve systematic user consultation, which follows in Stage 2.The output includes one or more maps that show the existing sanitation infrastructure and services, and highlights areas where sanitation problems are most acute.

Stage 2: The Need for assessment and consultation

Stage 2 entails a more detailed analysis of the current situation to reveal what types of improvements are needed and where they will have the most beneficial impact. It involves further technical investigations in priority areas identified from Stage 1, plus an assessment of existing services from the users' point of view.

This should provide a fuller understanding of why existing services have failed or are otherwise inadequate. This is also an opportunity to find out what type of improvements users want and would be willing to pay for, or at least contribute towards.

Stage 3: Identification of appropriate technologies

The objective of this stage is to eliminate technologies that are unlikely to be viable from a technical perspective and thus, narrow the field of options. The key question for each option at this stage is: 'Can it work?' A variety of additional factors (some of them financial and managerial) affect whether an option would in fact be viable and these are considered in Stage 4.

Stage 4: Development of cost options

Stage 3 identified technology options that are viable from a technical perspective. In order that technology choices can be made, this stage estimates the capital and operating costs associated with each option over its anticipated lifetime, and considers how the new services could be operated and maintained. This should confirm whether the technologies are viable in terms of the human and financial resources available locally.

For those that are viable, cost packages can be presented to the community in Stage 5 and agreement reached on the final choice.

Stage 5: Reaching consensus on preferred options

In the final stage, the options developed in Stage 4 can be presented back to the community. For each package, the technical, managerial and financial implications - including proposed operation and maintenance arrangements - need to be explained clearly. This should enable residents to engage in an informed discussion with municipal representatives resulting, hopefully, in consensus on the way forward.

The following shows preconditions important for a successful on-site system:

- Establish and enforce clear and effective policy frameworks
    - Update and enforce septic tank design codes
    - Mandate scheduled desludging
- Strengthen institutional and implementation capacity
    - Develop comprehensive awareness programmes, especially targeting septic tank users
    - Develop mechanisms for inter-agency coordination and dialogue
    - Develop comprehensive capacity building programmes that engage educational institutions
    - Apply economies of scale in deploying septage services
    - Leverage real estate development to build sewerage infrastructure
    - Engage private service providers
- Increase funding for septage management
    - Strengthen National financial support for septage management
    - Promote creative financing
    - Design innovative sewerage tariff structures
    - Develop progressive fee structures in line with willingness to pay
    - Create opportunities and incentives for commercial activities

9.4.4 Transport and Fate of Sludge

Sludge resulting from on-site treatment shall be treated and disposed of in consideration of its impact on the surrounding environment. The following shows precautions for this work:

- Sludge shall not pollute the environment
- Sludge shall not produce any diseases and pests
- Sludge shall not be disposed off illegally
- Sludge shall be reused as effectively as possible.

In any case, it is recommended to effectively change the septage and sludge to compost for agricultural use or to soil conditioners in its final disposition. The septage shall be disposed as per the advisory note issued by the MoUD, http://urbanindia.nic.in/programme/uwss/Advisory_SMUI.pdf

9.5 DEALING WITH SEPTAGE

The effluent from the septic tank can be collected in a network of drains and/or sewers and treated in a treatment plant designed appropriately on the lines discussed in Chapter 5. The accumulating sludge at the bottom of the septic tank however, has to be also removed and treated once it has reached the designed depth or at the end of the designed desludging period whichever occurs earlier. Such a removal is possible only by trucks. While sucking out the sludge, the liquid in the septic tank will also be sucked out. Such a mixture is referred to as septage. Obviously, the removal of septage from a household septic tank will occur approximately once in two or three years only.

9.5.1 Characterisation of Septage

9.5.1.1 Septic Tanks used only for Water Closets

In general, the septic tank is intended to be used only for the water closet and hence, the night soil alone is the causative factor for the organic load. Thus, as far as the BOD is concerned, the per capita contribution of night soil and the volume of ablution water and its frequency per day are relevant. The urine is the factor for the nitrogen content. The septic tank system reported in the twin drain system has recorded a range of characteristics of BOD, COD and SS as in Table 9.12 and in Table 8.1.

Table 9.12 Range values of BOD, COD and SS at inlet to septic tank in India

No.	Indicator	BOD, mg/L	COD, mg/L	SS, mg/L
1	Mean	1,290	2,570	4,140
2	Standard Deviation	143	290	542
3	Range	970 to 1,550	1,920 to 3,050	2,550 to 4,860

The average amount of ablution water used at this location was about 6 litres per use. The BOD from defecation is about 8 grams/day. This corresponds to the BOD value in the above table. This value of BOD can however vary drastically based on the volume of ablution water and the number of times per day though the usage rarely exceeds one usage per day. It stands to reason to infer that the BOD of septage is relatable to the liquid portion and the suspended matter and the rates at which these have undergone some degradation by anaerobiasis in the tank and the accumulation especially in the sludge zone. All these are highly variable and as such a theoretical basis for arriving at the characteristics of septage is fraught with uncertainty. In respect of the literature values reported from advanced countries in the west, the personal habits of ablution water vs. toilet paper is a crucial influencing factor defying the flat out adoption of the characteristics reported from those locations.

9.5.1.2 Septic Tanks used for all Domestic Sewage

The per capita BOD being 36 g per day and a water usage at about 100 lpcd will imply a BOD of 360 mg/l though it will be higher if the lpcd goes down.

9.5.1.3 Septic Tanks used for Sewage from Water Closets and Bathing

The US EPA "Handbook on Septage Treatment and Disposal - 62568409" identifies Septage as arising from water closets and bath tubs. This is understandably off the mark for the average Indian conditions where the bath tub is first of all a non-entity in the household except in high profile urban living, where incidentally the conventional sewage prima facie eliminates the septage issue.

9.5.1.4 Values Reported from Elsewhere

Given the above understanding of the overall scenario, it stands to reason not to be guided by the characterization data from western countries in the bath tub usage category. A value reported is "BOD concentrations between 2,000 and 20,000 mg/l and TSS values in excess of 50,000 mg/l, where septic tank effluent has values averaging 200 mg/l BOD and 300 mg/l TSS" (Septage Management Guide for Local Governments-David M Robbins). The US EPA in "Handbook on Septage Treatment and Disposal - 62568409" has reported the organic and heavy metals in septage as in Table 9-13 and Table 9-14 (overleaf). The characteristics of Septage reported from the city of Surabaya, Indonesia are BOD of 8,250 mg/l, COD of 17,250 mg/l, and TSS of 2,000 mg/l.

9.5.1.5 Values to be considered for Indian Conditions

Given the above wide variations in literature values and the various influencing factors, it becomes risky to hazard a guess on advocating a set of characteristics for septage in Indian conditions. However, in order to bring about an example of treatment of septage, the values in Table 9-15 (overleaf) are proposed to be advocated purely for illustration and it should be mandatory to carry out local sampling and analysis before designing the treatment and disposal system.

9.6 LOGISTICS OF SEPTAGE COLLECTION

Basically the septage collected should be treated as it cannot be let into the environment directly because of the characteristics in Table 9-15. Because of this, a treatment facility shall be set up or the septage added to an existing septage treatment facility. This implies a near uniform loading all the year round instead of peaking the discharge at certain days alone. This in turn demands the planned septage collection logistics round the year by the septage trucks. Hence the establishment of a septage collection unit becomes an adjunct to the decentralized sewerage system where septic tanks are the primary treatment at households.

9.7 SEPTAGE TREATMENT FACILITY

Sludge generated in an on-site treatment facility is regularly extracted and hygienically treated. The sludge treatment method includes (1) delivery to a sewage treatment facility and treatment with sludge generated in the sewage treatment process, (2) treatment in a special sludge treatment facility, (3) solar drying on a floor, and (4) treatment by a mobile dehydrating truck. This section describes the first and second method.

9.8 TREATMENT OF SEPTAGE IN EXISTING STP

This can be brought about in (a) existing STPs depending on the concentrations of BOD, flows and spare capacity available in them and (b) separate dedicated treatment facility for septage.

Table 9.13 Septage characteristics as per US EPA

Parameter	United States				Europe/Canada				EPA Mean	Suggested Design Value
	Average	Minimum	Maximum	Variance	Average	Minimum	Maximum	Variance		
TS	34,106	1,132	130,475	115	33,800	200	123,860	619	38,800	40,000
TVS	23,100	353	71,402	202	31,600	160	67,570	422	25,260	25,000
TSS	12,862	310	93,378	301	45,000	5,000	70,920	14	13,000	15,000
VSS	9,027	95	51,500	542	29,900	4,000	52,370	13	8,720	10,000
BOD_5	6,480	440	78,600	179	8,343	700	25,000	36	5,000	7,000
COD	31,900	1,500	703,000	469	28,975	1,300	114,870	88	42,850	15,000
TKN	588	66	1,060	16	1,067	150	2,570	17	677	700
NH_3-N	97	3	116	39	-	-	-	-	157	150
Total P	210	20	760	38	155	20	636	32	253	250
Alkalinity	970	522	4,190	8	-	-	-	-	-	1,000
Grease	5,600	208	23,368	112	-	-	-	-	9,090	8,000
pH	-	1.5	12.6	8	-	5.2	9.0		6.9	6.0
LAS	-	110	200	2	-	-	-		157	150

Note:

i) Values expressed as mg/L, except for pH.

ii) The data presented in this Table were compiled from many sources. The inconsistency of individual data sets results in some skewing of the data and discrepancies when individual parameters are compared. This is taken into account in offering suggested design values.

Source: USEPA

Table 9.14 Heavy metals in septage as per US EPA

Parameter	United States			Europe/Canada			Typical US Domestic Sludge Ranges	EPA Mean	Suggested Design Value for Septage
	Average	Minimum	Maximum	Average	Minimum	Maximum			
Al	48.00	2.00	200.0	-	-	-	-	48.00	50.00
As	0.16	0.03	0.5	-	-	-	0-0.7	0.16	0.20
Cd	0.27	0.03	10.8	0.05	-	0.35	0.1-44	0.71	0.70
Cr	0.92	0.60	2.2	0.63	-	5.00	0.9-1,200	1.10	1.00
Cu	8.27	0.30	34.0	4.65	1.25	15.00	3.4-416	6.40	8.00
Fe	191.00	3.00	750.0	-	-	-	-	200.00	200.00
Hg	0.23	0.0002	4.0	-	0.15	0.20	0-2.2	0.28	0.25
Mn	3.97	0.20	32.0	-	-	-	-	5.00	5.00
Ni	0.75	0.20	37.0	0.58	-	2.50	0.5-112	0.90	1.00
Pb	5.20	2.00	8.4	3.88	-	21.25	3.2-1,040	8.40	10.00
Se	0.076	0.02	0.3	-	-	-	-	0.10	0.10
Zn	27.4	2.90	153.0	38.85	1.25	90.00	79-655	49.00	40.00

Note:

i) Values expressed as mg/L.

ii) Values converted from µg/g assuming TS=40,000 mg/L.

Source: USEPA

Table 9.15 Illustrative characteristics of septage for Indian Conditions

No.	Source	Type A	Type B
		Public toilet or bucket latrine sludge	Septage
	Characteristics	Highly concentrated, mostly fresh Faecal Sludge; stored for days or weeks only	Faecal Sludge of low concentration; usually stored for several years; more stabilized than Type "A"
1	COD (mg/L)	20-50,000	<15,000
2	COD/BOD	5:1 to 10:1	5:1 to 10:1
3	NH_4-N (mg/l)	2-5,000	<1,000
4	TS (%)	≥ 3.5 %	< 3 %
5	SS (mg/l)	≥30,000	7,000 (approx.)
6	Helminth Eggs	20-60,000	4,000 (approx.)

9.8.1 Pre-Treatment of Septage

This is needed to (a) ensure a flow equalization tank for the septage flow so that it can be loaded onto the STP at as much uniform flow as possible through the 24 hours, (b) a degritting facility to segregate the grit content and prevent it from getting into aeration units and pumps etc. and (c) separate the liquid stream and sludge stream.

The equalization tank may be a relatively deeper tank equipped with sub surface mixers to maintain the contents in suspension. The surface aerators and diffused aeration will create odour problems.

The degritting facility is best designed as a vortex separator similar to the one described in Chapter 5. The sludge-liquid separation facility can be a filter press or belt press or screw press or centrifuges depending on the feed solids concentration being within the capacity of these equipment. Their designs will be the same as in Chapters 5 and 6.

The pumps however, can be submersible pump sets with open impellers. A typical receiving station facility is shown in Figure 9.18 (overleaf)

9.8.1.1 Co-treatment in Existing STPs-Liquid Stream

The basic consideration is the spare capacity at the existing STP. Normally, the septage volumes are not unduly significant in relation to the full-fledged STP volumes and would seldom exceed about say 5% and this way, even if the STP is functioning at design capacity, volume wise, it will not be a problem to add even up to 5% of flows. But it is the BOD load that comes in the way.

Considering a typical STP with about 300 mg/l of raw BOD and a septage volume of about 3% with a BOD of say 4,000 mg/l, this would result in a situation mentioned overleaf.

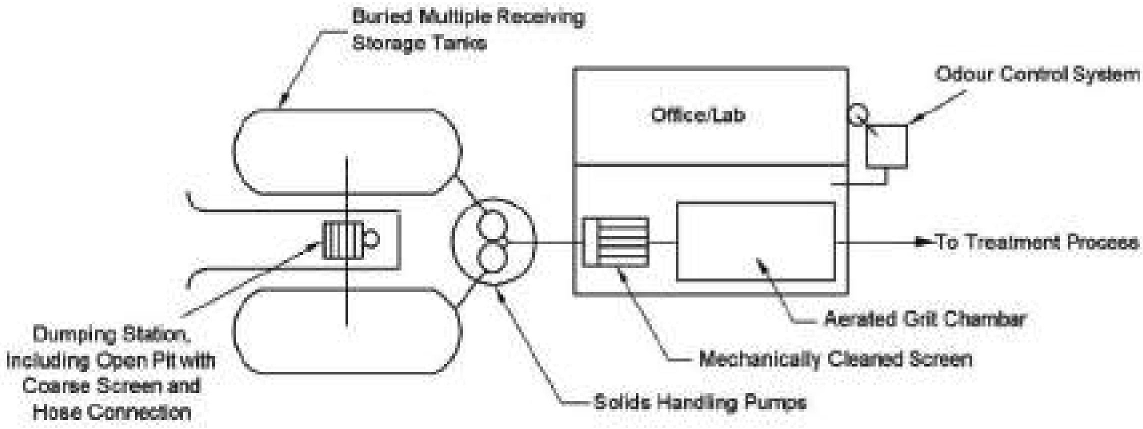


Figure 9.18 Typical septage receiving facility

Design capacity of the STP	= 1 mld
Actual operating capacity	= 0.65 mld
BOD load in to the STP	= 0.65 mld × 300 mg/l = 195 kg/day
BOD load from septage	= 0.03 mld × 4,000 mg/l = 120 kg/day
Total resulting BOD load	= 195 + 120 = 315 kg/day
Designed ability of the STP for BOD load	= 1 × 300 = 300 kg/day

Thus, it may be possible to accommodate the septage as long as the actual flow to STP does not increase. But then, over a period of time both the sewage volume and septage will increase and hence, it is not easy to use this option as a permanent measure. At the same time, if the spare capacity is available, then it is wiser to opt for this instead of rushing into a dedicated septage treatment facility. Yet another option will be to augment or upgrade the STP capacity, which is by far simpler and so far as the liquid stream is concerned.

9.8.1.2 Co-treatment in Existing STPs-Sludge Stream

The computations similar to that in respect of liquid stream shall be evaluated to verify whether the sludge treatment facilities of the existing STP can handle the extra sludge from the septage. Most often, this may be possible. In case it is not possible, add on sludge treatment standalone facilities shall be designed and constructed instead of trying to invasive augmentations of existing facilities.

9.8.1.3 Points of Addition of the Liquid and Sludge Streams

The points of addition of liquid and sludge streams provided spare capacities are available are suggested in Figure 9.19 (overleaf)

9.9 TREATMENT OF SEPTAGE AT INDEPENDENT SeTP

When the distance or the capacity of the plant becomes a limiting factor, it is not a feasible option to transport and treat the septage to the sewage treatment facilities.

In this case treatment plants specially meant for septage treatment becomes an attractive option. Independent septage treatment plants are designed specifically for septage treatment and usually have separate unit processes to handle both the liquid and solid portions of septage.

These facilities include mechanical dewatering, sludge drying beds, Waste stabilization ponds, etc. The benefit of using these treatment plants is that they provide a regional solution to septage management. Many septage treatment plants use lime to provide both conditioning and stabilization before the septage is dewatered. Dewatered sludge can be used as organic fertilizer after drying and composting. The remaining effluent/filtrate/supernatant can be released to another treatment process such as WSP, Anaerobic baffled reactor, constructed wetland or combination of these of extended aeration activated sludge where it can undergo further treatment and then finally can be safely discharged.

Choosing an appropriate septage management method relies not only on technical aspect but also on regulatory requirements. The management option selected should be in conformity with local, State and Central regulations. Some of the factors that determine the process of selection include: land availability and site conditions, buffer zone requirements, hauling distance, fuel costs, labour costs, disposal costs and other legal and regulatory requirements.

The technical options could be as follows:

Case 1: Land Area is not limited but Funds are Limited

Option - 1

Pretreatment - Anaerobic Digesters - Dewatered and Dried Sludge - Composting - Reuse as Organic Fertilizer; Filtrate of Sludge Drying Bed and Digester supernatant - Pumping - Reed beds (or) Constructed wetlands - Electricity generation from digester gas. Totally nature based system with mechanical equipment as needed.

Constructed wetlands are essentially on-site technologies involving sequential treatment of sewage on-site, in selective filter media and finally greenbelt development and have been developed by IIT Powai and called Soil Bio Technology and also NEERI and called as Phytorid.

A septage treatment facility handling nearly 51 MLD at Nonthaburi in Thailand is widely reported in literature. The treatment process is shown in Figure 9.20.

It is reported that the treatment is anaerobic digestion and the digested sludge is sent to drying beds. The filtrate is dewatered in sand beds and is sent to ponds and the pond effluent is used on public parks. The use of constructed wetlands has also been reported with solids loading rate of 250 kg/m^2/year, once a week application and percolate impounding for 6 days and harvesting twice a year with COD removal efficiency of 80 to 90%, and solids accumulation at 12 cm/year in the impoundment.

A photo view is presented in Figure 9.21.

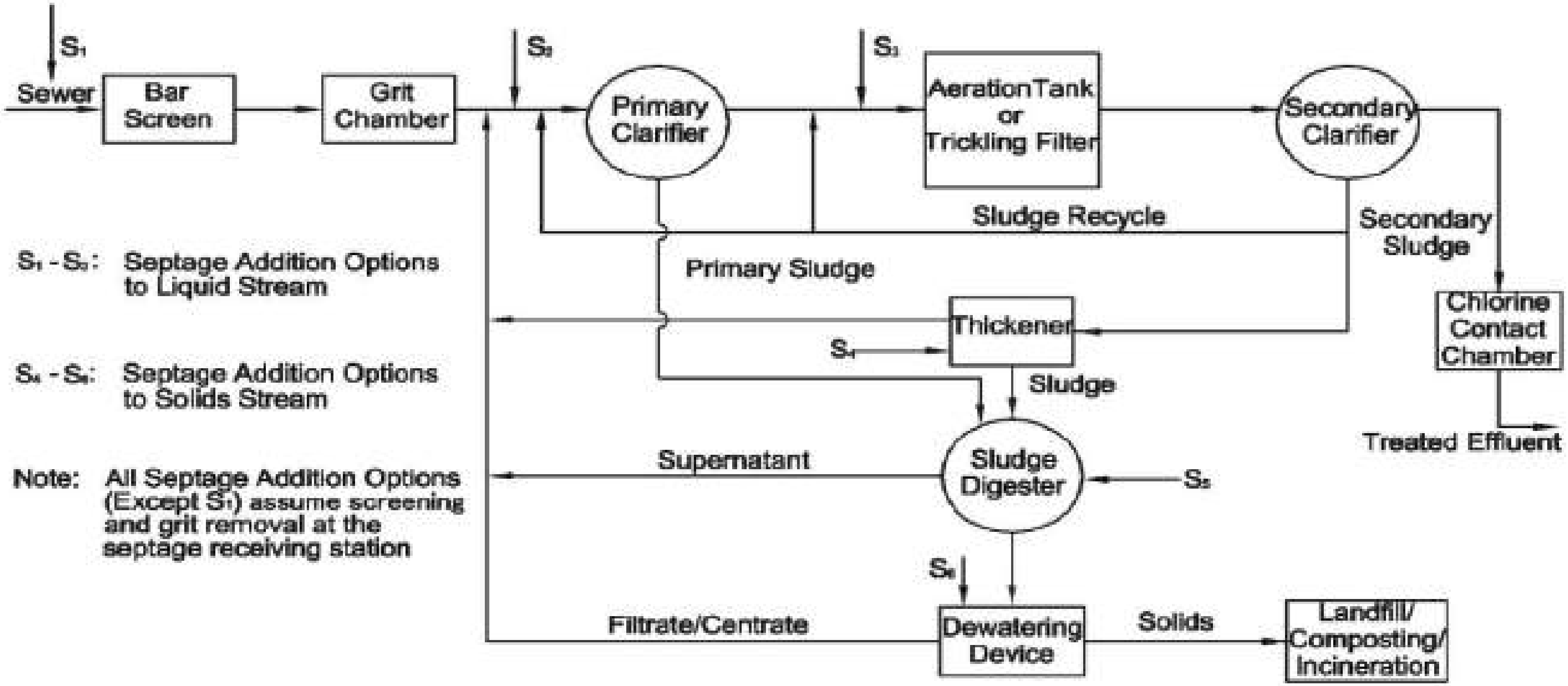
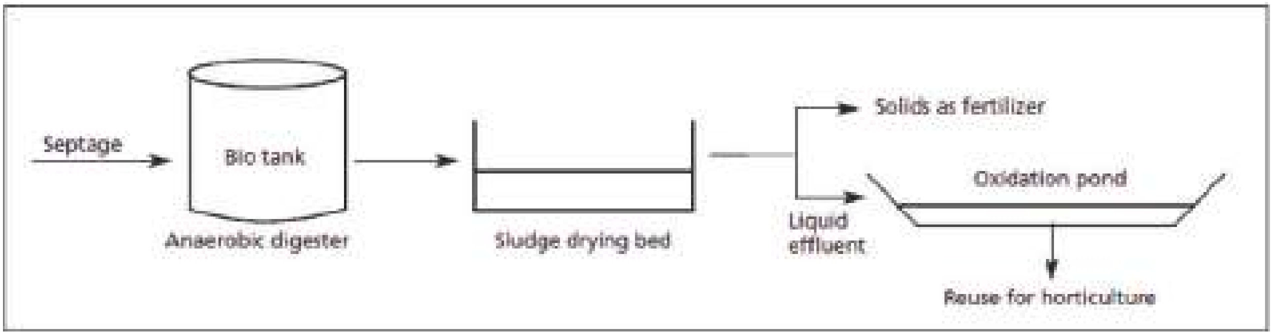


Figure 9.19 Points of likely addition of liquid and solids from septage on to existing STPs



Source:USAID, 2010

Figure 9.20 Septage treatment process at Nonthaburi, Thailand

The Nonthaburi Municipality's septage treatment facility has 30 anaerobic digestion tanks and is one of the best run facilities of its kind in Thailand. As part of a pilot project to improve septage treatment in Lampang, the Asian Institute of Technology partnered with Lampang Municipality to build a treatment facility that uses a grid of reed beds to treat septage leachate to secondary levels.

Source:USAID, 2010

Figure 9.21 A photo view of septage treatment process at Nonthaburi in Thailand

Option - 2

Pretreatment - lime stabilization (optional) - Pumping - Sludge Drying beds (FRP covered in regions of high rainfall) - Dewatered & Dried Sludge - Composting - Reuse as Organic Fertilizer; Filtrate of Sludge Drying Bed - Pumping - Anaerobic Baffled Reactor / Covered Anaerobic Ponds - Facultative - Aerobic / Maturation Ponds - chlorination - discharge'

Case 2: Land Area is Limited and Funds are also Limited.

Option - 1

Pretreatment - lime stabilization (optional) - Pumping - Mechanical Sludge dewatering system - Dewatered sludge - Solar drying or/and Composting - Reuse as Organic Fertilizer; Filtrate of Mechanical Dewatering Machine - Pumping - Anaerobic Baffled Reactor / Covered Anaerobic Ponds - Facultative - Aerobic / Maturation Ponds - chlorination - discharge. There should be 25% additional capacity of sludge drying beds in case of maintenance of dewatering machine and or unavailability of polyelectrolyte.

Option - 2

Pre-treatment - lime stabilization (optional) - Pumping - Sludge drying beds (FRP covered in regions of high rainfall) - Dewatered & Dried Sludge - Composting - Reuse as Organic Fertilizer; Filtrate of Sludge / septage Drying Bed - Pumping - Extended Aeration - Activated Sludge Process (Continuous or Batch) - chlorination - discharge

Case 3. Land Area is Limited and Funds are not limited

Pretreatment - lime stabilization (optional) - Pumping - Mechanical Sludge dewatering system - Dewatered sludge - Solar drying or/and Composting - Reuse as Organic Fertilizer; Filtrate of Mechanical dewatering machines - Pumping - Extended Aeration Activated Sludge Process (Continuous or Batch) - chlorination - discharge. There should be 25% addition capacity of sludge drying beds in case of maintenance of dewatering machine and or unavailability of polyelectrolyte.

9.9.1 Pre-Treatment of Septage

The pre-treatment facilities discussed earlier are the same in this case also (Figure 9-18).

In addition, if possible there should be lime stabilization facility to control odour, vector and pathogen destruction. Lime stabilization involves adding and thoroughly mixing lime (alkali) with each load of septage to ensure that the pH is raised to at least 12 for at least 30 minutes.

Lime addition could be done at any of these three points:

i) To the hauler truck before the septage is pumped,

ii) To the hauler truck while the septage is being pumped, or

iii) To a septage storage tank where septage is discharged from a pumper truck is shown in Figure 9.22 (overleaf)

9.9.2 Septage Dewatering

The septage after lime dosing is pumped to screw press or any other mechanical dewatering machine. Polyelectrolyte is added to increase the dewatering efficiency of the machine.

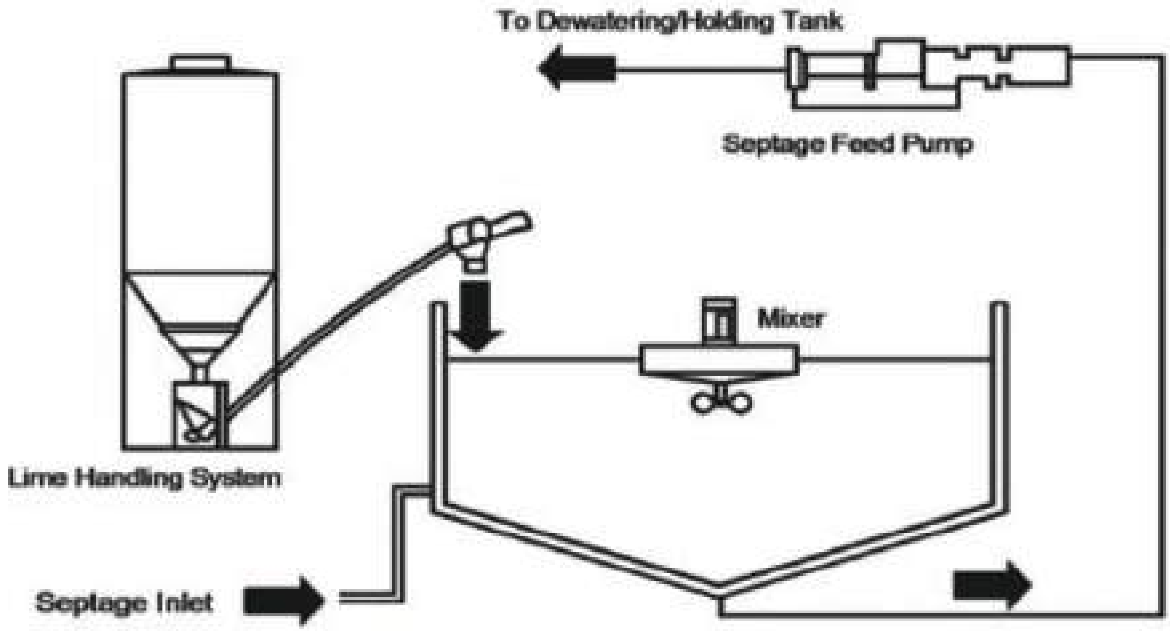
The liquid residual / pressate / filtrate / supernatant from dewatering machine can be discharged for further biological treatment. The dewatered sludge can be send for further drying or composting prior to reuse as organic fertilizer. The typical mechanical septage dewatering system is shown in Figure 9.23 (overleaf)

Instead of Screw Press the options can be:

i) Centrifuge

ii) Belt Press

iii) Filter Press



Supernatant to pH neutralization and dedicated STP

Underflow to mechanical dewatering with polyelectrolyte

Figure 9.22 Lime stabilization of septage

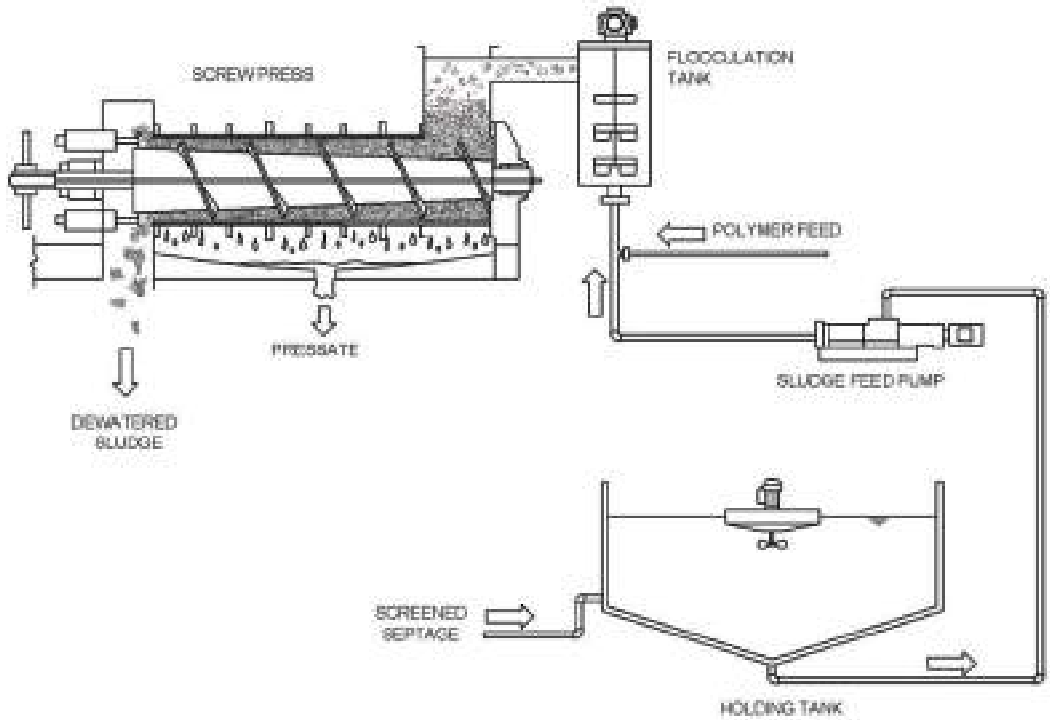


Figure 9.23 Typical Mechanical Septage Dewatering System

9.10 ADVANTAGES AND DISADVANTAGES OF THE SYSTEMS

Advantages and disadvantages of septage treatment at STPs and at independent septage treatment facility are given in Table 9.16 and Table 9.17, respectively.

Table 9-16 Advantages and disadvantages of septage treatment at sewage treatment plant

Method	Description	Advantages	Disadvantages
Treatment at STPs or Independe nt septage treatment facility in the vicinity of STPs	Septage is added to the pumping station, upstream manhole or sludge treatment process for co-treatment with sewage sludge. Septage volumes that can be accommodated depend on plant capacity and types of unit processes employed.	Most STPs in India are underutilized and are capable to handle some septage. As skilled personnel and laboratory facilities are available in STPs, easy to operate and maintain.	Potential for STP upset if plants are running at full capacity. Increased sludge treatment cost.

Table 9.17 Advantages and disadvantages of independent septage treatment facility

Method	Description	Advantages	Disadvantages
Treatment at independent septage treatment plants	A facility is constructed solely for the treatment of septage. Treatment generates residuals, i.e., dewatered sludge and filtrate which must be dried composted (dewatered sludge) and properly treated (filtrate) prior to being disposed off.	Provides regional solutions to the septage management.	High capital and operation and maintenance cost. Requires high skills of operation in case of mechanical dewatering.

9.11 COMPOSTING OF DEWATERED SEPTAGE OR SLUDGE

Another feasible option is composting where bulking agents are easily available. The humus is produced after composting that can be used as a soil conditioner. Composting is the stabilization of organic waste through aerobic biological decomposition. As described in more detail in Chapter 6, the process can be accomplished in various configurations. The different types of composting include two open-area methods: windrow and static pile composting and in-vessel mechanical composting. Operational parameters for septage composting are presented in Table 9-18 (overleaf) Compost products can be sold or given away.

Table 9.18 Operational parameters for dewatered septage composting

Parameter	Optimum range	Control mechanisms
Moisture content of compost mixture	40-60%	Dewatering of septage to 10 to 20% solids followed by addition of bulking material (amendments such as sawdust and woodchips), 3:1 by volume amendment: dewatered septage.
Oxygen	5-15%	Periodic turning (windrow), forced aeration (static pile), mechanical agitation with compressed air (mechanical).
Temperature (compost must reach)	55-65°C	Natural result of biological activity in piles. Too much aeration will reduce temperature.
pH	5-8	Septage is generally within this pH range, adjustments not normally necessary.
Carbon/nitrogen ratio	20:1 to 30:1	Addition of bulking material.

9.12 DEWATERED SEPTAGE SLUDGE REUSE

For dewatered septage/sludge agriculture application, it should satisfy the following criteria of Class A Biosolids of US EPA either by lime stabilization, solar drying and or composting.

- A faecal coliform density of less than 1,000 MPN/g total dry solids
- Salmonella sp. density of less than 3 MPN per 4 g of total dry solids (3 MPN/4 g TS)

Properly treated sludge can be reused to reclaim parched land by application as soil conditioner, and as a fertilizer in agriculture. Deteriorated land areas, which cannot support the plant vegetation due to lack of nutrients, soil organic matter, low pH and low water holding capacity, can be reclaimed and improved by the application of sludge.

Septage sludge has a pH buffering capacity resulting from lime addition that is beneficial in the reclamation of acidic sites, like acid mine spoils, and acidic coal refuse materials.

Sludge with a solid content of 30% or more handled with conventional end-loading equipment, and applied with agricultural manure spreaders. Liquid sludge, typically with solid content less than 6% managed and handled by normal hydraulic equipment.

Agricultural use of sludge matches best with priorities in waste management. Sewage sludge contains nutrients in considerable amounts, which can be used as discussed in Chapter 6 of the part A manual.

CHAPTER 10: PREPARATION OF CITY SANITATION PLAN

10.1 THE PLANNING PROCESS

Planning is a thinking process. In sewerage and sewage treatment, it aims at identifying how best the required infrastructure can be conceived in mind and given shape within the restrictions of available funds and satisfying the public as far as possible. For example, a twin pit latrine is a boon in remote hilly area, but totally unfit in a city. Thus, planning has to be above all "relevant to situation on hand". The planning process is a systematic method of:

1. Understanding the existing needs
2. Identifying the limitations and restrictions of funds
3. Collecting and analyzing available records of these
4. Identifying the options of potential remedies
5. Suggesting a set of actions, which may change the situation and step-by-step eliminate the problems
6. Evolve a suitable strategy for implementation with respect to a time frame
7. Go through a consultative process with the stakeholders to evolve a complete acceptance of physical, financial and managerial aspects
8. Evaluation of the actions taken for their success or failure and documentation for posterity
9. Thus, planning is a continual process and not a one-time process adopting principles and technology which are environment friendly, economically viable and sustainable
10. It also includes the reuse of the reclaimed water from treated sewage and conditioned sludge for feasible purposes that are hygienically safe
11. It needs close collaboration with other planning agencies at local, state and national levels to ensure co-ordination in allocation of priorities and resources
12. All these must be aimed to be reached in a step-by-step manner so that the lessons of the earlier step will improve the efforts in the next step

10.2 THE CITY SANITATION PLAN (CSP)

A city sanitation plan (CSP) is a living document as a result of the planning process.

Every ULB should have a city sanitation plan and undertake to implement it for all its citizens in an economic, environmentally friendly and sustainable manner.

10.3 DESIGN PERIOD

The following design period could be considered:

i) Short-term plan up to 5 years from base year
ii) Medium-term plan up to 15 years
iii) Long-term plan 30 years

The base year for short term will start when the completed infrastructure is put to use. The years of medium term and long term will start from the year of planning

The planning process involves close collaboration with other planning agencies at local, state and national levels to ensure better coordination in allocation of priorities and resources. The collection, transportation, treatment and disposal aspects, facilities, augmentation and replacement of the equipment and sites, allocation of priorities and resources should invariably be decided keeping in view the design period of the CSP.

10.4 POPULATION FORECAST

The design population will have to be estimated considering the decadal growth pattern and factors impacting growth such as economy, social, etc. Special factors causing sudden emigration or influx of population should also be foreseen to the extent possible. Worked out examples for estimation of the future population are given in Appendix A.2.2.

10.5 BASIC PLANNING MODEL

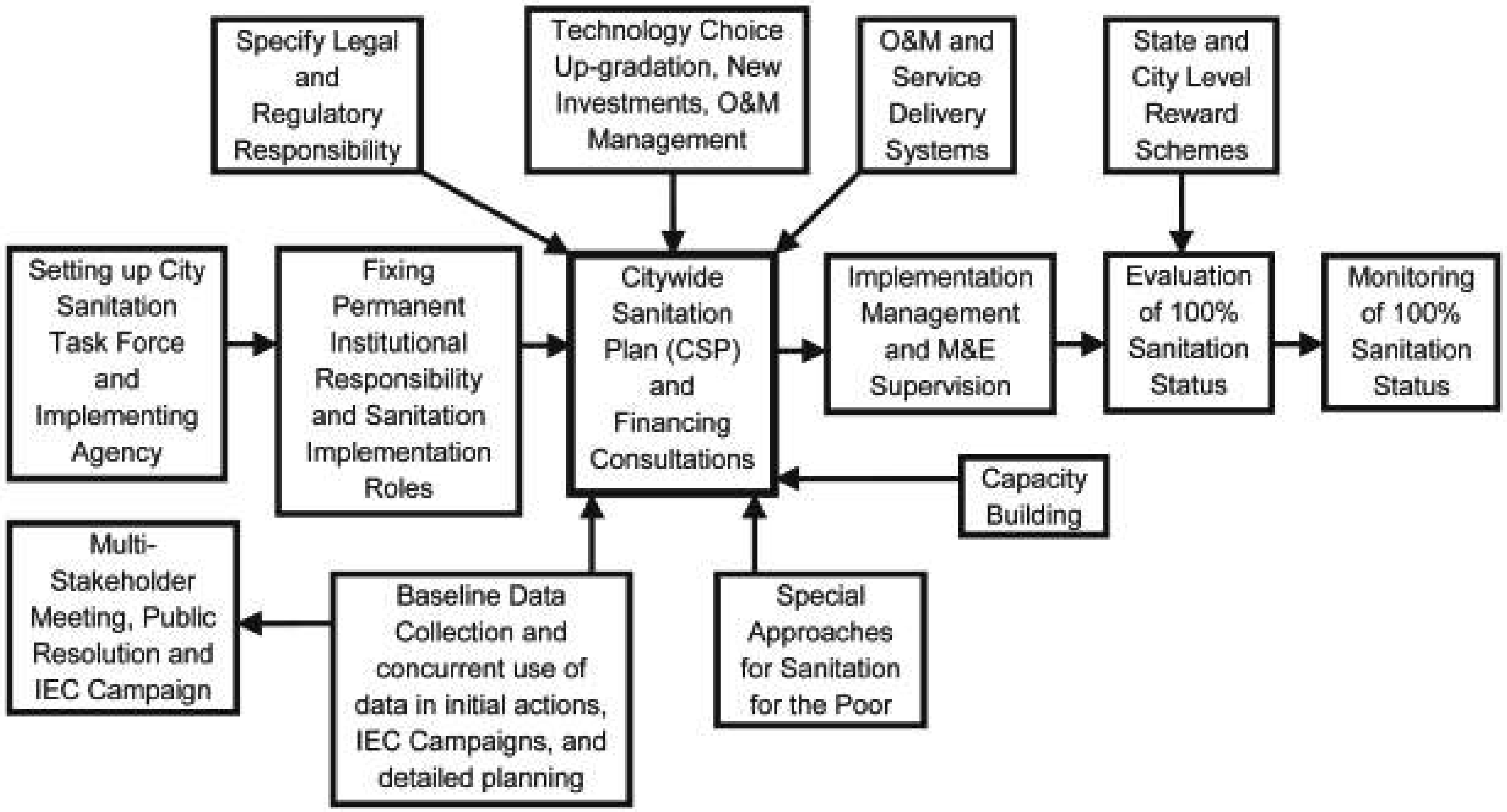
10.5.1 Draft Framework for a CSP under NUSP

10.5.1.1 Generic Elements of Planning

This shall be in accordance with the chart as contained in the National Urban Sanitation Policy (NUSP) and reproduced here as Figure 10.1 overleaf.

10.5.1.2 Purpose

The purpose of this framework is to assist ULBs, NGOs, community based organizations, citizens and private sector agencies through a series of steps towards the development of a plan for achieving the goal of 100% sanitation in their cities. The focus is the approach and “how to go about” the process to develop a comprehensive, wholesome citywide sanitation plan. Since each city will make choices based on demand and need, local context, availability of financial and human resources, and the opportunity for innovations, this chapter does not prescribe the options to choose. The framework may be adapted to suit the state’s urban sanitation strategy and used for its cities. To assist in thinking through the challenge, some core building blocks are outlined in Figure 10.1. Though apparently linear, the process needs to be iterative.



Source: MoUD, 2008

Figure 10.1 Generic elements of planning, implementation and monitoring and evaluation of citywide sanitation

The states will need to determine time-frames and deadlines to achieve the goals mentioned in the NUSP and will need to spell out a detailed road map, including the incremental targets for achievement of goals. For example, to achieve the goal of open defecation free (ODF) by the year 2013, a detailed plan for extending access will need to be formulated and implemented in a time-bound manner. The steps towards achieving universal access through individual, community or public toilets, the capital and operation and maintenance costs and the management arrangements needs to be detailed and made operational under the CSP. While some of the activities in the sanitation plan may be possible to complete with little financial resources e.g., better utilization of existing facilities, improved management systems for septage cleaning, awareness generation, etc.; others e.g. reconditioning or laying new sewers, may be more resource-intensive. The CSP will need to be prepared keeping in view the city's current sanitation arrangement and their technical and financial capability. It will be prudent to improve the effectiveness of existing facilities before embarking on new projects. Further, comprehensive and citywide solutions, and not just some piecemeal solution, will be necessary to achieve the goals in a comprehensive and systematic manner.

10.5.2 Steps for Achieving 100% Sanitation

10.5.2.1 Key Principles

The NUSP identified the following core principles that need to be addressed. These must be used as a guide by the cities:

- Institutional roles and responsibilities
- Awareness generation for changing mind-sets
- Citywide Approach
- Technology choice
- Reaching the un-served and poor
- Client focus and generation of demand
- Sustained improvements

10.5.2.2 Preparatory Actions

10.5.2.2.1 City Sanitation Task Force

Mobilize Stakeholders: The first step in making the cities 100% sanitized is to create awareness on the need to improve sanitation in the mind of municipal agencies, civil society and most importantly, amongst the people of the city. These can be done by the following approaches.

a) Constitute a multi-stakeholder City Sanitation Task Force comprising representatives from

- Agencies directly responsible for sanitation including on-site sanitation, sewerage, water supply, solid waste, drainage, etc., including the different divisions and departments of the Urban Local Bodies (ULB), Public Health Engineering Department (PHED), etc.,

- Agencies indirectly involved in or impacted by sanitation conditions including representatives from the civil society, colonies, slum areas, apartment buildings, etc.,
- Eminent persons and practitioners in civic affairs, health, urban poverty,
- Representatives from shops and establishments,
- Representatives of other large institutions in the city (e.g. Cantonment Boards, Government of India or State Government. Enterprise campuses, etc.),
- NGOs working on water and sanitation, urban development and slums, health and environment,
- Representatives of unions of safai karamcharis, sewerage sanitary workers, recycling agents / kabaris, etc.,
- Representatives from private firms/contractors formally or informally working in the sanitation sector (e.g. garbage collectors, septic tank desludging firms, technology providers for sewage and sludge treatment, etc.),
- Representatives from educational and cultural institutions,
- Elected members from the State Assembly and City Councils,
- Any other significant or interested stakeholders.

The Task Force should be headed by the Mayor with the executive head (e.g., Municipal Commissioner) as the Convener. Cities can also choose to appoint, as a part of the Task Force, City Sanitation Ambassadors chosen from eminent people, who enjoy outstanding credibility and influence amongst the city's leadership and population. Political leadership from all political parties and persuasions must be involved in the planning process so that the sanitation campaign has their full support and no opposition from any group.

One of the things to be considered by the Task Force is to organize a multi-stakeholder, multi-party meeting in the preparatory stage, and take a formal resolution to make the city 100% sanitized, and publicize the same and disclosing with all signatories.

b) The City Sanitation Task Force will be responsible for:

- Launching the City 100% Sanitation Campaign
- Generating awareness amongst the city's citizens and stakeholders
- Approving materials and progress reports provided by the implementing agency, other public agencies, as well as NGOs and private parties contracted by the Implementing Agency, for different aspects of implementation
- Approving the CSP for the city prepared by the Sanitation Implementation Agency after consultations with citizens
- Undertaking field visits from time to time to supervise progress

- Issue briefings to the press/media and state government about progress
- Providing overall guidance to the Implementation Agency
- Recommend to the ULB fixing of responsibilities for citywide sanitation on a permanent basis

The Task Force should meet formally frequently (at least once in two months) in the initial stages to monitor and guide the process of planning and implementation. At a later stage, meetings and field visits can be on an as-needed basis. In some cities, the City Sanitation Task Force may divide up roles and responsibilities amongst smaller sub-committees to focus on different aspects closely while keeping the overall character of the Task Force intact.

c) The Task Force should appoint one of the key agencies, preferably the ULB, as the Implementing Agency, which will be responsible for the implementation of the CSP for the city.

This agency will be responsible for day-to-day coordination, management and implementation of the sanitation programmes on a citywide basis. The agency will coordinate with and agree on joint actions with other public agencies, and contract in and supervise the services of NGOs (through Memorandum of Understanding) and private parties (through contracts) for preparing and disseminating materials for Information, Education and Communication (IEC), conducting baseline surveys and stakeholder consultations, maintaining a comprehensive GIS-based database, implementing physical works, letting out and supervising O&M management contracts, etc.

The ULB should formally notify and publicize the appointment of the City Sanitation Task Force and Implementing Agency.

d) Assign Institutional Responsibilities:

One of the key gaps in urban sanitation is lack of clear and complementary institutional responsibilities. This comprises two aspects: a) roles and responsibilities institutionalized on a permanent basis; and b) roles and responsibilities for the immediate campaign, planning and implementation of the City's Sanitation Plan - based on which the former can be outlined, experimented with, and finally institutionalized.

The Sanitation Task Force will recommend the assigning of permanent responsibilities for citywide sanitation to the ULB or other agencies including the following aspects:

- The ULB to have final overall responsibility for citywide sanitation, including devolving power, functions, functionaries and funds to them
- Planning and Financing including State Government and Government of India schemes
- Asset creation including improvement and augmentation
- Operations and Management (O&M) arrangements for all networks, on-site, community and public sanitation facilities and systems (including transportation up to final treatment and disposal of wastes)

- Fixing tariffs and revenue collections in order to make O&M sustainable
- Improving access and instituting special O&M arrangements for the urban poor and un-served populations in slum areas and in mixed areas
- Adopting standards for:
  - Environment Outcomes (e.g. State Pollution Control Board standards on effluent parameters),
  - Public Health Outcomes (e.g. State Health Departments),
  - Processes (e.g. safe disposal of on-site septage) and
  - Infrastructure (e.g. design standards) (PHEDs/Parastatals), and
  - Service Delivery standards (e.g. by Urban Development Departments)
- Adoption of Regulatory roles including environmental standards (e.g. State Pollution Control Boards), health outcomes (e.g. Health Departments).
- Measures in case specific stakeholders do not discharge their responsibilities properly
- Training and Capacity Building of implementing agency and related personnel
- Monitoring of 100% Sanitation involving multiple stakeholders

While the responsibilities for each of the above roles may temporarily be vested in one or the other stakeholders, for reasons of efficiency and effectiveness during the campaign period, the Task Force will recognize that these roles must be permanently institutionalized in the ULB and amongst other stakeholders. Therefore, the recommendation of later permanent roles may be different from those in the Campaign Period.

In many cases, Acts, rules and regulations exist, but these are not enforced. This may be a good entry point to start on roles and responsibilities. The roles and responsibilities for the Sanitation Plan implementation are outlined in the next section - this will also be the task of the City Sanitation Task Force.

10.5.2.3 Baseline Data Collection for Database / GIS

In parallel with the preparatory steps, the ULB / implementing Agency will collate the information on the current sanitation situation that exists in the city. This will include demographic, institutional, technical, social and financial information. In addition, it will commission a private agency or an NGO or both to carry out primary data collection on the missing items – the surveys will use a mix of structured and participatory techniques. All the data collected must be amenable to linking to an existing or proposed Geographic Information Systems (GIS) for the city. (If this does not exist, it is recommended that a GIS for water, sanitation and solid waste management be set up at the earliest). The baseline will be overlaid on plans for development of new areas and colonization, based on the Master Plan of the City. If a Master Plan does not exist, appropriate projections will be made after consulting real estate development public authorities as well as private agencies.

The combined database from the above exercise will form the basis for planning and implementing the campaign. Since such data collection can be time-consuming, ULBs must start very early on this activity and start using data as and when it starts becoming available.

One of the methods to make data collation and database preparation process efficient and adaptive to planning and implementation actions, is to break it down into simplified components like:

Stage I Data: use for initial preparatory actions

- ULB, and utility/service provider data on institutional parameters (organizational structure, investments and assets, personnel, O&M systems and finances),
- Census 2011 data on households, JnNURM / Urban infrastructure development scheme for small and medium towns (UIDSSMT) or other scheme's data compiled for poor households
- ULB and utility/service provider data on public sanitation and available crude data on conveyance and treatment.

Stage II Data: use for IEC Campaign and planning to achieve universal access to sanitation on a citywide basis.

- Refined secondary data on existing conditions of disposal and conveyance (sewers, on-site pits, availability and use of suction machines, etc.) and treatment systems (landfill sites, recycling, etc.)
- Baseline primary data on household arrangements for sanitation and waste disposal, and hygiene behaviour and perceptions about service providers
- Baseline primary data on citizen's demands and perceptions about sanitation arrangements, outcomes, health and environmental linkages

Stage III Data: Use for planning and implementing institutional changes, social mobilization and upgradation, improvements and new investments in assets and systems of O&M, monitoring and evaluation, etc.

- Primary data based on sample condition assessment surveys (see parameters above) of arrangements, disposal and treatment systems.
- Institutional Assessment detailed information on existing and required skills and capacities, systems and procedures, financial position
- Social – personal hygiene and public health behaviour and practices
- Economic – Surveys on willingness to pay for different options
- Financial – Costs of O&M, Revenue and tariffs, systems of community management of community and neighbourhood level systems

Usually, a baseline study needs to be completed in about three to four months (Class II and above), depending on the size of the city and complexities involved. About two months is adequate to complete baseline in cities of Class III and below. Combining participatory approaches with institutional and other stakeholders, with observation and community and household interactions using checklists, schedules, etc., makes the data collection efficient and economical. It may be noted that the baseline is not a census of all properties and households/units. It is rather an assessment, usually using sampling to cover all representative types of situations prevailing in the city, in order that progress can be measured at later points comparing with the baseline. Most immediately, baseline studies are required for planning the citywide sanitation plan. It is advisable to cover all aspects during the baseline: technical, institutional, social, economic, financial, urban poor, etc., and be cautious that none of the aspects are left out. Even if the baseline studies are completed in a short period – this is necessary so that planning processes are not kept on hold for long – further data collection and updating of records must continue later on too, and become a part of the ULB/ Implementation Agency's implementation management system.

10.5.2.4 Awareness Generation and Launch of 100% Sanitation Campaign

After a reasonable amount of data has been collated from secondary and primary sources, and the Task Force is in place, the first task will be launching a citywide 100% Sanitation Campaign. This will be ideally timed with GOI national media campaign, and a state wide campaign that the state government may choose to launch. If required, a professional media agency to work closely with the Task Force and Implementing Agency to package the messages and direct them effectively to different stakeholder groups in the city. NGOs may be commissioned to do group messaging and door-to-door campaigns with special stakeholders like slum-dwellers etc. Schools and Colleges can play a special role in propagating the messages in their institutions as well as in their families.

At the city level, it will be advisable to launch the campaign as a time-bound programme that all stakeholders need to work towards. Appropriate media like Newspapers, TV and city and ward/ neighbourhood level programmes (sweeping streets, health camps, tree-planting, etc.) may be engaged. There should be an intensive first round followed by successive rounds that may be focused on specific aspects and/or special type of stakeholders, or neighbourhoods. One of the methods that some cities or neighbourhoods may try out is to declare Clean City Week every year or half-year. The Task Force should enlist the participation of leaders and eminent persons to lead the campaigns. The messages and media/campaign strategy for each of the successive rounds must be planned carefully. There are a number of other programmes (e.g. health, education, HIV/AIDS, etc.) that have media campaigns. The 100% Sanitation campaign should be coordinated with such agencies so that maximum multipliers can be gained by collaborative and calibrated working of these initiatives. Wherever possible, messages should be put in other campaigns to reinforce the impact.

10.5.2.5 Specifying Legal and Regulatory Institutional Responsibilities

Even though there are municipal laws with regard to sanitation responsibilities of households and ULB, etc., these are neither clearly laid out nor comprehensive. The Implementing Agency will examine the law and rules in this regard and make recommendations regarding:

- Safe sanitary arrangements at unit level (household, establishment)
- Designs and systems for safe collection
- Norms for transport/conveyance
- Treatment and final disposal

The recommended standards and guidelines are available from the CPHEEO and the Environment Acts. These will need to be formally adopted including laying down the monitoring and regulatory responsibilities, and incentives and disincentives for doing so. This must include the system of user charges/fees, fines and community pressure mechanisms to help people move to desirable public health behaviour. Actions to be taken in case of institutional failure will also be specified clearly.

All the above recommendations will be considered by the Task Force and recommended to the ULB for appropriate action. Executive changes may be implemented immediately, whereas legal matters may be referred to the State Government if not within the ambit of the ULB. Expert advisors on the Sanitation Task Force will be the resources to utilize for this task – matters may be discussed with national or state level agencies if standards are not clear, or need to be further detailed. Interim and working standards may suffice in many cases to immediately adopt and implement, whereas the codification and detailing may be undertaken in parallel. In all cases, the Task Force will strive to make standards based on the goals of 100% Sanitation, as much as possible, simple and easy for ULBs and public to understand and adhere to.

10.5.2.6 Planning and Financing

The task of planning and finding sources of funding will be under the oversight of the Task Force, but carried out by the Implementing Agency. The Agency will take assistance from consultants, etc., to help prepare plans for the city for different aspects including institutional, social, technical, financial, etc. At all stages, the plans must be comprehensive and cover the whole of the city, and not just one part or aspect. Therefore, a number of innovative measures may have to be used.

The Government of India's Jawaharlal Nehru National Urban Renewal Mission (JNNURM), Basic Services to Urban Poor (BSUP), and Thirteenth Finance Commission (TFC) are the key programmes to source funding (others being special programmes for the North-East and satellite towns schemes, etc.), apart from State Government's own resources. Planning should be aligned to the above funding sources (as well as what customers are willing to pay by way of connection fees, user charges, etc.), and seek to derive maximum benefits from these sources for achieving 100% sanitation. The City and States will also need to explore other sources of finance to fund their sanitation plans since Government of India scheme resources may not be enough to fulfill all requirements. In this context, it may also be noted that investments will need to be financially sustainable and hence, cities may lay down options (different levels of infrastructure and service levels) depending on what they can afford in the medium term, and what will prevent them from getting trapped in high loan repayment liabilities, or O&M management expenditure bubble at a later point in time. The CSP must be prepared and presented by the Implementing Agency and presented to the Task Force for approval. While the exact contents of the CSP may vary depending on the local situation, the aspects mentioned overleaf must be covered:

- Plan for Development of Institutions/Organizations responsible for sanitation, and their roles and responsibilities
- Plan for ensuring 100% Sanitation Access to different socio-economic groups, and related O&M systems (including improving existing systems, supplementary facilities, O&M Management contracts using PPP and community management, etc.)
- Costs and tariffs for service provision'
- The issue of collection of dues needs to be emphasized as a means of ensuring accountability as well as financial sustainability.
- Investments and O&M systems for new development areas/market and public places, and residential and other habitations
- Plan for safe collection, conveyance and treatment of household wastes
- Plan for Monitoring and Evaluation of implementation, and of achieving and sustaining 100% Sanitation (including use of community monitoring, etc.)
- Issues such as diminishing water resources, impact of climate change, use of low energy intensive on-site/decentralized sewage treatment technologies, distributed utilities, etc.
- Manpower issues such as adequate remuneration, hazardous nature of work, employment on transparent terms and conditions, use of modern and safe technology, provision of adequate safety equipment such as gloves, boots, masks, regular health check-ups, medical and accident insurance cover, etc.
- Plans for other locally significant aspects.

Some of the bigger cities may choose to prepare the plans on a regional/district or ward-wise basis. This may be a good way to mobilize stakeholders of the respective wards/regions and generate competition. However, at all times, it must be emphasized that such divisions are only limited to convenience in execution and monitoring, and sanitation must be a citywide achievement. Hence, the Task Force will have a special role in ensuring the integration of all the regional or functional components of the CSP as outlined above.

In order to promote wide ownership reflecting the collective and collaborative spirit of the sanitation endeavour, the CSP should be presented to the public for feedback at different stages of its development. Notwithstanding the inclusive and representative character of the City Sanitation Task Force, it is to the city's benefit if all or significant number of city stakeholders is able to contribute to the Plan. Holding of at least one, preferably two (draft and final stages) public meetings, needs to be considered by the Task Force.

10.5.2.7 Technical Options

Technology choice poses a major problem in Indian cities not only because of lack of information on what exists at present, but also because of the constraints of land, tenure, and low budgetary priority accorded to sanitation historically.

This leads to estimations of investments using conventional technologies that are mind-boggling and paralyze any incremental action.

The key issues about the technical options are:

- Technologies come with attendant capital and O&M costs, and management systems that may or may not be appropriate to a city's situation at a given time. Very often we can fall into the trap of planning systems that are difficult to finance, institutions are not ready and geared to operate and maintain them, and people are not ready or willing to adopt these and pay for service provision. Also, technology is linked to a whole set of environmental, behavioural and cultural parameters that need be taken into account. A holistic approach is required for technology choice.

- Approach to difficult existing situations (e.g. dense on-site systems draining into nallahs) is to think about upgradation and retrofitting options to make the systems sanitary and safe and also perform to their existing capacity.

- Technologies need to be incremental – for instance, even if sewers are ideal for dense settlements, they may not be feasible to immediately execute. In such cases, interim (e.g. on-site, or community septic tanks, improved septic tanks, Japanese Johkasou, or latrines if space is a constraint) systems may be planned with a view to later upgrade these to more sophisticated system (e.g. sewerage). Refer to Chapter 9 On-site Sanitation for details.

- Technologies and attendant systems for new development areas can be planned in advance. This results in early investments leading to cheaper and more sustainable systems in future.

- Technologies are only a means and not an end. They are to enable sanitary and safe confinement and disposal and hence, the approach to design must be keeping these ends in view.

- Technologies that promote recycle and reuse of treated sewage should be encouraged.

There is considerable information available on existing options as also the experience with some new systems and processes. These need to be reviewed by the Implementing Agency and where needed, specialist advice sought from state and national level agencies, and the private and community sectors. Exposure visits and training programmes will be required to take an informed decision. Finally, customers are at the heart of such systems – households and establishments must be consulted on expressing their preference after being made aware of the pros and cons of each of the systems under consideration. Technology choice again should address the citywide nature of the challenge – a mix of options must add up to addressing the issue completely, not just in bits.

Finally, technologies need to be planned for the full cycle of arrangements at the unit level, conveyance/transport, and final treatment and disposal into the environment. Any combination of systems that does not lead to the output of 100% safe collection, conveyance, treatment, and disposal will not serve the purpose of achieving 100% sanitation for the city.

Situation Analysis: Studies show that the bulk of decision-making and unit level investments are made by households and establishments – with more focus on sanitation arrangements, and less attention to collection, treatment and disposal. Public agencies are concerned with collection, treatment and disposal, but boundaries of roles and responsibilities are not clear.

In many if not most of the cases, public agencies are also unable to accord much attention to the public infrastructure and systems for collection, treatment, and disposal (e.g. sewerage systems, sewage treatment plants), or leave it for the households to resolve their problems (e.g. cleaning of septage). Thus, issues of O&M and sustainability need to be kept in view when planning for technology options.

10.5.2.8 Reaching the Un-served Populations and Urban Poor

Experiences from many Indian cities show that a differentiated approach is necessary to extend good quality sanitation services to the poor – the group that suffers the most in terms of adverse impacts on health and lost earnings.

Participatory approaches are needed to consult the poor settlements and involve them in the process of planning and management of sanitation arrangements. Many settlements may have the necessary conditions to support the provision of individual on-site sanitation arrangements (e.g. as tried out in some pockets in Ahmedabad, etc.) that are ideal, in many others, tenure and legal issues prevent provision of individual toilets and hence, community toilets (CTs) are the only way for immediate succour and access (e.g., as is the case with Mumbai, Pune, etc.). In some places, conventional and shallow sewers have also been tried out as alternative to on-site solutions in dense settlements. Examination of legal/tenurial, space and affordability issues in close consultation with communities becomes a key step in planning innovative means that are owned by users and will be sustainably managed by them.

NGOs can play an important role in mobilizing slum communities. Further, when community groups themselves take over the O&M of community facilities, then sustainable services become possible. This is also a way of reducing costs (compared to say, pay and use public toilets) and making services affordable to the poorest of families.

Another segment of population normally without sanitation is those who live in dispersed urban locations not being slums or in groups of houses that have legally not been notified as slums. Innovative approaches are required to extend services to these population groups too. It may be noted that public sanitation is for general public or floating populations, whereas CTs are those, where an identifiable core group of users exist, even if floating population may occasionally use these facilities.

The Implementing Agency will need to take stock of the legal and non-notified settlements in the city, and in partnership with NGOs and Community Based Organizations (CBOs), initiate a process of collaborative planning and delivery of services. Sanitation services also serve as an entry point for improved water supply, drainage improvements and community managed solid waste disposal systems – these areas should also be targeted while planning for sanitation is being undertaken.

At least 20% of the funds under the sanitation sector should be earmarked for the urban poor.

The issues of cross subsidization of the urban poor and their involvement in the collection of O&M charges should be addressed.

Finally and not least of all the obstacles, is the mind set of officers of ULBs and other citizens: bias and myths often hinder proper service provision to poor settlements. There must be a concerted effort to raise awareness amongst all stakeholders about the huge health and environmental costs that all have to bear if services are not comprehensively provided to all citizens.

Two steps are necessary to achieve this change in mind-sets: a) orientation programmes must be conducted for ULB functionaries; and b) setting up permanent systems in ULBs, complemented with agreements with NGOs and CBOs, to deliver services and monitor outcomes on an urgent basis to all poor households, as well as others, who are either un-served or have insanitary arrangements for defecation, collection or disposal.

10.5.2.9 O&M and Service Delivery Systems

Institutional systems for O&M are at the heart of any successful set of systems and procedures to achieve and sustain 100% sanitation. As outlined above, responsibilities for institutions are weakly defined and even if stipulated hardly followed properly.

Therefore, existing systems must be examined with the question: which agency or institution is responsible for operating and maintaining the system or a part thereof? If they do not discharge their responsibilities, what corrective action or recourse exists and who is responsible for this? For new investments similar questions need to be asked so that assets and services do not suffer from lack of proper O&M. A citywide perspective is necessary since O&M is required for all parts of the sanitation systems, whether it is excreta removal, or drainage or solid waste management. Assigning institutional responsibility also must go hand in hand with technology selection, design and implementation/creation of assets.

While sewerage systems have limited responsibility of households (from own property to nearest street connection), institutions responsible for the rest of the conveyance systems are faced with a number of personnel, finance and incentives related constraints. These need to be mapped and clearly addressed – even with little resources; innovations need to be made in the organization responsible (relevant ULB department or service provider unit) to seek immediate remedies while a more systematic planned set of steps to improve O&M may be implemented during the plan.

In most on-site systems, households are left to fend for themselves – often, there is no check on unhealthy and illegal practices such as draining wastes in to nallahs and drains. These also need to be brought under the remit of the respective public agency and properly dealt with. Septage clearance services are another area where quick action can be initiated and the necessary fees charged from households. In drainage and solid waste too, a number of steps can be initiated (some of these have been successfully tried out in solid waste management in many Indian cities) to ensure proper O&M and service delivery, in which consumer households also have a stake and roles built in.

Preparing O&M Protocol for each of the sanitation facilities in the city is a good step in this direction, and their adherence needs to be monitored by senior officers, elected representatives and community members.

O&M systems often suffer because customers do not recognize this as a service, and do not pay for the poor service levels. O&M is closely related to the financial sustainability of service provision, and hence, the Implementing Agency must take full stock of the financial implications of improving current and future service levels. These should lead to proposals to the City Task Force, as a part of the CSP, on how to recover or fund the costs of O&M.

Customer complaints and redressal systems is another major area needing attention. One of the important changes that need to be effective amongst the ULB, or service providing agency is to treat citizens as customers of services. Accordingly, complaints, redressal and feedback systems can be instituted for sustained improvements. Preparing proper customer records and taking structured feedback are ways already tried out in other sectors with satisfactory results in improving public services. Providing orientation and training programmes, implementing customer relationship systems, and linking O&M performance to personnel performance are ways to examine improved service delivery systems.

Finally, in many cases, households and communities may be in a better position to carry out O&M tasks or monitor performance thereof. This approach works specially when communities have incentives to work together and/or there are considerable externalities of a particular behaviour (individual actions affecting others easily).

Maintenance management of CTs, maintaining cleanliness in neighbourhoods, keeping drains and nallahs clean, street sweeping, etc., are examples where community groups can easily monitor the performance of service providers. In case of poorer neighbourhoods and slums, some of these tasks may be formally entrusted to local groups too.

10.5.2.10 Capacity Building and Training

The role of capacity building and training is crucial in achieving and sustaining 100% sanitation. Because of the historical neglect, the know-how of sanitation is limited to a minuscule group of personnel in ULBs/service providing agencies – even these skills run down over time due to little scope for application and sometimes the narrow nature of the specific job. Therefore, two broad kinds of interventions are necessary:

a) Orientation, building of skills and aptitude for carrying out different types of activities in respect of total sanitation

b) Designing and implementing working systems in ULBs or service providing agencies to provide the right kind of structures, linkages and organizational systems and environments that utilize the skills and perspectives imparted above.

The task of building capacities is huge – this is compounded by the generally low levels of synthesis and dissemination of existing knowledge and experiences of working with different kind of technologies, management regimes, organizational systems and processes and institutional relationships. Therefore, there is a dual agenda of consolidating and applying existing and new knowledge in a learning-by-doing framework, and building capacities thereon in an adaptive manner that is able to accommodate a range of personnel with different kind of backgrounds.

The National and State level Resource Organizations including NGOs, need to be brought in by the City Task Forces, to assist in this huge agenda that needs to be woven closely with the sanitation campaign, planning, implementation, and monitoring and evaluation. Similarly, experts need to be deployed early with assistance of the Union and State Governments, so that the knowledge development on technologies and management regimes is quickly made available for the city to adapt. The role of NGOs will be valuable in training and capacity building for participatory methods and consultation techniques to be used with the urban poor and un-served households.

Two strategies are worth considering in the capacity building agenda: a) bulk training for a range of municipal, NGO/CBO, private sector personnel – right from the start of the campaign in the city; b) Differentiated and specialized training on a demand-basis to personnel in and outside the government over the period of the Sanitation Plan implementation.

One of the common failures of training and capacity building is the lack of incentives and organizational environment to practice the learnt perspectives and skills. This highlights the need for the Task Force and implementing organizations to plan the training of their personnel in such a manner that their skills can be put to productive use.

Agencies from the private sector, public and NGO training and capacity building institutions must be involved in the campaign process to carry out the necessary assessments and help the Task Force plan and devise a strategy for Human Resource Development and capacity development through the implementation cycle, and institute appropriate practices within the institutional framework of the ULB and other stakeholders for the future.

10.5.2.11 Implementation, Management, Monitoring and Evaluation

10.5.2.11.1 Implementation Management

The task of implementation management can prove to be onerous if the planning stages are done in a hurry or are inadequate in taking account of ground reality (including current assets, finances, capacities and availability of suppliers and vendors, and other environmental conditions). While the Implementation Agency will be responsible for overall implementation, it is useful to think about plan implementation and delivery mechanisms for each of the components of the Plan.

The typical components indicate that there need to be either in-house resources deployed for these tasks (e.g. as in bigger ULBs) or private and NGO service providers need to be contracted or commissioned to carry out the implementation. The following types of skills and competencies are required in these implementation agents:

- Institutions/Organizations Development, and financial (capital and O&M costs, tariffs, ULB finances, etc.)
- Socio-economic and community management
- Urban planning
- Health and environmental linkages to sanitation

- Technical capacities to implement new assets and facilities and set up O&M systems for new development areas
- Monitoring and Evaluation (M&E)
- Capacities to address plans for other local aspects

Expert institutions, Consultants, NGOs, etc. who were involved in planning, may be considered for participating in and providing project management support to the Implementation Agency. In some of the larger cities, this may be an effective way to achieve efficient implementation of a large-scale sanitation plan for which the city may not have all expertise and management competencies within the ULB, or where many parallel activities are to be implemented leading to shortage of personnel during peak activities.

Contracts and their management are crucial in making sure that the implementation is without delays and adheres to appropriate quality standards. Two broad kinds of services are required: hardware related capacities that have to do with implementing physical works and software/process related capacities, e.g., social mobilization, institutional development, training, etc. Since the ULB may not have requisite capacities and systems to effectively deal with the challenges of contracting and supervision of contracts, innovations are needed: these include taking assistance from State level agencies in selection and procurement; appointing contractors and consultants on a cost-plus basis; lump-sum or unit-price contracts for other components and so on. Memoranda of Understanding (MoU) (e.g. with NGOs) to arrive at a common shared understanding of responsibilities and deliverables are another tool to address some of the components. Finally, training in contract management may be an area that core members of the Implementing Agency need to go through if, requisite capacities are deemed to be wanting.

The presence and guidance of the City Sanitation Task Force will be an assurance of quality procedures, fairness, and focus on deliverables. Supervision and M&E of implementation will provide other methods of mid-course correction.

10.5.2.11.2 Monitoring, Evaluation and Supervision of Progress

The City Sanitation Task Force and the Implementing Agency need to think about M&E of the implementation as an integral part of the CSP. The mechanisms to be used in monitoring implementation include:

- Administrative data from Implementing Agency Reports and from the implementing consultants, contractors
- Task Force field visits to different parts of the city
- NGOs working in different parts of the city, e.g. an NGO working in certain slum pockets may be able to monitor changes in the relevant settlements since they work there, visit and interact with people regularly. A Memorandum of Understanding or undertaking to provide additional expenses may be required from the ULB, whereas some NGOs, especially those working on health, may be collecting some of this data as a part of their own work;

- Community groups asked to provide structured feedback to the implementing agency and the task force on progress of implementation and the condition in their respective neighbourhoods
- Independent third party assessments
- Concurrent evaluations by a survey agency.

An important aspect of monitoring and evaluation is to make the findings and reports available to the public so that feedback and suggestions can be received from other stakeholders. Sharing key features in monthly task force meetings and press briefings are also another way of mobilizing city stakeholders and eliciting their cooperation.

10.5.3 Evaluation of 100% Sanitation Status

The mechanisms and systems used for M&E often determine the quality of assessments of results as well as to a large extent the responses of different stakeholders. The Ministry of Urban Development Rating of Cities lists M&E indicators in terms of output, process and outcome related parameters.

While the Task Force and Implementing Agency may use a combination of mechanisms suggested above for implementation, for evaluation of 100% Sanitation Milestone achievements, a number of tools can be considered:

- A mix of self-assessment by the city sanitation task force – based on implementation agency data, citizens' groups feedback, and primary field visits
- Independent report cards and evaluation missions commissioned by the City Task Force and/or mounted by the State Government
- Cross-city monitoring with participation of State level and other-city stakeholders
- Government of India rating of cities, service level benchmarks, monitoring missions and independent agencies

Experiences from other sectors shows that multi-stakeholder M&E systems, using simplified formats to assess objective indicators are likely to build a shared ownership, and economically produce reliable results. Therefore, the City Sanitation Task Force may consider publicizing, as a part of the initial awareness generation campaign, the key indicators that all stakeholders should monitor, and devise a simplified mechanism to collect data and report on.

Introduction of competitive reward schemes within cities are another way to improve the quality of monitoring and evaluation of 100% sanitation achievements.

10.5.4 Monitoring of 100% Sanitation Status

In order to ensure that after the city or parts thereof do not slip back after the achievement of the milestone, there need to be systems instituted to ensure that this is not a one-time achievement, rather a permanent change in behaviour, systems and practices.

Again, multiple stakeholders need to be involved in this process, while the ULB or the Task Force may take the lead in doing so. The mechanisms to institute sustenance of change include:

- ULB Roles in monitoring processes, outputs and outcomes: the ULB will need to assume leadership and institutionalize the means of monitoring the 100% sanitation status. This will be closely tied to new investments and O&M roles and responsibilities within the ULB divisions, but it is recommended that a unit separate from the above units is made responsible for the overall outcomes of the city's achievements and their sustenance. The ULB will also be able to do this more effectively if it involves other government agencies (Environment, Health related within and outside its own organization) NGOs, CBOs, the urban poor, etc.

- The role of Citizens' Groups in monitoring on a day-to-day basis is invaluable and should be mobilized especially for the protection of neighbourhoods, incremental improvements, as well as immediate reporting of any deviance that needs solutions. At the overall city level of course, the erstwhile monitoring of implementation will transform into adding the responsibilities related to sustained change at the ground level.

- The best method of sustaining change is to regularly collect formal data and informal information and feedback, and make it public so that there is pressure created equally on the public agencies, private service providers, as well as households and communities, to keep to sustained practices. Rewards again serve as triggers for sustenance and in many cases, also to make improvements that will earn credit to the city. As outlined in Section 10.5.5 below, there are a number of other indirect benefits that accrue to cities becoming 100% sanitized and making constant improvements.

10.5.5 City Reward Schemes

Cities can institute their own reward schemes to incentivise local stakeholders to participate in the process of improvements for reaching 100% sanitation. Rewards could be given following the national guidelines on an area basis. For example, the following could be units for rewards:

a) Municipal Wards
b) Colonies or Residents' Associations
c) Schools, colleges and other educational institutions
d) Market and Bazaar Committees
e) City-based institutions or localities, e.g., Railway stations, Bus Depot, Office Bhawans, etc.
f) Other locations and institutions that may be in the city.

The reward may contain a nominal amount of money for further upkeep and maintenance of sanitary systems, improvements in infrastructure targeted to better health and environment, as also special purposes like holding environment fairs, health camps, etc. A scroll of honour, public function to accord recognition, and rating of wards may also be considered as a part of rewards. While such rewards are being instituted, it must be emphasized that the responsibility of any group or locality is not over by just its own achievements. It must be a citywide enterprise and no one will be safe and benefit from a healthy life and environment unless everyone in the city and its surroundings adopts improved personal and community practices of 100% sanitation.

The leadership of municipal ward elected representatives, local community leaders, citizens' groups and community based organizations, will be crucial in achieving and sustaining 100% sanitized wards or localities. They must be mobilized to compete in a healthy manner in achieving sanitation. Therefore, the reward scheme should become important in local community civic affairs, politics, and valorize the local economy too.

10.5.6 Cities with Special Institutions and Characteristics

i) There may be cities that have special institutional arrangements: cities where ULBs are not in place or have responsibilities only for a part of the city (other parts coming under a cantonment or a development authority). In such cities, a multi-agency Task Force will need to be created that can plan, guide and monitor the 100% sanitation campaign. It will be crucial that no part of the city is left out and as convenient and efficient, the authorities implement similar measures in their respective jurisdictions.

ii) Cities where ULBs are only partially responsible for sanitation, other responsibilities are vested in parastatal agencies like PHED/PWD, Water Boards, etc. The City Sanitation Task Force must involve representatives from all agencies involved in sanitation. This will include all agencies responsible for household/unit level sanitation, sewerage, water supply, health and environment.

iii) Some cities have unique topographical, environmental features (e.g., hilly or coastal regions), and therefore may be vulnerable to natural phenomena like floods, landslides, earthquakes, etc. Specialist advice may be sought by such cities from relevant national and state level agencies, and private firms. Such specialists may be invited to become members in the City Sanitation Task Force, and contribute their specialist knowledge and advice to the process. In cities vulnerable to natural disasters, special measures for sanitation must be explicitly incorporated in their Disaster Preparedness and Mitigation Plan.

If such a plan does not exist, the Task Force must layout the steps to be taken for the city to cope with such disasters including:

a) Institutional roles and responsibilities for disaster preparedness

b) Incorporation of disaster preparedness in the design and O&M of sanitation arrangements and systems (at household/unit level, in transport and conveyance, and in sewage treatment / disposal)

c) Emergency measures and rehabilitation measures in the event of disasters

d) Building key points from above in public awareness generation campaigns.

10.6 COORDINATION BETWEEN CMP AND CSP

The essence of planning is coordination. Planning requires resolution of conflicting interests, allocation of available funds and other resources, inter and intra central and state government departmental cooperation, and establishment of priorities.

The City Master Plan (CMP) describes the vision for the city's future.

A comprehensive CMP guides development, conservation and capital improvement projects to improve the quality of life in the community. The plan must comply with the State's regulatory requirements, one of which is in review every 10 years.

Topics addressed in the CMP include the City's goals and objectives, land use plan, urban design, housing, infrastructure, parks, open space, transportation, economic development and preservation of historical monuments.

The CMP is constantly under revision as the needs of the community change and state or ULB requirements are incorporated into the document. Residents are welcome to share input on the CMP and are encouraged to get involved keeping in view of environmental and physical status of the city.

The planning period of CMP is a function of various developmental plans as stated above and should be fairly of a longer period for sustainability of other development plans.

In order to have sustainable CMP and other developmental plans, there is a need for inter and intra departmental coordination of central and state departments including parastatal agencies.

From the standpoint of the direction and overall needs of National Government, a CSP is one among several functional plans, such as those dealing with highways, natural resources, education, health, etc. CSP, therefore, should relate to, and not conflict with, other plans of the city.

It is essential that the city sanitation planning be included in the overall plan of the jurisdiction that will ultimately implement it. In this way, the agency responsible for sanitation services will be able to compete effectively for funds, personnel, and other resources and facilities.

10.7 CITY SANITATION PLAN OUTLINE

The basic planning model can be translated into an outline for reporting the established plan. Such a format communicates the logic inherent in the planning procedure. Planning initiative and innovation are desirable.

However, each civic body is expected to formulate its own systematic outline and report, taking into account its particular needs as indicated in the sample format, described later in this chapter, for the preparation of the CSP.

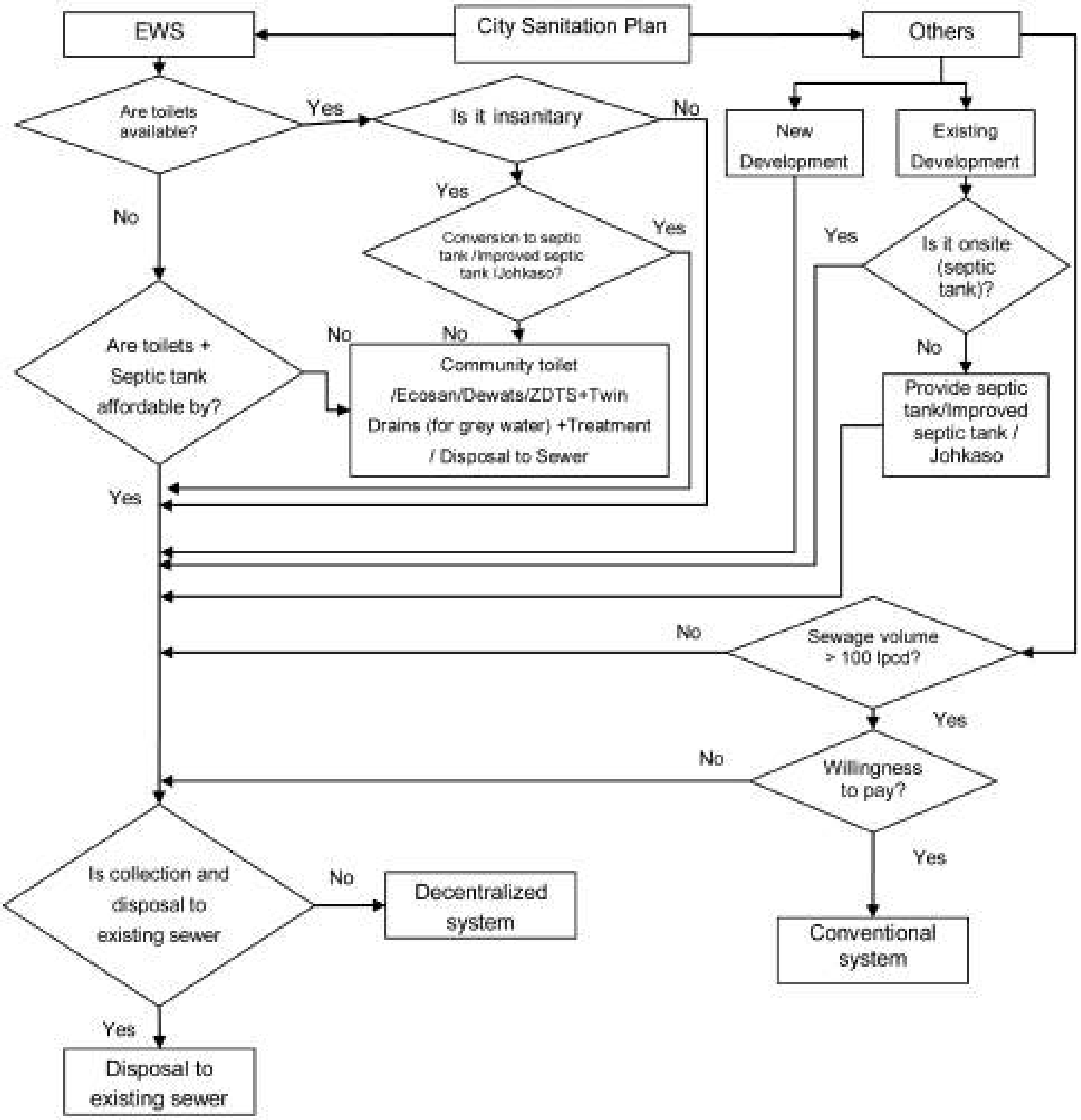
10.8 ALGORITHM FOR DECISION MAKING ON SEWERAGE OPTIONS

The algorithm is presented in Figure 10.2 overleaf.

10.9 REPORTING

The report shall be simple, easy to read as a running text including all calculations, charts and tabular columns in the Annexure for easy understanding of readers. This will help in a quick grasp for the management decisions.

The contents can be on the following sequence.



EWS: Economically Weaker Section

Bahao toilets are the toilets directly connected to stormwater drain

Non Conventional sewers: simplified sewers, settled sewers and twin drains

Figure 10.2 Decision Tree: Selecting the technical option (On-site, Decentralized or Conventional)

1) Introduction and Need – not to exceed one page
2) Executive summary – not to exceed one page
3) Physical setting of the study area
4) Existing sanitation arrangements
5) Socio-economic setting of the study area
6) Health statistics of the study area
7) Financial position of the local authority
8) Human resources of the local body
9) Recommended sanitation Plan
10) Ways and means to strengthen the resources of the local body
11) Optional models for project delivery mechanisms.

The sample format for preparing the city sanitation plan is mentioned in Table 10.1 (overleaf).

Table 10.1 Sample format for preparing City Sanitation Plan

Elements of the Report
Section I Introduction Purposes of the plan. **Section II Executive Summary** (Note: This section should be written last and may come at the beginning of the report) **Section III Background of the Planning Area** 1. Jurisdictions a. National b. State c. City/Town (Civic Authorities) d. Location Map e. Population (size and densities) f. Housing (types and locations) g. Land uses (residential, commercial, industrial, agricultural, extractive, recreational, and other relevant land uses) h. Transportation corridors **Section IV Existing Sanitation Conditions** 1. Arrange data according to specific needs of the planning agency. As far as possible all the information related to sanitation has to be collected. 2. Describe and analyse all existing conditions affecting management of sanitation. a. Storage and collection of sewage b. Quantities of sewage generated, collected, treated, reused and disposed of c. Reuse and disposal practices d. General management practices (e.g., utilization of manpower and equipment) e. Public awareness and knowledge about sanitation problems and willingness to pay for better services f. Expenditures for sanitation management **Section V Future Conditions and Problem Definition** 1. Relevancy for the future (from the analysis of the data of existing conditions accumulated in sections III and IV, determine which conditions will have a bearing on the future).

Continued

Elements of the Report
2. Future problems defined a. Types b. Locations c. Extent d. Persistence e. Others 3. All existing conditions and problems bearing upon the future should be forecast at this stage. **Section VI Objectives** Objectives should be clearly stated and based upon need to solve problems defined earlier. Civic authority might specify any of the following objectives to solve its sanitation problems: 1. Acceptable methods for storage 2. Acceptable methods for collection of sewage and septage 3. Acceptable sewage treatment practices 4. Acceptable sludge treatment practices 5. Acceptable method of recycle and reuse 6. Acceptable methods of disposal 7. Development of sanitation management organizational structure 8. Development of better trained personnel (operating and management levels) 9. Better informed public regarding sanitation problems and service requirements 10. Provision of sufficient financial support for sanitation 11. Others **Section VII Recommendations for Solution (The Plan)** 1. This section should specify what the civic authority intends to accomplish in order to solve its sanitation problems. It should include designation of the following: a. System improvement b. Timing and priorities of intended action (consider short and long-term objectives) c. Who should act (i.e. agency, department) d. Estimated costs e. Problems that will be solved f. Others 2. It is suggested that the following aspects should be considered in the intended action plan. Proposals for this action should be accompanied by procedures for accomplishment and a schedule of initiation of this action. a. Establishment of sanitation operating departments and identifying its jurisdictions b. Recruitment, selection and hiring of operating personnel c. Human resources development programme

Continued

Elements of the Report
d. Technical assistance to operating units e. Provisions for inspection and enforcement f. Licensing of facilities g. Framing legislation, amendments to rules and regulations h. Development of budgeting procedures, financing, cost-effectiveness, special charge features and other operating management features i. Public information, education and communication programme/system j. Others
Section VIII Implementation (occurs outside the plan document but is guided by it)
Appendices This section of the report should include supporting materials and information used to develop the analyses, objectives, and plan. Content of this section might include: a. Charts b. Additional tables c. References d. Legislation and regulations e. Definition of terms f. Methodologies of research and analyses g. Others
Section IX Monitoring and Performance Evaluation of the Programme This section of the report should include monitoring of various activities of sanitation services and also evaluation of the performance of all the related activities with reference to the objectives/targets envisaged, once the programme is implemented.

Source: MoUD, 2008

The text of the CSP for a city should explain in detail all the above elements that are to be contained in the plan report and conforming to the above outline.

सत्यमेव जयते

Government of India

Ministry of Urban Development

Nirman Bhawan, New Delhi 110 011, India

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MANUAL ON SEWERAGE AND SEWAGE TREATMENT SYSTEMS

PART A: ENGINEERING - APPENDIX

THIRD EDITION - REVISED AND UPDATED

MINISTRY OF URBAN DEVELOPMENT, NEW DELHI

http://moud.gov.in

CENTRAL PUBLIC HEALTH AND ENVIRONMENTAL ENGINEERING ORGANIZATION

IN COLLABORATION WITH

JAPAN INTERNATIONAL COOPERATION AGENCY

NOVEMBER 2013

In keeping with the advancements in this sector, updates as and when found necessary will be hosted in the Ministry website: http://moud.gov.in/ and the reader is advised to refer to these also.

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APPENDICES

ABBREVIATIONS

AL	Aluminium	LT	Low Tension
ASP	Activated Sludge Process	M	Meter
Avg	Average	MBR	Membrane Bio Reactor
BIS	Bureau of Indian Standards	MBBR	Moving Bed Bio Reactor
BOD	Biochemical Oxygen Demand	ML	Million litres
BOD -T	Biochemical Oxygen Demand - Total	MLD	Million Litres Daily
BOD-S	Biochemical Oxygen Demand - Soluble	MLSS	Mixed Liquor Suspended Solids
CPCL	Chennai Petroleum Corporation Ltd	MPN	Most Probable Number
COD	Chemical Oxygen Demand	ORP	Oxidation Reduction Potential
COD -T	Chemical Oxygen Demand - Total	PAC	Poly Aluminium Chloride
COD-S	Chemical Oxygen Demand - Soluble	S	Slope
DEWAT	Decentralised Wastewater Treatment	SBR	Sequencing Batch Reactor
D O	Dissolved Oxygen	SD	Standard Deviation
E coli	Escherichia Coliform	ST	Sewage Treatment
Fe	Iron	STP	Sewage Treatment Plant
HRT	Hydraulic Retention Time	SVI	Sludge Volume Index
IIT	Indian Institute of Technology	TSS	Total Suspended Solids
KW	Kilowatt	V Valley	Vrishabhavathi valley

APPENDIX - A

Symbols

(Retained as in 1993 edition of this manual and updated)

deg C	degrees centigrade	kwh	kilowatt hour
amp	ampere	l	litre
BOD_5	5 days BOD	lpm	litre per minute
cc	cubic centimetre	lps	litre per second
cm	centimetre	m	metre
CM	cement mortar	m^2	square metre
cumecs	cubic metre per second	m^3	cubic metre
d	day	meq	mill equivalent
eq or eqn	equation	min	minute
F/M	food to micro-organisms ratio	ml	millilitre
GI	galvanised iron	mld	million litres per day
gm	gram	mm	millimetre
h, hr	hour	mps	metre per second
ha	hectare	MWL	maximum water level
HSV	hydraulic subsidence value	ORP	oxidation reduction potential
IS	Indian standard	Pr	primary
kcal	kilo calorie	PSS	percent soluble sodium
kg	kilogram	rph	revolution per hour
kgf	kilogram force	rpm	revolution per minute
kL	kilo litre	S.S.T	secondary sedimentation tank
kLD	kilo litre per day	T	tonnes
km	kilometre	TF	trickling filter
kw	kilowatt	w	watt

APPENDIX.AA-List of Indian Standards Relating to Sewerage and Sewage Treatment

Also refer to (http://www.bis.org.in/) for further Standards under a particular subhead.

No	IS code	Indian Standard
1	SP 7	National Building Code of India 2005 (group 1 to 5)
2	SP 35	Handbook on Water Supply and Drainage with special emphasis on plumbing
3	IS 15883 : Part 1	Construction project management - Guidelines : Part 1 General
4	IS 269	Specification for ordinary and low heat portland cement
5	IS: 383	Specification for coarse and fine aggregates from natural sources for concrete
6	IS: 432	Specification for mild steel and medium tensile steel bars and hard drawn steel wire for concrete reinforcement
7	IS: 1786	Specification for high strength deformed steel bars and wires for concrete reinforcement
8	IS 456	Code of practice for plan and reinforced concrete
9	IS 457	Code of practice for general construction of plain and reinforced concrete for dams and other massive structures
10	IS 460 : Part 1	Specification for test sieves part I wire cloth test sieves
11	IS 460 : Part 2	Specification test sieves part II Perforated plate test sieves
12	IS 460 : Part 3	Specification for test sieves : part III methods of examination of apertures of test sieves
13	IS: 2116	Specification for sand for masonry mortars
14	IS: 2250	Code of practice for preparation and use of masonry mortars
15	IS 962	Code of practice for architectural and building drawings
16	IS 1200 : Part 16	Code of practice for architectural and building drawings
17	IS 1200 : Part 19	Method of measurement of building and civil engineering works Part 19: water supply, plumbing and drains

No	IS code	Indian Standard
18	IS 1343	Code of practice for prestressed concrete
19	IS 1554 : Part 1	PVC insulated (heavy duty) electric cables: Part 1 For working voltage up to and including 1100 volts
20	IS 1554 : Part 2	PVC insulated (heavy duty) electric cables: Part 2 For working voltage form 3.3 KV up to and including 11 KV.
21	IS 1607	Methods for test sieving
22	IS 2064	Code of practice for selections, installation and maintenance of sanitary appliances
23	IS 2212	Code of practice for brickworks
24	IS 2431	Steel wheel barrows (single wheel type) Specification for steel wheel barrows (single
25	IS 3370 : Part 1	Code of practice for concrete structures for the storage of liquids; part 1 General requirements
26	IS 3370 : Part 2	Code of practice for concrete structures for the storage of liquids: Part II reinforced concrete structures.
27	IS 3370 : Part 3	Code of practice for concrete structures for the storage of liquids: Part II Prestressed Concretes Structures.
28	IS 3370 : Part 4	Code of practices of concrete structures for the storage of liquids: Part IV Design tables.
29	IS 3764	Safety code for excavation work.
30	IS 3861	Method of measurement of plinth, carpet and rentable area of buildings
31	IS 4081	Safety code for blasting and related drilling operations.
32	IS 4682 : Part 1	Code of practice of for lining of vessels and equipment for chemical processes Part I Rubber lining.
33	IS 1200 : Part 16	Part 2 Glass Enamel lining
34	IS 4682 : Part 3	Part 3 Lead lining
35	IS 4682 : Part 4	Part 4 Lining with Sheet Thermoplastics

No	IS code	Indian Standard
36	IS 4682 : Part 5	Part 5 Epoxide resin lining
37	IS 4682 : Part 6	Part 6 Phenolic resin lining
38	IS 4682 : Part 7	Part 7 Corrosion and heat resistant metals
39	IS 4682 : Part 8	Part 8 Precious metal
40	IS 4682 : Part 9	Part 9 Titanium
41	IS 4682 : Part 10	Part 10 Brick and tile
42	IS 4854 : Part 1	Glossary of terms for valves and their parts: Part 1 Screw down stop check and gate valve and their parts.
43	IS 4854 : Part 2	Glossary of terms for valves and their parts: Part 2 Plug valves and cocks and their parts.
44	IS 4854 : Part 3	Glossary of terms for valves and their parts: Part 3 Butterfly valves
45	IS 4885	Specification for sewer bricks
46	IS 4926	Ready mixed concrete - Code of Practice
47	IS 5421	Glossary of terms relating to test sieves and test sieving
48	IS 5742 : Part 1	Terms and symbols for sieve bottoms: Part 1 Woven and welded wire screens
49	IS 5742 : Part 2	Terms and symbols for sieve bottoms: Part II Perforated Plates.
50	IS 7331	Code of practice for inspection and maintenance of cross-drainage works
51	IS 7357	Code of practice for structural design of surge tanks
52	IS 7784 : Part 1	Code of practice for design and of cross drainage work : Part 1 General features
53	IS 7784 : Part 2 : Sec 1	Code of practice for design of cross drainage works: Part 2 Specific requirements section 1 Aqueducts.
54	IS 7784 : Part 2 : Sec 2	Code of practice for design of cross drainage works: Part 2 specific requirements section 2 Super passages.
55	IS 7784 : Part 2 : Sec 3	Code of practice for design of cross drainage works : Part 2 Specific requirements section 3 Canal syphons

No	IS code	Indian Standard
56	IS 7784 : Part 2 : Sec 4	Code of practice for design of cross drainage works : Part 2 Specific requirements section 4 Level Crossings
57	IS 7784 : Part 2 : Sec 5	Code of practice for design of cross drainage works : Part 2 Specific requirements section 5 Syphon aqueducts
58	IS 7861 : Part 1	Code of practice for extreme weather concreting part 1 Recommended practice for hot weather concreting
59	IS 7861 : Part 2	Code of practice for extreme weather concerting: Part II Recommended practice for cold weather concreting.
60	IS 7969	Safety code for handling and storage of building materials
61	IS 9913	Code of practice for construction of cross drainage works
62	IS 10262	Recommended guidelines for concrete mix design proportioning
63	IS 10483	Code for designating perforation of industrial plate sieves (Identical with ISO/DIS 7806)
64	IS 11389	Methods of test for performance of concrete vibrators; immersion type
65	IS 11993	Code of practice tor use of screed board concrete vibrators.
66	IS 12119	General requirements for pan mixers for concrete
67	IS 12440	Specification for precast concrete stone masonry blocks
68	IS: 1077	Specification for common burnt clay building bricks
69	IS: 3495	Methods of tests of burnt clay building bricks
70	IS 12468	General requirements for vibrators for mass concreting : immersion type
71	IS 12592	Precast concrete manhole covers and frames
72	IS 5329	Code of practice for sanitary pipe work above ground for buildings
		Pipes and Fittings
73	IS 782	Specification for Caulking lead
74	IS 5382	Specification for Rubber sealing rings for gas mains, water mains and sewers
75	IS 6837	Specification for Three wheel type pipe cutter

No	IS code	Indian Standard
76	IS 6843	Technical supply conditions for pipe cutters
77	IS 6881	Specification for Link type pipe cutters
78	IS 10883	Specification for Single wheel type pipe cutters
79	IS 11906	Recommendations for cement-mortar lining for cast-iron, mild steel and ductile-iron pipes and fittings for transportation of water
80	IS 12820	Dimensional requirements for rubber gaskets for mechanical joints and bush joints for use with cast iron pipes water, gas and sewage. Dimensional Requirements of Rubber Gaskets for Mechanical Joints and Push-on Joints for Use with Cast Iron Pipes and Fit
		Concrete
81	IS 458	Precast Concrete Pipes (with and without reinforcement)
82	IS 783	Code of practice for laying of concrete pipes
83	IS 784	Prestressed concrete pipes (including fittings)
84	IS 1916	Steel cylinder pipe with concrete lining and coating
85	IS 3597	Methods of test for concrete pipes
86	IS 7322	Specials for steel cylinder reinforced concrete pipes
87	IS: 516	Methods of tests for strength of concrete
		Asbestos Cement
88	IS 6908	Asbestos cement pipes and fittings for sewerage and drainage
89	IS 5531	Cast iron specials for asbestos cement pressure pipes for water, Gas and Sewage
90	IS 5913	Methods of test for asbestos cement products
91	IS 6530	Code of practice for laying of asbestos cement pressure pipes
92	IS 8794	Cast iron detachable joints for use with asbestos cement pressure pipe
93	IS 9627	Specification for asbestos cement pressure pipes (light duty)

No	IS code	Indian Standard
94	IS 10232	Dimensional requirement for rubber sealing rings for cast iron detachable joints in asbestos cement piping
95	IS 10299	Cast Iron saddle pieces for service connection from asbestos cement pressure pipes
96	IS 11769 : Part 1	Guidelines for safe use of products containing asbestos: Part 1 Asbestos cement products
97	IS 12987	Guidelines for safe use of products containing asbestos- Cast Iron detachable joints for use with asbestos cement pressure pipes (light duty)
98	IS 12988	Dimensional requirements for rubber sealing rings for CID joints in asbestos cement piping for light duty AC pipes
		Cast Iron pipes
99	IS 1729	Cast iron rain water pipes and fittings. Cast iron/Ductile iron Drainage Pipes and Pipe Fittings for over ground non pressure pipeline socket and spigot series.
100	IS 1536	Centrifugally cast (spun) iron pressure pipe for water, gas and sewage
101	IS 1537	vertically cast iron pressure pipes for water, gas and sewage
102	IS 1538	Cast iron Fittings for pressure pipes for water, gas and sewerage
103	IS 1879	Malleable cast iron pipe fittings
104	IS 3114	Code for practice for laying of cast iron pipes
105	IS 3989	Centrifugally cast (spun) iron spigot and socket soil, waste and ventilating pipes, fittings and accessories
106	IS 12592	Horizontally cast iron double flanged pipes for water, gas and sewage
107	IS 8329	Centrifugally cast (spun) ductile iron pressure pipes for water gas and sewage
108	IS 11606	Methods of sampling of cast iron pipes and fittings.
109	IS 12288	Code of practice for use and laying of ductile iron pipes
		Steel

No	IS code	Indian Standard
110	IS 3589	Electrically welded steel pipes for water, gas and sewage (150 to 2000 mm nominal size)
111	IS 5504	Specification for Spiral welded pipes
112	IS 5822	Code of practice for laying of electrically welded steel pipes for water supply
113	IS 6392	Steel pipe flanges
114	IS 8062 : Part 2	Code of Practice for Cathodic Protection of Steel Structures - Part II : Underground Pipelines
		Stoneware
115	IS 651	Glazed stoneware pipe and fittings - Specification
116	IS 3006	Chemical resistant glazed stoneware pipes and fitting
117	IS 4127	Code of practice for laying of glazed stoneware pipes
		Plastic (Please also refer no 357 and 358)
118	IS 4984	High density polyethylene pipes for potable water supplies, sewage and industrial effluents
119	IS 7634 : Part 1	Code of practice for plastic pipe work for potable water supplies-Choice of materials and general recommendations
120	IS 7634 : Part 2	Code of practice for plastics pipe work for potable water supplies-Laying and jointing of polyethylene (PE) pipes.
121	IS 7634 : Part 3	Plastics Pipe Selection, Handling, Storage and Installation for Potable Water Supplies - Code of Practice : Part 3 - Laying and Jointing of UPVC Pipes
122	IS 8008 : Part 1	Specification for injection moulded HDPE fittings for potable water supplies: Part 1 General requirements
123	IS 8008 : Part 2	Specification for injection moulded HDPE fittings for potable water supplies: Part 2 Specific requirements for 90 degrees bends.
124	IS 8008 : Part 3	Specification for injection moulded HDPE fittings for potable water supplies: Part 3 Specific requirements for 90 degrees tees.

No	IS code	Indian Standard
125	IS 8008 : Part 4	Specification for injection moulded HDPE fittings for potable water supplies: Part 4 Specific requirements for reducers.
126	IS 8008 : Part 5	Specification for injection moulded HDPE fittings for potable water supplies: Part 5 Specific requirements for ferrule reducers.
127	IS 8008 : Part 6	Specification for injection moulded HDPE fittings for potable water supplies: Part 6 Specific requirements for pipe ends.
128	IS 8008 : Part 7	Specification for injection moulded HDPE fittings for potable water supplies: Part 7 Specific requirements for sandwich flanges
129	IS 8008 : Part 8	Injection Moulded/Machined High Density Polyethylene (HDPE) Fittings for Potable Water Supplies - : Part 8 Specific Requirements for Reducing Tees
130	IS 8008 : Part 9	Injection Moulded/Machined High Density Polyethylene (HDPE) Fittings for Potable Water Supplies - : Part 9 Specific Requirements for Ends Caps
131	IS 8360 : Part 1	Fabricated high density polyethylene (HDPE) fittings for potable water supplies: Part 1 General requirements
132	IS 8360 : Part 2	Specification for Fabricated high density polyethylene (HDPE) fittings for potable water supplies: Part 2 Specific requirements for 90 degree tees.
133	IS 8360 : Part 3	Specification for Fabricated high density polyethylene (HDPE) fittings for potable water supplies: Part 3 Specific requirements for 90 degree bends.
134	IS 12709	Specification to glass fibre reinforced plastics (GRP) pipes for use for water supply
135	IS 14402	GRP pipes joints and fittings for use sewerage, industrial waste and water (other than potable)
136	IS 15328	Unplasticized Non-Pressure Polyvinyl Chloride (PVC -U) Pipes for use in Underground Drainage and Sewerage Systems
137	IS: 9271	Unplasticized Polyvinyl Chloride (UPVC) Single Wall Corrugated Pipes for Drainage
		Pitch Impregnated Fibre
138	IS 11925	Specification for pitch-impregnated fibre pipes and fittings for drainage purposes
		Public Health & Sanitation

No	IS code	Indian Standard
		Waste water handling equipment
139	IS 5600	Pumps - Sewage and Drainage - Specification
140	IS 6279	Specification for Equipment for grit removal devices
141	IS 6280	Specification for Sewage screens
142	IS 7232	Method for imhoff cone test.
143	IS 8413 : Part 1	Requirements for biological treatment equipment part 1 trickling filters
144	IS 8413 : Part 2	Requirements for biological treatment equipment part 2 activated sludge process and its modifications
145	IS 9110	Specification for Hand operated augers for cleaning water closets, pipes and sewers.
146	IS 9213	Specification for BOD Bottle
147	IS 10037 : Part 1	Requirements for sludge dewatering equipment Part 1 Sludge drying beds-sand, gravel and under drains.
148	IS 10037 : Part 2	Requirements for sludge dewatering equipment Part 2 vacuum filtration equipment
149	IS 10037 : Part 3	Requirements for sludge dewatering equipment Part 3 Centrifugal equipment (Solid bowl type)
150	IS 10261	Requirements for settling tank (clarifier equipment) for waste wafer
151	IS 10552	Specification for buckets to be used in power driven buckets type sewer cleaning machine
152	IS 10553 : Part 1	Requirements for chlorination equipment: Part I General guidelines for chlorination plants including handling, storage and safety of chlorine cylinders and drums
153	IS 10553 : Part 2	Requirements for chlorination equipment: Part 2 Vacuum feed type chlorinators
154	IS 10553 : Part 4	Requirements for chlorination equipment: Part 4 Gravity feed type gaseous chlorinators

No	IS code	Indian Standard
155	IS 10553 : Part 5	Requirements of chlorination equipment: Part 5 Bleaching powder solution feeder displacement type chlorinator
156	IS 10595	Requirements for power driven bucket-type sewer cleaning machine
157	IS 11117	Requirements for power driven rodding machine for sewers.
158	IS 11387	Requirements for high pressure jetting machine for sewer cleaning.
159	IS 11397	Specification for attachment tools for power driven rodding machine for sewers.
		Code of Practice
160	IS 1172	Code of basic requirements for water supply, drainage and sanitation
161	IS 2527	Code of practice for fixing rainwater gutters and downpipes for roof drainage
162	IS 1742	Code of practice for Building drainage
163	IS 2470 : Part 1	Code of practice for installation of septic tanks; Part 1 Design, criteria and construction
164	IS 2470 : Part 2	Code of practice for installation of septic tanks : Part 2 secondary treatment and disposal of septic tank effluent
165	IS 9872	Precast concrete septic tanks
166	IS 4111 : Part 1	Code of practice for ancillary structures in sewerage system -Manholes
167	IS 4111 : Part 2	Code of practice for ancillary structures in sewerage system -Flushing tanks
168	IS 4111 : Part 3	Code of practice for ancillary structures in sewerage system Inverted syphon
169	IS 4111 : Part 4	Code of practice for ancillary structures in sewerage system-Pumping stations and pumping mains (rising main).
170	IS 4111 : Part 5	Code of practice for ancillary structures in sewerage system-Tidal outfalls
171	IS 5455	Specification for cast-iron steps for manholes
172	IS 210	Specification for grey iron castings

No	IS code	Indian Standard
173	IS 5329	Code of practice for sanitary pipe work above ground for buildings
174	IS 5611	Code of practice for waste stabilization ponds (facultative type)
175	IS 6295	Code of practice for water supply and drainage in high attitudes and/or sub-zero temperature regions (first revision)
176	IS 6924	Code of practice for the construction of refuse chutes in multi-storeyed buildings
177	IS 7740	Code of practice for construction and maintenance of road gullies
178	IS 9872	Specifications for precast concrete septic tanks.
179	IS 12251	Code of practice for drainage of building basements
180	IS 12314	Code of practice for sanitation with leaching pits for rural communities.
181	IS 11972	Code of practice for safety precautions to be taken when entering a sewerage system
		Solid wastes
		Code of practice
182	S 10447	Guidelines for utilization and disposal of solid waste from integrated steel plants.
183	IS 12647	Guidelines for collection equipments. Solid Waste Management System - Collection Equipment
184	IS 12662 : Part 1	Guidelines for use of vehicles for collection of municipal solid wastes : Part 1 Selection of vehicles
185	IS 12662 : Part 2	Vehicles for Collection of Municipal Solid Wastes - Part 2 : Guidelines for Maintenance
		Methods of testing
186	IS 9234	Methods for preparation of solid waste sample for chemical and microbiological analysis.
187	IS 9235	Methods for physical analysis and determination of moisture in solid wastes (excluding industrial wastes)
188	IS 10158	Method of analysis of solid wastes (Excluding industrial wastes)

No	IS code	Indian Standard
		Glossary of Terms-Water Pollution
189	IS 7022 : Part 1	Glossary of terms relating to water, sewage and industrial effluent part 1
190	IS 7022 : Part 2	Glossary of terms relating to water, sewage and industrial effluent part 2
191	IS 10446	Glossary of terms for water supply and sanitation.
		Methods of sampling and analysis- Sewage and industrial effluents
192	IS 6582	Bio-assay methods for evaluating acute toxicity of industrial effluents and waste waters.
193	IS 6582 : Part 2	Bio-assay methods for evaluating acute toxicity of industrial effluents and wastewaters : Part 2 Using Toxicity Factor to Zebra Fish
		Methods of sampling and analysis- wastewater
194	IS 1622	Methods of sampling and microbiological examination of water
195	IS 3025 : Part 1	Methods of sampling and test (physical and chemical) for water and wastewater-Sampling
196	IS 3025 : Part 2	Methods of Sampling and Test (Physical and Chemical) for water and Wastewater - Part 2 : Determination of 33 Elements by Inductively Coupled Plasma Atomic Emission Spectroscopy
197	IS 3025 : Part 3	Methods of Sampling and Test (Physical and Chemical) for water and Wastewater - Part 3 : Precision and Accuracy
198	IS 3025 : Part 4	Methods of Sampling and Test (Physical and Chemical) for water and Wastewater - Part 4 : Colour
199	IS 3025 : Part 5	Methods of sampling and test (physical and chemical) for water and wastewater-Odour
200	IS 3025 : Part 6	Methods of sampling and test (physical and chemical) for water and wastewater-Odour threshold
201	IS 3025 : Part 7	Methods of sampling and test (physical and chemical) for water and wastewater-Taste threshold
202	IS 3025 : Part 8	Methods of sampling and test (physical and chemical) for water and wastewater-Taste rating
203	IS 3025 : Part 9	Methods of sampling and test (physical and chemical) for water and wastewater-Temperature
204	IS 3025 : Part 10	Methods of sampling and test (physical and chemical) for water and wastewater-Turbidity

No	IS code	Indian Standard
205	IS 3025 : Part 11	Methods of sampling and test (physical and chemical) for water and wastewater-PH Value
206	IS 3025 : Part 14	Methods of sampling and test (physical and chemical) for water and wastewater-Specific conductance (Wheatstone bridge, conductance cell)
207	IS 3025 : Part 15	Methods of sampling and test (physical and chemical) for water and wastewater-Total residue (total solids- dissolved and suspended)
208	IS 3025 : Part 16	Methods of sampling and test (physical and chemical) for water and wastewater-Filterable residue (Total Dissolved solids)
209	IS 3025 : Part 17	Methods of sampling and test (physical and chemical) for water and wastewater-Non-filterable residue (total suspended solid)
210	IS 3025 : Part 18	Methods of sampling and test (physical and chemical) for water and wastewater-Volatile and fixed residue (total filterable and non filterable)
211	IS 3025 : Part 19	Methods of sampling and test (physical and chemical) for water and wastewater-Settle able matter
212	IS 3025 : Part 20	Methods of sampling and test (physical and chemical) for water and wastewater-Volatile and fixed residue (total filterable and non filterable)
213	IS 3025 : Part 21	Methods of sampling and test (physical and chemical) for water and wastewater-Total hardness
214	IS 3025 : Part 22	Methods of sampling and test (physical and chemical) for water and wastewater-Acidity
215	IS 3025 : Part 23	Methods of sampling and test (physical and chemical) for water and wastewater-Alkalinity
216	IS 3025 : Part 24	Methods of sampling and test (physical and chemical) for water and wastewater-Sulphates
217	IS 3025 : Part 25	Methods of sampling and test (physical and chemical) for water and wastewater-Chlorine, demand
218	IS 3025 : Part 26	Methods of sampling and test (physical and chemical) for water and wastewater-Chlorine, residual
219	IS 3025 : Part 27	Methods of sampling and test (physical and chemical) for water and wastewater-Cyanide
220	IS 3025 : Part 28	Methods of sampling and test (physical and chemical) for water and wastewater-Sulphite
221	IS 3025 : Part 29	Methods of sampling and test (physical and chemical) for water and wastewater-Sulphide

No	IS code	Indian Standard
222	IS 3025 : Part 30	Methods of sampling and test (physical and chemical) for water and wastewater-Bromide
223	IS 3025 : Part 31	Methods of sampling and test (physical and chemical) for water and wastewater-Phosphorous
224	IS 3025 : Part 32	Methods of sampling and test (physical and chemical) for water and wastewater-Chloride
225	IS 3025 : Part 33	Methods of sampling and test (physical and chemical) for water and wastewater-Iodide
226	IS 3025 : Part 34	Methods of sampling and test (physical and chemical) for water and wastewater-Nitrogen
227	IS 3025 : Part 35	Methods of sampling and test (physical and chemical) for water and wastewater-Silica
228	IS 3025 : Part 36	Methods of sampling and test (physical and chemical) for water and wastewater-Ozone, residual
229	IS 3025 : Part 37	Methods of sampling and test (physical and chemical) for water and wastewater-Arsenic
230	IS 3025 : Part 38	Methods of sampling and test (physical and chemical) for water and wastewater-Dissolved oxygen
231	IS 3550	Methods of test for routine control for water used in industry.
		Treatment and Disposal of industries effluents
232	IS 7967	Criteria for controlling pollution of marine coastal areas.
233	IS 8032	Guide for treatment and disposal of distillery effluents.
234	IS 8073	Guide for treatment and disposal of steel plant effluents
235	IS 8682	Guide for treatment of effluents of dairy industry
236	IS 9427	Code of practice for operation and maintenance of deionizing columns
237	IS 9508	Guide for treatment and disposal of effluents of cotton and synthetic textile industry.
238	IS 9509	Guide for treatment and disposal of effluents of viscose rayon industry

No	IS code	Indian Standard
239	IS 9841	Guide for treatment and disposal of effluents of fertilizer industry.
240	IS 10044	Guide for treatment and disposal of effluents of petroleum refining industry.
241	IS 10495	Guide for treatment and disposal of effluents of wool processing industry
		Chemical hazards
		General
242	IS 1446	Classification of dangerous goods.
243	IS 4155	Glossary of terms relating to chemical and radiation hazards and hazardous chemicals
		Code of Safety
244	IS 4209	Code of safety in Chemical laboratories
245	IS 4262	Sulphuric acid- Code of safety
246	IS 4263	Code of safety for Chlorine
247	IS 4264	Code of safety for Caustic soda
248	IS 4312	Code of safety for Lead and its compounds
249	IS 4544	Ammonia- Code of safety
250	IS 4560	Code of safety for Nitric Acid
251	IS 4644	Code of safety for Benzene, toluene and xylene
252	IS 4906	Code of safety for Radiochemical laboratory
253	IS 5184	Code of safety for Hydrofluoric Acid
254	IS 5208	Code of safety for Acetic acid
255	IS 5311	Acetic anhydride Code of safety for Carbon Tetrachloride
256	IS 5685	Carbon Tetrachloride Code of safety for Carbon Disulphide (Carbon Bisulphide)

No	IS code	Indian Standard
257	IS 5931	Code of safety for Handling of cryogenic liquids
258	IS 6156	Code of safety for Chlorosulphonic acid
259	IS 6164	Code of safety for Hydrochloric acid
260	IS 6269	Code of safety for Ethylene oxide
261	IS 6270	Code of safety for Phenol
262	IS 6818	Code of safety for Phosphoric acid
263	IS 6819	Code of safety for Calcium carbide
264	IS 6955	Code of safety for Bromine
265	IS 6954	Code of safety for Caustic potash
266	IS 7415	Code of safety for Aniline
267	IS 7420	Code of safety for phthalic anhydride
268	IS 7444	Code of safety for Methanol
269	IS 7445	Code of safety for Acetone
270	IS 7812	Code of safety for Mercury
271	IS 8185	Code of safety for Phosgene
272	IS 8388	Code of safety for Nitrobenzene
273	IS 9052	Code of safety for Aluminium chloride, anhydrous
274	IS 9053	Code of safety for M-dinitrobenzene
275	IS 9277	Code of safety for Monochlorobenzene
276	IS 9278	Code of safety for Zinc phosphide
277	IS 9279	Code of safety for Aluminium phosphide
278	IS 9744	Code of safety for thionyl chloride
279	IS 9785	Code of safety for Aluminium alkyls
280	IS 9786	Code of safety for Vinyl chloride monomer (VCM)

No	IS code	Indian Standard
281	IS 9787	Code of safety for Phosphoryl chloride
282	IS 10870	Code of safety for Hexane
283	IS 10872	Code of safety for Malathion
284	IS 10920	Code of safety for Phosphorus Trichloride
285	IS 11141	Code of safety for Acrylonitrile
286	IS 12033	Code of safety for Dinitro Toluene (DNT)
287	IS 12034	Code of safety for Methyl bromide
288	IS 12035	Code of safety in Microbiological Laboratories
289	IS 12141	Code of safety for methyl ethyl ketone
290	IS 12142	Code of safety for 1.1.1 trichloro ethane
291	IS 12143	Code of safety for tetrachloroethane
		Sanitary Appliances & Valves
292	IS 771 : Part 1	Specification for glazed fire clay sanitary appliances: part 1 General requirements.
293	IS 771 : Part 2	Specification for glazed fire-clay sanitary appliances : part 2 specific requirements of kitchen and laboratory sinks
294	IS 771 : Part 3 : Sec 1	Specification for glazed fire-clay sanitary appliances part 3 specific requirements of urinals section 1 slab urinals
295	IS 771 : Part 3 : Sec 2	Specification for glazed fire-clay sanitary appliances part 3 specific requirements of urinals section 2 stall urinals
296	IS 771 : Part 4	Specification for glazed fire-clay sanitary appliances part 4 specific requirements of post-mortem slabs
297	IS 771 : Part 5	Specification for glazed fire-clay sanitary appliances part 5 specific requirements of shower trays
298	IS 771 : Part 6	Specification for glazed fire-clay sanitary appliances part 6 specific requirements of bed pan sinks
299	IS 771 : Part 7	Specification for glazed fire-clay sanitary appliances part 7 specific requirements of slope sinks

No	IS code	Indian Standard
300	IS 772	Specification for general requirements for enamelled cast iron sanitary appliances
301	IS 774	Specification for flushing cistern for water closets and urinals (other than plastic cisterns)
302	IS 14846	Specification for Sluice Valve for Water Works Purposes (50 to 1200 mm Size)
303	IS 1726	Specification for cast iron manhole covers and frames
304	IS: 5455	Specification for cast iron steps for manholes
305	IS 2064	Code of practice for selection, installation and maintenance of sanitary appliances
306	IS 2326	Specification for automatic flushing cisterns for urinals
307	IS 2548 : Part 1	Specification for plastic seats and covers for water closets : part 1 Thermo set seats and covers
308	IS 2548 : Part 2	Specification for plastic seats and covers for water closets : part 2 Thermo set plastic seats and covers
309	IS 2556 : Part 1	Specification for vitreous sanitary appliances (vitreous china) : Part 1 General requirements
310	IS 2556 : Part 2	Specification for vitreous sanitary appliances (Vitreous china) : part 2 specific requirements of wash down water-closets
311	IS 2556 : Part 3	Specification for vitreous sanitary appliances (Vitreous china) : part 3 specific requirements of squatting pans
312	IS 2556 : Part 4	Specification for vitreous sanitary appliances (Citreous china): part 4 specific requirements of wash basins
313	IS 2556 : Part 5	Specification for vitreous sanitary appliances (vitreous china) : part 5 specific requirements of laboratory sinks
314	IS 2556 : Part 6	Specific requirement of urinals and partition plates
315	IS 2556 : Part 7	Specification for vitreous sanitary appliances (vitreous china) Part 7 Specific requirements of half round channels Specific requirements of accessories for sanitary appliances
316	IS 2556 : Part 8	Specification for vitreous sanitary appliances (vitreous china) Part 8 Specific requirements of pedestal closed coupled wash-down and syphonic water closets

No	IS code	Indian Standard
317	IS 2556 : Part 9	Specification for vitreous sanitary appliances (vitreous china) : Part 9 Specific requirements of pedestal type bidets
318	IS 2556 : Part 14	Specification tor vitreous sanitary appliances (vitreous china): Part 14 Specific requirements of integrated squatting pans.
319	IS 2556 : Part 15	Specification for vitreous sanitary appliances (vitreous china) Part 15 specific requirements of universal water closets.
320	IS 2685	Code of practice for selection, installation and maintenance of sluice valves
321	IS 2963	Specification for copper alloy waste fittings for wash basins and sinks
322	IS 3042	Specification for single faced sluice gates (200 to 1200 mm size)
323	IS 3311	Specification for waste plug and its accessories for sinks and wash-basins
324	IS 3950	Specification for surface boxes for sluice valves
325	IS 4038	Specification for foot valves for water works purposes
326	IS 4346	Specification for washers for use with fittings for water services
327	IS 5219	Specification for cast copper alloys traps, part 1 ‘P’ and S’ traps
328	IS 5312 : Part 1	Swing Check Type Reflux (Non-Return] Valves for Water Works Purposes - Part 1 : Single-Door Pattern
329	IS 5312 : Part 2	Swing Check Type Reflux (non-return) Valves for Water Works Purpose - Part 2 : Multi-Door Pattern
330	IS 5961	Cast iron gratings for drainage purposes
331	IS 6411	Gel-coated glass fibre reinforced polyester resin bath tubs
332	IS 7231	Plastic flushing cisterns for water closets and urinals
333	IS 9739	Specification for pressure reducing valves for domestic water supply systems.
334	IS 9758	Specification for flush valves and fittings for water closets and urinals
335	IS 9762	Specification for polyethylene floats (spherical) for float valves

No	IS code	Indian Standard
336	IS 11246	Specification for glass fibre reinforced polyester resins (GRP) squatting pans
337	IS 12234	Specification for plastic equilibrium float valves for cold water services
338	IS 12701	Specification for rotational moulded polyethylene water storage tanks.
		Fluid Flow Measurements
339	IS 1192	Velocity area methods for measurement of flow of water in open channels.
340	IS 2912	Liquid Flow Measurement in Open Channels - Slope Area Method
341	IS 15122	Measurement of Liquid Flow in Open Channels Under Tidal Conditions
342	IS 15119 : Part 1	Measurement of Liquid Flow in Open Channels - Part 1 : Establishment and Operation of a Gauging Station
343	IS 15119 : Part 2	Measurement of Liquid Flow in Open Channels - Part 2 : Determination of the Stage-Discharge Relation
344	IS 14615 : Part 1	Measurement of Fluid Flow by Means of Pressure Differential Devices - Part 1 : Orifice Plates, Nozzles and Venturi Tubes Inserted in Circular cross-section conduits running full
345	IS 4477 : Part 2	Methods of measurement of fluid flow by means of venturi meters Part 2 compressible fluids
346	IS 14974	Liquid flow Measurement in Open Channels by Weirs and Flumes - Rectangular Broad-crested Weirs
347	IS 6062	Method of measurement of flow of water in open channels using standing wave flume - fall.
348	IS 6063	Method of measurement of flow of water in open channels using standing wave flumes
349	IS 6330	Liquid Flow Measurement in Open Channels by Weirs and Flumes - End Depth Method for Estimation of Flow in Rectangular Channels with a Free Overfall (Approximate Method)
350	IS 9108	Liquid flow measurement in open channels using thin plate weirs
351	IS 9115	Method for estimation of incompressible fluid flow in closed conduits by bend meters
352	IS 14574	Measurement of Liquid Flow in Open Channels by Weirs and Flumes - End Depth Method for Estimation of Flow in Non-rectangular Channels with a Free Overfall (Approximate Method)

No	IS code	Indian Standard
353	IS 9119	Method for flow estimation by jet characteristics (approximate method).
354	IS 9163 : Part 1	Dilution methods of measurement of steady flow part 1 constant rate injection method.
355	IS 9922	Measurement of Liquid Flow in Open Channels - General Guidelines for Selection of Method
356	IS 12752	Guidelines for the selection of flow gauging structures
357	IS 16098 : Part 1	Structured Wall Plastics Piping Systems for Non-pressure Drainage and Sewerage, Pipes and Fittings with smooth external surface, Type A
358	IS 16098 : Part 2	Structured Wall Plastics Piping Systems for Non-pressure Drainage and Sewerage, Pipes and Fittings with non-smooth external surface, Type B.

Note: For the elaborate list refer to the Bureau of Indian Standards.

APPENDIX A.1.1 THE PROHIBITION OF EMPLOYMENT AS MANUAL SCAVENGERS AND THEIR REHABILITATION ACT, 2013

रजिस्ट्री सं॰ डी॰ एल॰—(एन)04/0007/2003—13 REGISTERED NO. DL—(N)04/0007/2003—13

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इस भाग में भिन्न पृष्ठ संख्या दी जाती है जिससे कि यह अलग संकलन के रूप में रखा जा सके।
Separate paging is given to this Part in order that it may be filed as a separate compilation.

MINISTRY OF LAW AND JUSTICE

(Legislative Department)

New Delhi, the 19*th September*, 2013/*Bhadra* 28, 1935 (*Saka*)

The following Act of Parliament received the assent of the President on the 18th September, 2013, and is hereby published for general information:—

THE PROHIBITION OF EMPLOYMENT AS MANUAL SCAVENGERS AND THEIR REHABILITATION ACT, 2013

No. 25 of 2013

[18*th September*, 2013.]

An Act to provide for the prohibition of employment as manual scavengers, rehabilitation of manual scavengers and their families, and for matters connected therewith or incidental thereto.

Whereas promoting among the citizens fraternity assuring the dignity of the individual is enshrined as one of the goals in the Preamble to the Constitution;

And whereas the right to live with dignity is also implicit in the Fundamental Rights guaranteed in Part III of the Constitution;

And whereas article 46 of the Constitution, *inter alia,* provides that the State shall protect the weaker sections, and, particularly, the Scheduled Castes and the Scheduled Tribes from social injustice and all forms of exploitation;

And whereas the dehumanising practice of manual scavenging, arising from the continuing existence of insanitary latrines and a highly iniquitous caste system, still persists in various parts of the country, and the existing laws have not proved adequate in eliminating the twin evils of insanitary latrines and manual scavenging;

AND WHEREAS it is necessary to correct the historical injustice and indignity suffered by the manual scavengers, and to rehabilitate them to a life of dignity.

BE it enacted by Parliament in the Sixty-fourth Year of the Republic of India as follows:—

CHAPTER I

PRELIMINARY

Short title, extent and commencement.

1. (*1*) This Act may be called the Prohibition of Employment as Manual Scavengers and their Rehabilitation Act, 2013.

(*2*) It extends to the whole of India except the State of Jammu and Kashmir.

(*3*) It shall come into force on such date as the Central Government may, by notification in the Official Gazette, appoint:

Provided that the date so notified shall not be earlier than sixty days after the date of publication of the notification in the Official Gazette.

Definitions.

2. (*1*) In this Act, unless the context otherwise requires,—

(*a*) "agency" means any agency, other than a local authority, which may undertake sanitation facilities in an area and includes a contractor or a firm or a company which engages in development and maintenance of real estate;

(*b*) "appropriate government", in relation to Cantonment Boards, railway lands, and lands and buildings owned by the Central Government, a Central Public Sector Undertaking or an autonomous body wholly or substantially funded by the Central Government, means the Central Government and in all other cases, the State Government;

(*c*) "Chief Executive Officer", in relation to a Municipality or Panchayat, means, its senior-most executive officer, by whatever name called;

(*d*) "hazardous cleaning" by an employee, in relation to a sewer or septic tank, means its manual cleaning by such employee without the employer fulfilling his obligations to provide protective gear and other cleaning devices and ensuring observance of safety precautions, as may be prescribed or provided in any other law, for the time being in force or rules made thereunder;

(*e*) "insanitary latrine" means a latrine which requires human excreta to be cleaned or otherwise handled manually, either *in situ*, or in an open drain or pit into which the excreta is discharged or flushed out, before the excreta fully decomposes in such manner as may be prescribed:

Provided that a water flush latrine in a railway passenger coach, when cleaned by an employee with the help of such devices and using such protective gear, as the Central Government may notify in this behalf, shall not be deemed to be an insanitary latrine.

(*f*) "local authority" means,—

(*i*) a Municipality or a Panchayat, as defined in clause (*e*) and clause (*f*) of article 243P of the Constitution, which is responsible for sanitation in its area of jurisdiction;

(*ii*) a Cantonment Board constituted under section 10 of the Cantonments Act, 2006; and 41 of 2006.

(*iii*) a railway authority;

(*g*) "manual scavenger" means a person engaged or employed, at the commencement of this Act or at any time thereafter, by an individual or a local authority or an agency or a contractor, for manually cleaning, carrying, disposing of, or otherwise handling in any manner, human excreta in an insanitary latrine or in an open drain or pit into which the human excreta from the insanitary latrines is disposed of, or on a railway track or in such other spaces or premises, as the Central Government or a State Government may notify, before the excreta fully decomposes in such manner as may be prescribed, and the expression "manual scavenging" shall be construed accordingly.

Explanation.—For the purpose of this clause,—

(*a*) "engaged or employed" means being engaged or employed on a regular or contract basis;

(*b*) a person engaged or employed to clean excreta with the help of such devices and using such protective gear, as the Central Government may notify in this behalf, shall not be deemed to be a 'manual scavenger';

(*h*) "National Commission for Safai Karmacharis" means the National Commission for Safai Karamcharis constituted under section 3 of the National Commission for Safai Karmacharis Act, 1993 and continued by Resolution of the Government of India in the Ministry of Social Justice and Empowerment *vide* No.17015/18/2003-SCD-VI, dated 24th February, 2004 and as amended from time to time;

64 of 1993.

(*i*) "notification" means a notification published in the Official Gazette and the expression "notify" shall be construed accordingly;

(*j*) "occupier", in relation to the premises where an insanitary latrine exists, or someone is employed as a manual scavenger, means the person who, for the time being, is in occupation of such premises;

(*k*) "owner", in relation to the premises where an insanitary latrine exists or someone is employed as a manual scavenger, means, the person who, for the time being has legal title to such premises;

(*l*) "prescribed" means prescribed by the rules made under this Act;

(*m*) "railway authority" means an authority administering railway land, as may be notified by the Central Government in this behalf;

(*n*) "railway land" shall have the meaning assigned to it in clause (*32A*) of section 2 of the Railways Act, 1989;

24 of 1989.

(*o*) "sanitary latrine" means a latrine which is not an 'insanitary latrine';

(*p*) "septic tank" means a water-tight settling tank or chamber, normally located underground, which is used to receive and hold human excreta, allowing it to decompose through bacterial activity;

(*q*) "sewer" means an underground conduit or pipe for carrying off human excreta, besides other waste matter and drainage wastes;

(*r*) "State Government", in relation to a Union territory, means the Administrator thereof appointed under article 239 of the Constitution;

(*s*) "survey" means a survey of manual scavengers undertaken in pursuance of section 11 or section 14.

(*2*) Words and expressions used and not defined in this Act, but defined in the Cantonments Act, 2006, shall have the same meanings respectively assigned to them in that Act.

41 of 2006.

(*3*) The reference to a Municipality under Chapters IV to VIII of this Act shall include a reference to, as the case may be, the Cantonment Board or the railway authority, in respect of areas included within the jurisdiction of the Cantonment Board and the railway land, respectively.

Act to have overriding effect.

3. The provisions of this Act shall have effect notwithstanding anything inconsistent therewith contained in the Employment of Manual Scavengers and Construction of Dry Latrines (Prohibition) Act, 1993 or in any other law, or in any instrument having effect by virtue of any other law. 46 of 1993.

CHAPTER II

IDENTIFICATION OF INSANITARY LATRINES

Local authorities to survey insanitary latrines and provide sanitary community latrines.

4. (*1*) Every local authority shall,—

(*a*) carry out a survey of insanitary latrines existing within its jurisdiction, and publish a list of such insanitary latrines, in such manner as may be prescribed, within a period of two months from the date of commencement of this Act;

(*b*) give a notice to the occupier, within fifteen days from the date of publication of the list under clause (*a*), to either demolish the insanitary latrine or convert it into a sanitary latrine, within a period of six months from the date of commencement of this Act:

Provided that the local authority may for sufficient reasons to be recorded in writing extend the said period not exceeding three months;

(*c*) construct, within a period not exceeding nine months from the date of commencement of this Act, such number of sanitary community latrines as it considers necessary, in the areas where insanitary latrines have been found.

(*2*) Without prejudice to the provisions contained in sub-section (*1*), Municipalities, Cantonment Boards and railway authorities shall also construct adequate number of sanitary community latrines, within such period not exceeding three years from the date of commencement of this Act, as the appropriate Government may, by notification, specify, so as to eliminate the practice of open defecation in their jurisdiction.

(*3*) It shall be the responsibility of local authorities to construct community sanitary latrines as specified in sub-sections (*1*) and (*2*), and also to make arrangements for their hygienic upkeep at all times.

Explanation.—For the purposes of this section, "community" in relation to railway authorities means passengers, staff and other authorised users of railways.

CHAPTER III

PROHIBITION OF INSANITARY LATRINES AND EMPLOYMENT AND ENGAGEMENT AS MANUAL SCAVENGER

Prohibition of insanitary latrines and employment and engagement of manual scavenger.

5. (*1*) Notwithstanding anything inconsistent therewith contained in the Employment of Manual Scavengers and Construction of Dry Latrines (Prohibition) Act, 1993, no person, local authority or any agency shall, after the date of commencement of this Act,— 46 of 1993.

(*a*) construct an insanitary latrine; or

(*b*) engage or employ, either directly or indirectly, a manual scavenger, and every person so engaged or employed shall stand discharged immediately from any obligation, express or implied, to do manual scavenging.

(*2*) Every insanitary latrine existing on the date of commencement of this Act, shall either be demolished or be converted into a sanitary latrine, by the occupier at his own cost, before the expiry of the period so specified in clause (*b*) of sub-section (*1*) of section 4:

Provided that where there are several occupiers in relation to an insanitary latrine, the liability to demolish or convert it shall lie with,—

(*a*) the owner of the premises, in case one of the occupiers happens to be the owner; and (*b*) all the occupiers, jointly and severally, in all other cases:

Provided that the State Government may give assistance for conversion of insanitary latrines into sanitary latrines to occupiers from such categories of persons and on such scale, as it may, by notification, specify:

Provided further that non-receipt of State assistance shall not be a valid ground to maintain or use an insanitary latrine, beyond the said period of nine months.

(*3*) If any occupier fails to demolish an insanitary latrine or convert it into a sanitary latrine within the period specified in sub-section (*2*), the local authority having jurisdiction over the area in which such insanitary latrine is situated, shall, after giving notice of not less than twenty one days to the occupier, either convert such latrine into a sanitary latrine, or demolish such insanitary latrine, and shall be entitled to recover the cost of such conversion or, as the case may be, of demolition, from such occupier in such manner as may be prescribed.

Contract, agreement, etc., to be void.

6. (*1*) Any contract, agreement or other instrument entered into or executed before the date of commencement of this Act, engaging or employing a person for the purpose of manual scavenging shall, on the date of commencement of this Act, be terminated and such contract, agreement or other instrument shall be void and inoperative and no compensation shall be payable therefor.

(*2*) Notwithstanding anything contained in sub-section (*1*), no person employed or engaged as a manual scavenger on a full-time basis shall be retrenched by his employer, but shall be retained, subject to his willingness, in employment on at least the same emoluments, and shall be assigned work other than manual scavenging.

Prohibition of persons from engagement or employment for hazardous cleaning of sewers and septic tanks.

7. No person, local authority or any agency shall, from such date as the State Government may notify, which shall not be later than one year from the date of commencement of this Act, engage or employ, either directly or indirectly, any person for hazardous cleaning of a sewer or a septic tank.

Penalty for contravention of section 5 or section 6.

8. Whoever contravenes the provisions of section 5 or section 6 shall for the first contravention be punishable with imprisonment for a term which may extend to one year or with fine which may extend to fifty thousand rupees or with both, and for any subsequent contravention with imprisonment which may extend to two years or with fine which may extend to one lakh rupees, or with both.

Penalty for contravention of section 7.

9. Whoever contravenes the provisions of section 7 shall for the first contravention be punishable with imprisonment for a term which may extend to two years or with fine which may extend to two lakh rupees or with both, and for any subsequent contravention with imprisonment which may extend to five years or with fine which may extend to five lakh rupees, or with both.

Limitation of prosecution.

10. No court shall take cognizance of any offence punishable under this Act except upon a complaint thereof is made by a person in this behalf within three months from the date of the occurrence of the alleged commission of the offence.

CHAPTER IV

IDENTIFICATION OF MANUAL SCAVENGERS IN URBAN AND RURAL AREAS AND THEIR REHABILITATION

Survey of manual scavengers in urban areas by Municipalities.

11. (*1*) If any Municipality has reason to believe that some persons are engaged or employed in manual scavenging within its jurisdiction, the Chief Executive Officer of such Municipality shall cause a survey to be undertaken to identify such persons.

(*2*) The content and methodology of the survey referred to in sub-section (*1*) shall be such as may be prescribed, and it shall be completed within a period of two months from its commencement in the case of Municipal Corporations, and within a period of one month in the case of other Municipalities.

(*3*) The Chief Executive Officer of the Municipality, in whose jurisdiction the survey is undertaken, shall be responsible for accurate and timely completion of the survey.

(*4*) After completion of the survey, the Chief Executive Officer shall cause to be drawn up a provisional list of persons found to be working as manual scavengers within the jurisdiction of his Municipality and fulfilling the eligibility conditions as may be prescribed, shall cause such provisional list to be published for general information in such manner, as may be prescribed, and shall invite objections to the list from the general public.

(*5*) Any person having any objection, either to the inclusion or exclusion of any name in the provisional list published in pursuance of sub-section (*4*), shall, within a period of fifteen days from such publication, file an objection, in such form as the Municipality may notify, to the Chief Executive Officer.

(*6*) All objections received in pursuance of sub-section (*5*), shall be enquired into, and thereafter a final list of persons found to be working as manual scavengers within the local limits of the municipality, shall be published by it in such manner, as may be prescribed.

(*7*) As soon as the final list of manual scavengers, referred to in sub-section (*6*) is published, the persons included in the said list shall, subject to the provisions of sub-section (*2*) of section 6, stand discharged from any obligation to work as manual scavengers.

Application by an urban manual scavenger for identification.

12. (*1*) Any person working as a manual scavenger in an urban area, may, either during the survey undertaken by the Municipality in pursuance of section 11, within whose jurisdiction he works, or at any time thereafter, apply, in such manner, as may be prescribed, to the Chief Executive Officer of the Municipality, or to any other officer authorised by him in this behalf, for being identified as a manual scavenger.

(*2*) On receipt of an application under sub-section (*1*), the Chief Executive Officer shall cause it to be enquired into, either as part of the survey undertaken under section 11, or, when no such survey is in progress, within fifteen days of receipt of such application, to ascertain whether the applicant is a manual scavenger.

(*3*) If an application is received under sub-section (*1*) when a survey under section 11 is not in progress, and is found to be true after enquiry in accordance with sub-section (*2*), action shall be taken to add the name of such a person to the final list published under sub-section (*6*) of section 11, and the consequences mentioned in sub-section (*7*) thereof shall follow.

Rehabilitation of persons identified as manual scavengers by a Municipality.

13. (*1*) Any person included in the final list of manual scavengers published in pursuance of sub-section (*6*) of section 11 or added thereto in pursuance of sub-section (*3*) of section 12, shall be rehabilitated in the following manner, namely:—

(*a*) he shall be given, within one month,—

(*i*) a photo identity card, containing, *inter alia,* details of all members of his family dependent on him, and

(*ii*) such initial, one time, cash assistance, as may be prescribed;

(*b*) his children shall be entitled to scholarship as per the relevant scheme of the Central Government or the State Government or the local authorities, as the case may be;

(*c*) he shall be allotted a residential plot and financial assistance for house construction, or a ready-built house, with financial assistance, subject to eligibility and willingness of the manual scavenger, and the provisions of the relevant scheme of the Central Government or the State Government or the concerned local authority;

(*d*) he, or at least one adult member of his family, shall be given, subject to eligibility and willingness, training in a livelihood skill, and shall be paid a monthly stipend of not less than three thousand rupees, during the period of such training;

(*e*) he, or at least one adult member of his family, shall be given, subject to eligibility and willingness, subsidy and concessional loan for taking up an alternative occupation on a sustainable basis, in such manner as may be stipulated in the relevant scheme of the Central Government or the State Government or the concerned local authority;

(*f*) he shall be provided such other legal and programmatic assistance, as the Central Government or State Government may notify in this behalf.

(*2*) The District Magistrate of the district concerned shall be responsible for rehabilitation of each manual scavenger in accordance with the provisions of sub-section (*1*) and the State Government or the District Magistrate concerned may, in addition, assign responsibilities in his behalf to officers subordinate to the District Magistrate and to officers of the concerned Municipality.

Survey of manual scavengers in rural areas by Panchayats.

14. If any Panchayat has reason to believe that some persons are engaged in manual scavenging within its jurisdiction, the Chief Executive Officer of such Panchayat shall cause a survey of such manual scavengers to be undertaken, *mutatis mutandis,* in accordance with the provisions of section 11 and section 12, to identify such person.

Application by a rural manual scavenger for identification.

15. (*1*) Any person working as a manual scavenger, in a rural area, may, either during the survey undertaken by the Panchayat within whose jurisdiction he works, in pursuance of section 14 or at any time thereafter, apply, in such manner, as may be prescribed, to the Chief Executive Officer of the concerned Panchayat, or to any other officer authorised by him in this behalf, for being identified as a manual scavenger.

(*2*) On receipt of an application under sub-section (*1*), the Chief Executive Officer shall cause it to be enquired into, either as part of the survey undertaken under section 14 or when no such survey is in progress, within fifteen days of receipt of such application, so as to ascertain whether the applicant is a manual scavenger.

Rehabilitation of persons identified as manual scavengers by a Panchayat.

16. Any person included in the final list of manual scavengers, published in pursuance of section 14 or added thereto in pursuance of sub-section (*2*) of section 15 shall be rehabilitated, *mutatis mutandis*, in the manner laid down for urban manual scavengers in section 13.

CHAPTER V

IMPLEMENTING AUTHORITIES

Responsibility of local authorities to ensure elimination of insanitary latrines.

17. Notwithstanding anything contained in any other law for the time being in force, it shall be the responsibility of every local authority to ensure, through awareness campaign or in such other manner that after the expiry of a period of nine months, from the date of commencement of this Act,—

(*i*) no insanitary latrine is constructed, maintained or used within its jurisdiction; and

(*ii*) in case of contravention of clause (*i*), action is taken against the occupier under sub-section (*3*) of section 5.

Authorities who may be specified for implementing provisions of this Act.

18. The appropriate Government may confer such powers and impose such duties on local authority and District Magistrate as may be necessary to ensure that the provisions of this Act are properly carried out, and a local authority and the District Magistrate may, specify the subordinate officers, who shall exercise all or any of the powers, and perform all or any of the duties, so conferred or imposed, and the local limits within which such powers or duties shall be carried out by the officer or officers so specified.

Duty of District Magistrate and authorised officers.

19. The District Magistrate and the authority authorised under section 18 or any other subordinate officers specified by them under that section shall ensure that, after the expiry of such period as specified for the purpose of this Act,—

(*a*) no person is engaged or employed as manual scavenger within their jurisdiction;

(*b*) no one constructs, maintains, uses or makes available for use, an insanitary latrine;

(*c*) manual scavengers identified under this Act are rehabilitated in accordance with section 13, or as the case may be, section 16;

(*d*) persons contravening the provisions of section 5 or section 6 or section 7 are investigated and prosecuted under the provisions of this Act; and

(*e*) all provisions of this Act applicable within his jurisdiction are duly complied with.

Appointment of inspectors and their powers.

20. (*1*) The appropriate Government may, by notification, appoint such persons as it thinks fit to be inspectors for the purposes of this Act, and define the local limits within which they shall exercise their powers under this Act.

(*2*) Subject to any rules made in this behalf, an inspector may, within the local limits of his jurisdiction, enter, at all reasonable times, with such assistance as he considers necessary, any premises or place for the purpose of,—

(*a*) examining and testing any latrine, open drain or pit or for conducting an inspection of any premises or place, where he has reason to believe that an offence under this Act has been or is being or is about to be committed, and to prevent employment of any person as manual scavenger;

(*b*) examine any person whom he finds in such premises or place and who, he has reasonable cause to believe, is employed as a manual scavenger therein, or is otherwise in a position to furnish information about compliance or non-compliance with the provisions of this Act and the rules made thereunder;

(*c*) require any person whom he finds on such premises, to give information which is in his power to give, with respect to the names and addresses of persons employed on such premises as manual scavenger and of the persons or agency or contractor employing or engaging them;

(*d*) seize or take copies of such registers, record of wages or notices or portions thereof as he may consider relevant in respect of an offence under this Act which he has reason to believe has been committed by the principal employer or agency; and

(*e*) exercise such other powers as may be prescribed.

(*3*) Any person required to produce any document or thing or to give any information required by an inspector under sub-section (*2*) shall be deemed to be legally bound to do so within the meaning of section 175 and section 176 of the Indian Penal Code. 45 of 1860.

(*4*) The provisions of the Code of Criminal Procedure, 1973, shall, so far as may be, apply to any such search or seizure under sub-section (*2*) as they apply to such search or seizure made under the authority of a warrant issued under section 94 of the said Code. 2 of 1974.

CHAPTER VI

PROCEDURE FOR TRIAL

Offences to be tried by Executive Magistrate.

21. (*1*) The State Government may confer, on an Executive Magistrate, the powers of a Judicial Magistrate of the first class for the trial of offences under this Act; and, on such conferment of powers, the Executive Magistrate, on whom the powers are so conferred, shall be deemed, for the purposes of the Code of Criminal Procedure, 1973, to be a Judicial Magistrate of the first class. 2 of 1974.

(*2*) An offence under this Act may be tried summarily.

Offence to be cognizable and non-bailable.

22. Notwithstanding anything contained in the Code of Criminal Procedure, 1973, every offence under this Act shall be cognizable and non-bailable. 2 of 1974.

Offences by companies.

23. (*1*) Where an offence under this Act has been committed by a company, every person who, at the time the offence was committed, was in charge of, and was responsible to, the company for the conduct of the business of the company, as well as the company, shall be deemed to be guilty of the offence and shall be liable to be proceeded against and punished accordingly.

(*2*) Notwithstanding anything contained in sub-section (*1*), where any offence under this Act has been committed by a company and it is proved that offence has been committed with the consent or connivance of, or is attributable to, any neglect on the part of, any director, manager, secretary or other officer of the company, such director, manager, secretary or other officer shall be deemed to be guilty of that offence and shall be liable to be proceeded against and punished accordingly.

Explanation.—For the purposes of this section,—

(*a*) "company" means any body corporate and includes a firm or other association of individuals; and

(*b*) "director" in relation to a firm, means a partner in the firm.

CHAPTER VII

VIGILANCE COMMITTEES

Vigilance Committees.

24. (*1*) Every State Government shall, by notification, constitute a Vigilance Committee for each district and each Sub-Division.

(*2*) Each Vigilance Committee constituted for a district shall consist of the following members, namely:—

(*a*) the District Magistrate—Chairperson, *ex officio*;

(*b*) all members of the State Legislature belonging to the Scheduled Castes elected from the district—members:

Provided that if a district has no member of the State Legislature belonging to the Scheduled Castes, the State Government may nominate such number of other members of the State Legislature from the district, not exceeding two, as it may deem appropriate.

(*c*) the district Superintendent of Police— member, *ex officio*;

(*d*) the Chief Executive Officer of,—

(*i*) the Panchayat at the district level—member, *ex officio*;

(*ii*) the Municipality of the district headquarters—member, *ex officio*;

(*iii*) any other Municipal Corporation constituted in the district—member, *ex officio*;

(*iv*) Cantonment Board, if any, situated in the district—member, *ex officio*;

(*e*) one representative be nominated by the railway authority located in the district;

(*f*) not more than four social workers belonging to organisation working for the prohibition of manual scavenging and rehabilitation of manual scavengers, or, representing the scavenger community, resident in the district, to be nominated by the District Magistrate, two of whom shall be women;

(*g*) one person to represent the financial and credit institutions in the district, to be nominated by the District Magistrate;

(*h*) the district-level officer in-charge of the Scheduled Castes Welfare—Member-Secretary, *ex officio*;

(*i*) district-level officers of Departments and agencies who, in the opinion of the District Magistrate, subject to general orders, if any, of the State Government, have a significant role to play in the implementation of this Act.

(*3*) Each Vigilance Committee, constituted for a Sub-Division, shall consist of the following members, namely:—

(*a*) the Sub-Divisional Magistrate—Chairperson, *ex officio*;

(*b*) the Chairpersons and the Chief Executive Officers of Panchayats at intermediate level of the Sub-Division, and where Panchayats at intermediate level, do not exist, Chairpersons from two Panchayats at Village level to be nominated by the Sub-Divisional Magistrate—member, *ex officio*;

(*c*) the Sub-Divisional Officer of Police—member, *ex officio*;

(*d*) Chief Executive Officer of—

(*i*) the Municipality of the Sub-Divisional headquarters—member, *ex officio*; and

(*ii*) Cantonment Board, if any, situated in the Sub-Division—member, *ex officio*;

(*e*) one representative to be nominated by the railway authority located in the Sub-Division—member, *ex officio*;

(*f*) two social workers belonging to the organisation working for the prohibition of manual scavenging and rehabilitation of the manual scavengers, or representing the scavenger community resident in the Sub-Division, to be nominated by the District Magistrate, one of whom shall be a woman;

(*g*) one person to represent the financial and credit institutions in the Sub-Division, to be nominated by the Sub-Divisional Magistrate;

(*h*) the Sub-Divisional level officer in-charge of Scheduled Castes welfare—Member-Secretary, *ex officio;*

(*i*) Sub-Divisional level officers of Department and agencies who in the opinion of the Sub-Divisional Magistrate, subject to any general orders of the State Government or the District Magistrate, have a significant role to play in the implementation of this Act—member, *ex officio.*

(*4*) Each Vigilance Committee constituted at district and Sub-Divisional level shall meet at least once in every three months.

(*5*) No proceeding of a Vigilance Committees shall be invalid merely by reason of any defect in its constitution.

Functions of Vigilance Committee.

25. The functions of Vigilance Committee shall be—

(*a*) to advise the District Magistrate or, as the case may be, the Sub-Divisional Magistrate, on the action which needs to be taken, to ensure that the provisions of this Act or of any rule made thereunder are properly implemented;

(*b*) to oversee the economic and social rehabilitation of manual scavengers;

(*c*) to co-ordinate the functions of all concerned agencies with a view to channelise adequate credit for the rehabilitation of manual scavengers;

(*d*) to monitor the registration of offences under this Act and their investigation and prosecution.

State Monitoring Committee.

26. (*1*) Every State Government shall, by notification, constitute a State Monitoring Committee, consisting of the following members, namely:—

(*a*) the Chief Minister of State or a Minister nominated by him—Chairperson, *ex officio*;

(*b*) the Minister-in-charge of the Scheduled Castes Welfare, and such other Department, as the State Government may notify;

(*c*) Chairperson of the State Commissions for Safai Karamcharis, and Scheduled Castes, if any— member, *ex officio;*

(*d*) representatives of the National Commission for Scheduled Castes, and Safai Karamcharis—member, *ex officio;*

(*e*) not less than two members of the State Legislature belonging to the Scheduled Castes, nominated by the State Government:

Provided that if any State Legislature has no member belonging to the Scheduled Castes, the State Government may nominate the members belonging to the Scheduled Tribes;

(*f*) the Director-General of Police— member, *ex officio;*

(*g*) Secretaries to the State Government in the Departments of Home, Panchayati Raj, Urban Local Bodies, and such other Departments, as the State Government may notify;

(*h*) Chief Executive Officer of at least one Municipal Corporation, Panchayat at the district-level, Cantonment Board and railway authority as the State Government may notify;

(*i*) not more than four social workers belonging to organisation working for the prohibition of manual scavenging and rehabilitation of manual scavengers, or, representing the scavenger community, resident in the State, to be nominated by the State Government, two of whom shall be women;

(*j*) State-level head of the convener Bank of the State Level Bankers' Committee—member, *ex officio*;

(*k*) Secretary of the Department of the State Government dealing with development of the Scheduled Castes—Member-Secretary, *ex officio*;

(*l*) such other representative of Departments of the State Government and such other agencies which, in the opinion of the State Government, are concerned with the implementation of this Act.

(*2*) The State Monitoring Committee shall meet at least once in every six months and shall observe such rules of procedure in regard to the transaction of business at its meetings as may be prescribed.

Functions of the State Monitoring Committee.

27. The functions of the State Monitoring Committee shall be—

(*a*) to monitor and advise the State Government and local authorities for effective implementation of this Act;

(*b*) to co-ordinate the functions of all concerned agencies;

(*c*) to look into any other matter incidental thereto or connected therewith for implementation of this Act.

Duty of States or Union territories to send periodic reports to the Central Government.

28. Every State or Union territory Government and Union territory administration shall send such periodic reports to the Central Government about progress of implementation of this Act, as the Central Government may require.

Central Monitoring Committee.

29. (*1*) The Central Government shall, by notification, constitute a Central Monitoring Committee in accordance with the provisions of this section.

(*2*) The Central Monitoring Committee shall consist of the following members, namely:—

(*a*) The Union Minister for Social Justice and Empowerment—Chairperson, *ex officio*;

(*b*) Chairperson of the National Commission for Scheduled Castes—member, *ex officio*;

(*c*) Minister of State in the Ministry of Social Justice and Empowerment—member, *ex officio*;

(*d*) Chairperson, National Commission for Safai Karamcharis— member, *ex officio*;

(*e*) the Member of the Planning Commission dealing with development of the Scheduled Castes—member, *ex officio*;

(*f*) three elected members of Parliament belonging to Scheduled Castes, two from the Lok Sabha and one from the Rajya Sabha;

(*g*) Secretaries of the Ministries of,—

(*i*) Social Justice and Empowerment, Department of Social Justice and Empowerment;

(*ii*) Urban Development;

(*iii*) Housing and Urban Poverty Alleviation;

(*iv*) Drinking Water and Sanitation;

(*v*) Panchayati Raj;

(*vi*) Finance, Department of Financial Services; and

(*vii*) Defence,

members, *ex officio*;

(*h*) Chairman, Railway Board—member, *ex officio*;

(*i*) Director-General, Defence Estates— member, *ex officio*;

(*j*) representatives of not less than six State Governments and one Union territory, as the Central Government may, notify;

(*k*) not more than six social workers belonging to organisation working for the prohibition of manual scavenging and rehabilitation of manual scavengers, or, representing the scavenger community, resident in the country, to be nominated by the Chairperson, two of whom shall be women;

(*l*) Joint Secretary, Department of Social Justice and Empowerment in the Ministry of Social Justice and Empowerment, looking after development of Scheduled Castes—Member-Secretary, *ex officio*;

(*m*) such other representatives of Central Ministries or Departments and agencies which, in the opinion of the Chairperson, are concerned with the implementation of this Act.

(*3*) The Central Monitoring Committee shall meet at least once in every six months.

Functions of the Central Monitoring Committee.

30. The functions of the Central Monitoring Committee shall be,—

(*a*) to monitor and advise the Central Government and State Government for effective implementation of this Act and related laws and programmes;

(*b*) to co-ordinate the functions of all concerned agencies;

(*c*) to look into any other matter incidental to or connected with implementation of this Act.

31. (*1*) The National Commission for Safai Karamcharis shall perform the following functions, namely:— *Functions of National Commission for Safai Karamcharis.*

(*a*) to monitor the implementation of this Act;

(*b*) to enquire into complaints regarding contravention of the provisions of this Act, and to convey its findings to the concerned authorities with recommendations requiring further action; and

(*c*) to advise the Central and the State Governments for effective implementation of the provisions of this Act.

(*d*) to take *suo motu* notice of matter relating to non-implementation of this Act.

(*2*) In the discharge of its functions under sub-section (*1*), the National Commission shall have the power to call for information with respect to any matter specified in that sub-section from any Government or local or other authority.

32. (*1*) The State Government may, by notification, designate a State Commission for Safai Karamcharis or a State Commission for the Scheduled Castes or such other statutory or other authority, as it deems fit, to perform, within the State, *mutatis mutandis*, the functions specified in sub-section (*1*) of section 31. *Power of State Government to designate an appropriate authority to monitor the implementation of this Act.*

(*2*) An authority designated under sub-section (*1*) shall, within the State, have, *mutatis mutandis*, the powers of the National Commission for Safai Karamcharis as specified in sub-section (*2*) of section 31.

CHAPTER VIII

MISCELLANEOUS

33. (*1*) It shall be the duty of every local authority and other agency to use appropriate technological appliances for cleaning of sewers, septic tanks and other spaces within their control with a view to eliminating the need for the manual handling of excreta in the process of their cleaning. *Duty of local authorities and other agencies to use modern technology for cleaning of sewers, etc.*

(*2*) It shall be the duty of the appropriate Government to promote, through financial assistance, incentives and otherwise, the use of modern technology, as mentioned in sub-section (*1*).

34. No suit, prosecution or other legal proceeding shall lie against an appropriate Government or any officer of the appropriate Government or any member of the Committee for anything which is in good faith done or intended to be done under this Act. *Protection of action taken in good faith.*

35. No civil court shall have jurisdiction in respect of any matter to which any provision of this Act applies and no injunction shall be granted by any civil court in respect of anything, which is done or intended to be done, by or under this Act. *Jurisdiction of civil courts barred.*

36. (*1*) The appropriate Government shall, by notification, make rules for carrying out the provisions of this Act, within a period not exceeding three months from the date of commencement of this Act. *Power of appropriate Government to make rules.*

(*2*) In particular, and without prejudice to the generality of the foregoing power, such rules may provide for all or any of the following matters, namely:—

(*a*) the obligation of an employer, under clause (*d*) of sub-section (*1*) of section 2;

(*b*) the manner in which the excreta fully decomposes under clauses (*e*) and (*g*) of sub-section (*1*) of section 2;

(*c*) the manner of carrying out survey of insanitary latrine and publishing list thereof under clause (*a*) of sub-section (*1*) of section 4;

(*d*) procedure of giving notice and recovering cost of demolition of an insanitary latrine under sub-section (*3*) of section 5;

(*e*) content and methodology of the survey under sub-section (*2*) of section 11;

(*f*) the eligibility conditions for identification of manual scavengers and publication of provisional list of persons found to be working as manual scavengers under sub-section (*4*) of section 11;

(*g*) publication of final list of persons found to be working as manual scavengers under sub-section (*6*) of section 11;

(*h*) manner of application to be made to the Chief Executive Officer of the municipality, or to an officer authorised by him in this behalf, under sub-section (*1*) of section 12 or, as the case may be, sub-section (*1*) of section 15;

(*i*) provision of initial, one time, cash assistance under sub-clause (*ii*) of clause (*a*) of sub-section (*1*) of section 13;

(*j*) such other powers of Inspectors under clause (*e*) of sub-section (*2*) of section 20; and

(*k*) any other matter which is required to be, or may be, prescribed.

(*3*) Every rule made under this Act by the Central Government shall be laid, as soon as may be after it is made, before each House of Parliament, while it is in session, for a total period of thirty days which may be comprised in one session or in two or more successive sessions, and if, before the expiry of the session immediately following the session or the successive sessions aforesaid, both Houses agree in making any modification in the rule or both Houses agree that the rule should not be made, the rule shall thereafter have effect only in such modified form or be of no effect, as the case may be; so, however, that any such modification or annulment shall be without prejudice to the validity of anything previously done under that rule.

(*4*) Every rule made under this Act by the State Government shall, as soon as may be after it is made, be laid before each House of State Legislature, where there are two Houses and where there is one House of State Legislature, before that House.

Power of Central Government to make model rules.

37. (*1*) Notwithstanding anything contained in section 36 of this Act:—

(*a*) the Central Government shall, by notification, publish model rules for the guidance and use of State Governments; and

(*b*) in case the State Government fails to notify the rules under section 36 of this Act within the period of three months specified therein, then the model rules as notified by the Central Government shall be deemed to have come into effect, *mutatis mutandis*, in such State, till such time as the State Government notifies its rules.

(*2*) The model rules made by the Central Government under this Act shall be laid, as soon as may be after they are made, before each House of Parliament while it is in session, for a total period of thirty days which may be comprised in one session or in two or more successive sessions, and if, before the expiry of the session immediately following the session or the successive sessions aforesaid, both Houses make any modification in the rule, the rule shall thereafter have effect only in such modified form; so, however, that any such modification shall be without prejudice to the validity of anything previously done under that rule.

Power to remove difficulties.

38. (*1*) If any difficulty arises in giving effect to the provisions of this Act, the Central Government may, by order published in the Official Gazette, make such provisions, not inconsistent with the provisions of this Act, as may appear to it to be necessary or expedient for the removal of the difficulty:

Provided that no such order shall be made in relation to a State after the expiration of three years from the commencement of this Act in that State.

(*2*) Every order made under this section shall, as soon as may be after it is made, be laid before each House of Parliament.

39. (*1*) The appropriate Government may, by a general or special order published in the Official Gazette, for reasons to be recorded, and subject to such conditions as it may impose, exempt any area, category of buildings or class of persons from any provisions of this Act or from any specified requirement contained in this Act or any rule, order, notification, bye-laws or scheme made thereunder or dispense with the observance of any such requirement in a class or classes of cases, for a period not exceeding six months at a time.

Power to exempt.

(*2*) Every general or special order made under this section shall be laid, as soon as may be after it is made, before each House of Parliament or each House of State Legislature, where there are two Houses and where there is one House of State Legislature, before that House.

P.K. MALHOTRA,
Secy. to the Govt. of India.

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APPENDIX A 1.2
TIME FRAME FOR FULFILMENT OF RESPONSIBILITIES AND CERTAIN ACTIVITIES AND IMPLEMENTING AGENCIES

Table A1.2.1 Time frame for fulfilment of responsibilities and certain activities

No	Section under the Act	Subject Matter	Time period from the commencement of the Act	Responsibility
1	4(1)(a)	Carry out a survey of the insanitary latrines existing within its jurisdiction, and publish a list of such insanitary latrines	2 months	Local Authority
2	4(1)(b)	Give a notice to the occupier within 15 days from the date of publication of the list of insanitary latrines, to demolish and convert into a sanitary latrine	15 days from the publication of the list	Local Authority
3	4(1)(b)	(the occupier) to either demolish the insanitary latrine or convert it into a sanitary latrine	Within a period of 6 months (for sufficient reasons in recording, the local authority may extend the period by not exceeding another 3 months)	Occupier (Resident)
4	4(1)(c)	Construction of appropriate number of sanitary community latrines in the areas where insanitary latrines have been found	Within a period of note exceeding 9 months	Local Authority
5	4(2)	Construct adequate number of sanitary community latrines in the areas to eliminate open defecation	3 years from the date of commencement of the Act	Municipalities Cantonment Boards Railway Authorities
6	7	Not to engage or employ, either directly or indirectly, any person for hazardous cleaning of a sewer or septic tank	Within one year, as may be notified by the State Govt.	Person Local authority Agency
7	11(1)	Identification of manual scavengers	From the date of commencement of the Act	1 month for municipalities 2 months for municipal corporations

Every concerted effort should be taken so that there should be no further tolerance for pushing back the time frames for eradication of manual scavenging and failure to eradicate without reasonable cause beyond this would be contravention of the provisions of The Prohibition of Employment as Manual Scavengers and their Rehabilitation Act, 2013 and treated as an offence by public officials, with severe penalties.

Implementing Agencies

The implementing agencies under the Act should be the District Collector and Municipal Commissioner. As per the Section 18 of the Act, State Government shall confer such powers and impose such duties on local authority (Panchayat or municipal body) and District Magistrate as may be necessary to ensure that the provisions of this Act are properly carried out. It is incumbent upon the district magistrate or chief executive officer of the local authority to specify and confer such powers to the subordinate officers to perform all the duties as per the provisions of the Act. It is to be ensured by the authorities as notified by the appropriate government to ensure the following:

a. No person is engaged or employed as manual scavenger within their jurisdiction
b. No one constructs, maintains, uses or makes available for use, an insanitary latrine;
c. Identification and rehabilitation of the manual scavengers
d. Investigation and prosecution of persons contravening the provisions of the Act;

As per Section 33, sub-section 1 of the Act, “it shall be the duty of every local authority and other agency to use appropriate technological appliances for cleaning of sewers, septic tanks and other spaces within their control with a view to eliminating the need for manual handling of excreta in the process of their cleaning”. Sub-section 2 further specifies that, “it shall be the duty of the appropriate government to promote, through financial assistance, incentives and otherwise, the use of modern technology, as mentioned in sub-section 1”.

APPENDIX A 2.1
PERFORMANCE OF KNOWN SEWAGE TREATMENT PROCESSES UNDER INDIAN CONDITIONS

A study of pollution potential of water courses by sewage even after treatment was instituted by the Central Pollution Control Board and carried out by M/S Anna University on identified STPs in south India and M/S IIT Roorkee on identified STPs in north India. The results of performance of the STPs evaluated are reproduced here.

Table A 2.1.1 Summary of Results of Anna University
(Figures in Parameters are arithmetic average)

Treatment Plant	Total Coliform MPN/100 mL Influent X 10^6	Total Collform MPN/100 mL Effluent X 10^4	Prercent Reduct ion	Fecal Collform MPN /100 mL Influent X 10^6	Fecal Coliform MPN / 100 mL Effluent X 10^4	Percent Reduction
AABF	5 - 500	5 - 380	99.0 - 99.2	2.90 - 500	3.7 - 190	98.7 - 99.6
ASP	6.8 - 5,000	50 - 6,800	92.6 - 98.6	5 - 3,700	37 - 3,800	92.6 - 98.9
Tertiary Treated	0.68 - 98	<2 (all but one day)	97.1 - 99.9	1.1 - 68	<2 (all but one day)	98.2 - 99.9
OP	03.8 - 680	0.05 - 900	99.9-98.7	0.7 -98	0.038 - 500	99.9-94.4
TF	6.8 - 500	2 - 3,000	99.7-94	3.0 - 370	2-1.100	99.5-97.0

Table A 2.1.2 Annual performance of the five sewage treatment plants for Physico - Chemical characteristic

Characteristics		DEWATs		Nesapakkam		CPCL		Pondicherry		V. Valley	
		Raw	Treated	Raw	Treated	Raw	Treated	Raw	Treated	Raw	Treated
TSS (mg/L)	Min	288	17	450	33	67	2	93	19	168	31
	Max	1037	358	1482	534	328	164	730	286	764	299
	Avg	594	189	1059	234	206	23	397	154	383	106
	SD	233	123	318	169	85	42	229	86	223	96
COD-T (mg/L)	Min	272	53	519	56	104	3	235	98	277	69
	Max	1120	207	1656	288	421	160	800	240	560	181
	Avg	637	118	1232	128	230	57	497	171	429	115
	SD	256	57	275	59	88	46	177	54	92	44
COD-S (mg/L)	Min	128	24	149	37	66	0	64	32	75	37
	Max	352	133	635	203	261	107	480	208	240	117
	Avg	233	65	397	102	139	34	210	99	159	65
	SD	75	37	132	49	67	31	127	52	62	29
BOD-T (mg/L)	Min	206	19	236	15	36	0.8	77	36	74	31
	Max	749	138	671	90	174	65	356	123	320	75
	Avg	343	59	520	53	93	14	212	55	181	43
	SD	174	36	108	26	47	17	116	33	61	15
BOD-S (mg/L)	Min	80	14	98	5	18	0.4	37	15	35	15
	Max	344	70	379	73	88	37	217	45	185	57
	Avg	201	30	238	33	46	8	102	29	95	29
	SD	78	18	75	21	22	10	59	9	50	13

Table A 2.1.3 Annual performance of the five sewage treatment plants for microbiological characteristics

Characteristics		DEWATs		Nesapakkam		CPCL		Pondicherry		V. Valley	
		Raw	Treated	Raw	Treated	Raw	Treated	Raw	Treated	Raw	Treated
Total Coliforms	Min	500	5	680	50	68	0.0002	380	0.78	680	38
	Max	5000	380	500000	6800	9800	0.37	68000	3000	190000	5000
	Avg	5926	43	32152	1325	893	0.15	12492	565	21235	712
	SD	11125	65	77099	1691	1559	0.58	150501	587	38918	1183
Faecal Coliforms	Min	380	3.7	500	37	50	0.0002	380	0.78	500	19
	Max	5000	190	370000	5000	6800	0.98	38000	1100	68000	3800
	Avg	4555	28	22672	823	633	0.078	8506	371	9611	427
	SD	9887	38	57296	1034	1054	0.23	9787	321	14636	775
E.Coli	Min	190	0.68	98	18	38	0.0002	190	0.5	380	19
	Max	3800	98	180000	3800	3700	0.68	37000	980	38000	1900
	Avg	2569	15	12730	459	366	0.056	5387	243	4958	225
	SD	6301	21	29621	644	631	0.17	6540	237	7802	395
Faecal Streptococci	Min	13	0.29	68	5	1.3	0.0002	13	0.5	78	1.1
	Max	1300	3.8	5000	180	680	0.19	980	68	980	50
	Avg	2691	1.2	1373	51	95	0.016	393	10	317	10
	SD	393	1.2	1593	47	185	0.048	326	18	305	17

Table A 2.1.4 Summary Of Results of IIT, Roorkee (Figures in parameters are arithmetic average)

Treatment Plant	Total Coliform MPN/100 mL Influent X 10^6	Total Collform MPN/100 mL Effluent X 10^4	Percent Reduct ion	Fecal Collform MPN /100 mL Influent X 10^6	Fecal Coliform MPN / 100 mL Effluent X 10^4	Percent Reduction
UASB	2.3 - 23	1.9 - 23	99.2 - 99	0.23 - 15	0.023 - 23	99.9 - 98.5
UASB	1.6 - 43	1.1 - 23	99.3 - 99.5	0.39 - 23	0.11 - 4-3	99.7 - 99.8
OP	4.3 - 930	1.5 - 430	99.7 - 99.5	0.023 - 230	0.15 - 210	99.3 - 99.1
ASP	1.5 - 4.3	9.3 - 930	93.8	0.21 - 1.5	0.15 - 930	99.3
Anaerobic Filter	28	450	83.9	6.8	130	80.9

Table A 2.1.5 Sewage Treatment Plants in Delhi

Sl.No	Name of the STP's & Capacity (mgd)	Design capacity (MLD)	Actual flow (MLD)	Type of STP	Present Status
1	Coronation Pillar STP's 1) (10) 2) (10 + 20)	 45.46 45.46 90.92	 40.87 63.46 56.55	Activated sludge process (ASP), trickling filter & ASP	Under utilised Over the Des. Cap. Under Utilized
2.	Delhi Gate (2.2)	10.00	10.00	High rate bio-filters Densadeg technology	Running on designed capacity
3.	Ghitorni (5)	22.73	Nil	Activated sludge process	Not in operation
4.	Keshopur STPs 1) (12) 1) (20) 2) (40)	 54.55 90.92 181.84	 46.55 95.10 106.46	All the three plants designed on activated sludge process	i)12 mgd not running, sewage passes through PST. ii) Over the Des. Cap. iii) Under- utilized
5.	Kondli STP's 1) (10-Phase-I) 2) (25 -Phase-II) 3) (10-Phase-III)	 45.46 113.65 45.46	 56.55 57.96 28.36	All three activated sludge process	Over the capacity Under- utilized Under- utilized
6.	Mehrauli STP (5)	22.73	4.95	Extended aeration	Under-utilized
7.	Najafgarh STP (5)	22.73	2.27	Activated sludge proc.	Under- utilized
8.	Nilothi STP (40)	181.84	15.0	Activated sludge process	Under- utilized
9.	Narela STP (10)	45.46	2.50	Activated sludge process	Under- utilized
10.	Okhla STP's 1) (12) 2) (16) 3) (30) (37) (45)	 54.55 72.73 136.38 168.20 204.57	 39.09 40.91 136.98 159.11 181.84	All the plants designed on activated sludge process	Under- utilized Under- utilized Running in cap. Under-utilized Under-utilized
11.	Papankalan STP (20)	90.92	37.73	Activated sludge process	Under-utilized
12	Rithala STP's 1) (40) Old 2) (40) New	 181.84 181.84	 46.28 185.07	Activated sludge process & High rate aerobic ASP & biofor/biofilter	Under-utilized Over the des. cap.
13.	Rohini STP (15)	68.19	Nil	Activated sludge process	Not in operation
14.	Sen N.H. STP (2.2)	10.0	10.0	High rate Bio filter	Running on designed capacity.
15.	Timarpur O.P. (6)	27.27	4.79	Oxidation ponds	Under-utilized
16.	Yamuna Vihar STP's 1) Ph-I(10) 2) Ph-II(10)	 45.46 45.46	 27.27 14.77	Activated sludge process	Under-utilized Under-utilized
17.	Vasant Kunj STP's 1) (2.2) 2) (3.0)	 10.00 13.63	 3.18 4.36	ASP & Extended aeration	Under-utilized Under-utilized
Total	**30**	**2330**	**1478**		

Table A 2.1.6 Performance Evaluation of Sewage Treatment Plants in Delhi

Sl. No.	Name of STP & Capacity (mgd)	Design Capacity , M LD	Actual flow, MLD	Performance Evaluation of STP' (24 Hour composite Monitoring for every three hourly samples)										% reduction		
				Influent Quality					Effluent Quality							
				pH	TSS	COD	BOD	Cond.	pH	TSS	COD	BOD	Cond.	TSS	COD	BOD
1	Cor. Pillar(10)	45.46	40.87	7.2	179	317	112	908	7.4	35	61	18	1090	80.45	80.76	83.93
	(20+10)	136.38	120.01	6.44	342	172	48	1700	6.9	93	48	15	1730	72.81	72.09	68.75
2	Keshopur(12*)	54.55	46.55	-	-	-	-	-	-	-	-	-	-	-	-	-
	(20)	90.92	95.1	7.3	404	560	282	1390	7.6	78	149	45	1390	80.69	73.39	84.04
	(40)	181.84	106.46	7.3	404	560	282	1390	7.8	21	55	10	1520	94.80	90.18	96.45
3	Okhla(12)	54.55	39.09	7.3	498	517	204	1440	7.8	21	54	10	1460	95.78	89.56	95.10
	(16)	72.73	40.91	7.4	291	486	207	1510	7.7	83	108	48	1400	71.48	77.78	76.81
	(30)	136.38	136.98	7.4	647	551	222	1480	7.6	76	153	45	1470	88.25	72.23	79.73
	(37)	168.2	159.11	7.3	480	515	249	1590	7.8	32	62	12	1540	93.33	87.96	95.18
	(45)	204.57	181.84	7.3	480	515	249	1590	7.7	27	51	19	1530	94.38	90.10	92.37
4	Narela (10)	45.46	2.5	7.4	426	447	100	1720	8	38	72	8	1720	91.08	83.89	92.00
5	Y. Vihar (Ph.-I 10,	45.46	27.27	7.1	391	505	174	1110	7.7	44	84	17	1050	88.75	83.37	90.23
	Ph.-II 10)	45.46	14.77	7.2	405	538	199	1020	7.5	39	44	20	1070	90.37	91.82	89.95
6	Timarpur O.P -(6)	27.27	4.79	6.7	412	272	106	1650	7.3	11	26	4	1650	97.33	90.44	96.23
7	Najafgarh (5)	22.73	2.27	7.4	165	205	54	810	7.7	29	38	1	687	82.42	81.46	98.15
8	Nilothi (40)	181.84	15	7.7	432	328	90	2340	7.8	21	26	4	1960	95.14	92.07	95.56
9	Dr. SenN.H.(2.2)	10	10	7.5	370	585	236	1680	7.4	36	46	16	1660	90.27	92.14	93.22
10	Delhi Gate (2.2)	10	10	7.5	263	605	147	1020	7.3	26	62	20	1030	90.11	89.75	86.39
11	Papankalan (20)	90.92	37.73	7.6	142	275	103	2190	7.9	39	46	10	1580	72.54	83.27	90.29
12	Kondli Ph.-I (10)	45.46	56.55	7.3	363	507	241	1390	7.8	68	140	27	1390	81.27	72.39	88.80
	Ph.-II (25)	113.65	57.96	7.3	604	588	261	1550	7.6	45	50	34	1350	92.55	91.50	86.97
	Ph.-III (10)	45.46	28.36	7.3	519	615	237	1530	7.8	16	50	14	1220	96.92	91.87	94.09
13	Mehrauli(5)	22.73	4.95	7.8	251	326	126	1090	8.1	12	35	7	1180	95.22	89.26	94.44
14	Rithala [(40 Old)	181.84	46.28	7.2	330	399	205	1260	7.5	75	54	14	1240	77.27	86.47	93.17
	(40 New)]	181.84	185.07	7.2	330	399	205	1260	7.3	47	151	55	1230	85.76	62.16	73.17
15	Vasant Kunj (2.2)	10	3.18	7.5	379	460	323	1710	7.8	23	43	7	1450	93.93	90.65	97.83
	(3)	13.63	4.36	7.4	479	565	306	1400	7.9	49	80	20	1470	89.77	85.84	93.46
16	Rohini (15)	68.19	Nil	-	-	-	-	-	-	-	-	-	-	-	-	-
17	Ghitorni (5)	22.73	Nil	-	-	-	-	-	-	-	-	-	-	-	-	-
Total		2330	1478													

- Keshopur 12 mgd STP is not running fully, it is observed that the sewage passes though Primary Settling Tank. All values are in mg/1 except pH and conductivity (μ mhos / cm)

Table A 2.1.7 Performance of Bacteriological Reduction in Sewage Treatment Plants in Delhi

Sl. No.	Name of the STP & capacity (mgd)	Performance evaluation of Sewage Treatment Plants in Delhi				% Reduction	
		Influent Bacteriological Quality		Effluent Bacteriological Quality			
		Total Coliform (Nos/100ml)	Faecal Coliform (Nos/100ml)	Total Coliform (Nos/100ml)	Faecal Coliform (Nos/100ml)	Total Coliform	Faecal Coliform
1.	Najafgarh (5)	10900000	5100000	320000	120000	**97.06**	**97.65**
2.	Papankala (20)	13100000	10300000	120000	70000	**99.08**	**99.32**
3.	Delhi gate (2.2)	26000000	19000000	1700000	1100000	**93.46**	**94.21**
4.	Dr. Sen N. H. (2.2)	133000000	102000000	240000	21700	**99.82**	**99.98**
5.	Nilothi (40)	61000000	50000000	120000	70000	**99.80**	**99.86**
6.	Cor. Pillar (10)	39000000	32000000	200000	110000	**99.49**	**99.66**
7.	Cor. Pillar (30)	78000000	44000000	700000	200000	**99.10**	**99.55**
8.	Narela (5)	17000000	10000000	110000	40000	**99.35**	**99.60**
9.	Vasant Kunj (3)	6900000	3900000	178000	101000	**97.42**	**97.41**
10.	Vasant Kunj (2.2)	71000000	46000000	17000	8000	**99.98**	**99.98**
11.	Okhla (12)	370000000	65000000	2900000	230000	**99.22**	**99.65**
12.	Okhla (16)	51000000	27000000	990000	530000	**98.06**	**98.04**
13.	Okhla (30)	204000000	107000000	115000000	25000000	**43.63**	**76.64**
14.	Okhla (37)	197000000	111000000	1280000	710000	**99.35**	**99.36**
15.	Okhla (45)	197000000	111000000	4100000	600000	**97.92**	**99.46**
16.	Y. Vihar (Ph.-I 10)	1210000000	410000000	19400000	4600000	**98.40**	**98.88**
17.	Y. Vihar (Ph.-II 10)	1570000000	370000000	8500000	5200000	**99.46**	**98.59**
18.	Keshopur (20)	430000000	135000000	91000000	7200000	**78.84**	**94.67**
19.	Keshopur (40)	430000000	135000000	11500000	5100000	**97.33**	**96.22**
20.	Kondli (Ph.-I 10)	670000000	320000000	24000000	13900000	**96.42**	**95.66**
21.	Kondli (Ph.-II 25)	910000000	480000000	5500000	1800000	**99.40**	**99.63**
22.	Kondli (Ph.-III 10)	570000000	370000000	2700000	140000	**99.53**	**99.96**
23.	Rithala (40 Old)	1080000000	710000000	32000000	4600000	**97.04**	**99.35**
24.	Rithala (40 New)	1080000000	710000000	49000000	5900000	**95.46**	**99.17**
25.	Mehrauli (5)	290000000	210000000	490000	20000	**99.83**	**99.99**
26.	Dr. Sen N.H (2.2) (After Bio-filter)	133000000	102000000	179000	13500	**99.87**	**99.99**
27.	Dr. Sen N.H (2.2) (After U.V. treatment)	133000000	102000000	27000	11200	**99.80**	**99.99**

The values reported for the CPCL plant in Table A 2.1.3 relate to the quality of recovered water after the secondary treated sewage is further treated by chemical coagulation, sedimentation, filtration and R O.

The performance of oxidation ponds one in the south coastal temperate climate at Puducherry and the other at north in the cold hilly region of Rishikesh have also been evaluated in this study. In the Puducherry ponds, removals were 93% for total BOD and from 37,000/100 ml to 500/100 ml for faecal coliforms. In the Rishikesh pond system, the removals were 82% for total BOD and from 6000000/100 ml to 28000/100 ml for faecal coliforms.

The faecal coliform reduction from 3000/100 ml and 70,000/100 ml to almost nil have been documented in the study when secondary treated sewage was chemically coagulated with alum or Iron salts and chlorinated with of 3 to 4 mg/l as in Figure A 2.1.1

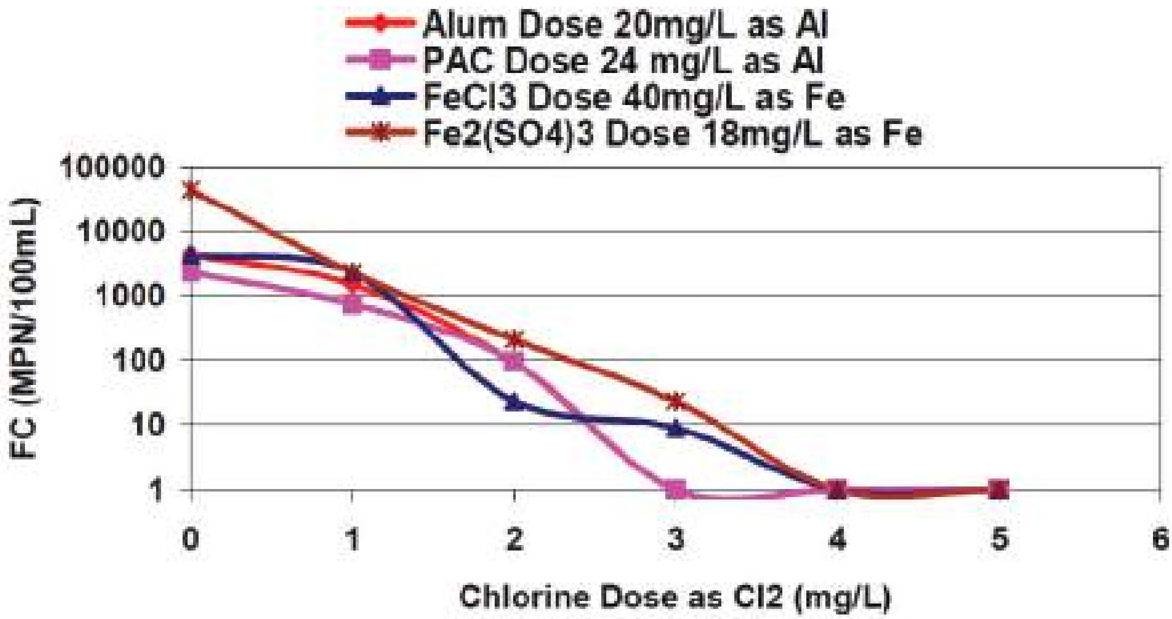


Figure A 2.1.1 Removal of FC with chlorination after coagulation with different coagulants at optimum doses.

The performance results do not cover the later day STP processes such as SBR, MBBR and MBR etc as these STPs were not in popular use at that time. All the same, the studies establish the position that the same STP process exhibits varying degrees of removals under varying conditions of climate, capacity utilization, nature of inlet sewage etc and almost all processes can be expected to attain a required degree of removal of BOD and faecal coliforms provided the design takes into account the insights as brought out above.

APPENDIX A 2.2

ESTIMATION OF FUTURE POPULATION

(Retained as in 2nd edition)

1.PROBLEM

Assuming that the scheme of water supply will commence to function from 1986, it is required to estimate the population 30 years hence, i.e. in 2016 and also the intermediate population 15 years after 1986, i.e. in 2001.

Table A2.2-1

Year	Population	Increment
1921	40,185	---
1931	44,522	15,873
1941	60,395	15,873
1951	75,614	15,219
1961	98,886	23,272
1971	124,230	25,344
1981	158,800	34,570
Total		118,615
Average		19,769

2. SOLUTION

2.1 ARITHMETIC PROGRESSION METHOD

Increase in population from 1921 to 1981

i.e. in 6 decades = 1,58,800

(-) 40,185

= 1,18,615

= 1,58,800 - 40,185 = 1,18, 615

or increase per decade = 1/6 × 1,18,615 = 19,769

Population in 2001 = Population in 1981 + Increase for 2 decades

= 1,58,800 + 2 × 19,769

= 1,58,800 + 39,538 = 1,98,338

Population in 2016 = Population in 1981 + Increase for 3.5 decades

= 1,58,800 + 3.5 × 19,769 = 2,27,992

2.2 GEOMETRIC PROGRESSION METHOD

Rate of growth (r) per decade between

1931 and 1921	=	4,337/40,185	=	0.108
1941 and 1931	=	15,873/44,522	=	0.356
1951 and 1941	=	15,219/60,395	=	0.252
1961 and 1951	=	23,272/75,614	=	0.308
1971 and 1961	=	25,344/98,886	=	0.256
1981 and 1971	=	34,570/1,24,230	=	0.278

Geometric Mean,

$$r_g = \sqrt[6]{0.108 \times 0.356 \times 0.252 \times 0.308 \times 0.256 \times 0.278} \quad = \quad 0.2442$$

Assuming that the future growth follows the geometric mean for
the period 1921 to 1981 rg = 0.2442

Population in 2001 = Population in 1981 × $(1 + rg)^2$

= 1,58,800 × $(1.2442)^2$ = 2,45,800

Population in 2016 = Population in 1981 × $(1 + rg)^{3.5}$

= 1,58,800 × $(1.2442)^{3.5}$ = 3,41,166

2.3 METHOD OF VARYING INCREMENT OR INCREMENTAL INCREASE METHOD

In this method, a progressively decreasing or increasing rather than a constant rate is adopted. This is a modification over the Arithmetical Progression method.

Table A2.2-2

Year	Population	Increase (X)	Incremental Increase (Y)
1921	40,185		
1931	44,522	4,337	
1941	60,395	15,873	11,536
1951	75,614	15,219	- 654
1961	98,886	23,272	8,053
1971	124,230	25,344	2,072
1981	158,800	34,570	9,226
Total		1,18,615	30,233
Average		1/6 × 118,615 = 19,769	1/5 × 30,233 = 6,047

Population can be projected using the formula:

$$P_n = P_1 + nY + \frac{n(n + 1)Y}{2}$$

Therefore, population in 2001 can be given as

$$P_{2001} = P_{1981} + 2 \times 19769 + \frac{2 \times 3 \times 6047}{2}$$

= 1,58,800 + 39,538 + 18,141
= 2,16,479

$$P_{2016} = P_{1981} + 3.5 \times 19769 + \frac{3.5 \times 4.5 \times 6047}{2}$$

Similarly, population in 2016 can be given as
= 1,58,800 + 69,192 + 24,188
= 2,75,612

2.4 GRAPHICAL PROJECTION METHOD

From the Figure presented on the following page, the figures for 2001 and 2016 years obtained are as follows:
2001 - 253,000
2016 - 362,000

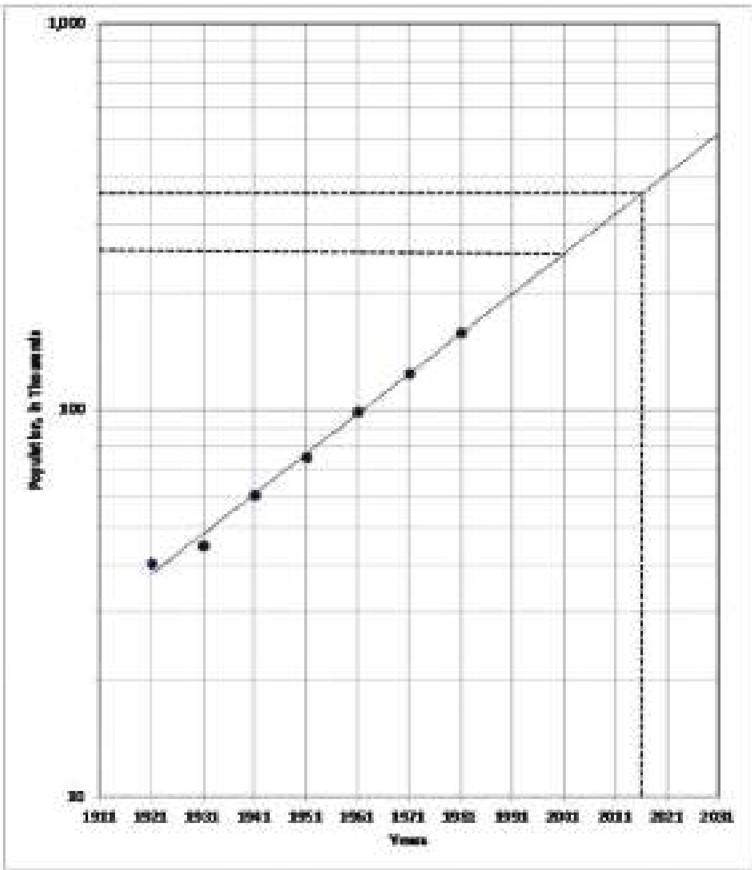


Figure A2.2-1 Semi log graph for estimation of future population

APPENDIX A 3.1
COMPUTATION OF STORM RUNOFF AND DESIGN OF STORM SEWER
(Retained as in 2nd edition)

PROBLEM

Design a system of storm sewers for the area shown in Figure A3.1-1 based on the Rational Formula for the estimation of peak runoff.

Basic Data and Assumptions imperviousness

Built up and paved area	-	0.7
Open space, lawns, etc.	-	0.2
Inlet time		
Built up and paved area (t_b)	-	8 minutes.
Open space, lawns (t_1)	-	15 minutes.
Minimum velocity in sewer	-	0.8 mps
Minimum depth of cover above crown	-	0.5 metres.

Rainfall intensity = consider one year storm as the area is central and high priced. (Use Table 3.7 for the record of rainfall intensity and frequency of rainfall). Use Manning's chart for sewer design.

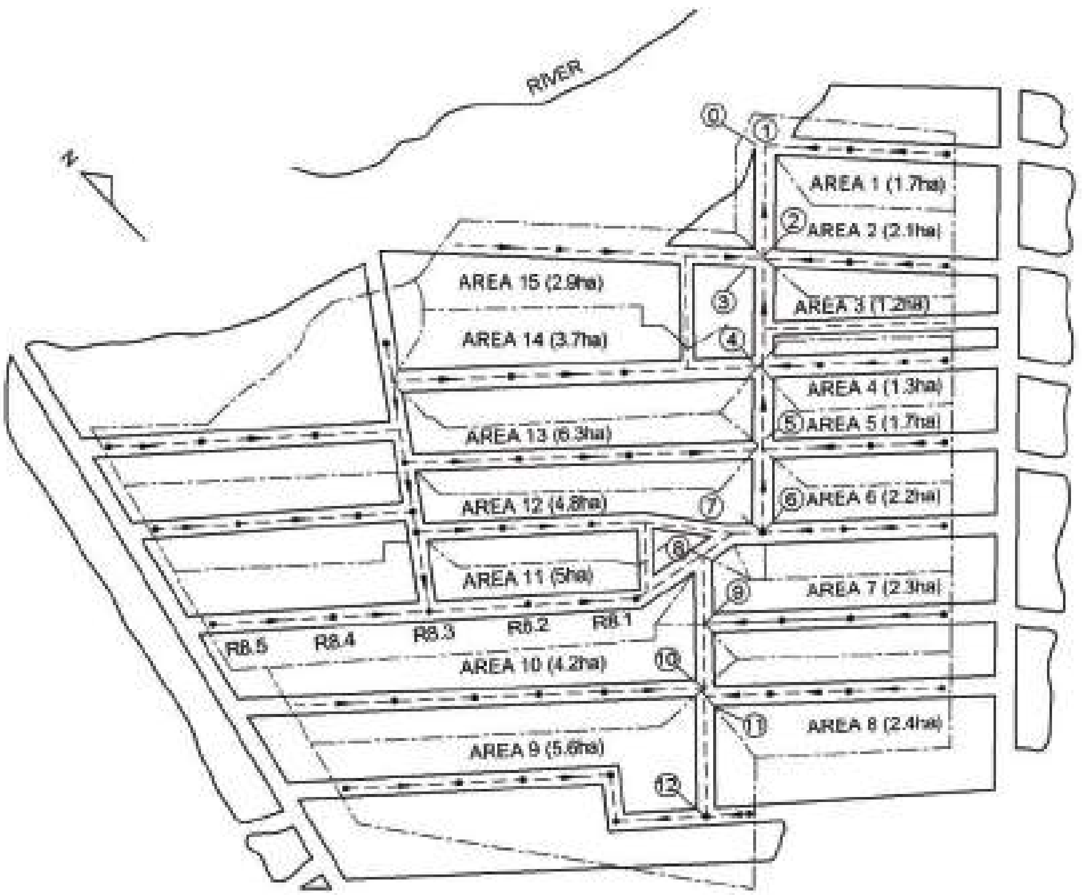


Figure A3.1-1

1 SOLUTION

Quantity of storm water runoff is calculated using the Rational Formula given in Section 3.9.2.1. i.e.]

Q = 10 C i A

Where,

Q : Runoff in m^3/hr

C : Coefficient of runoff

i : Intensity of rainfall in mm/hr and

A : Area of drainage district in hectares

Storm water runoff is determined in the following manner.

I) From the rainfall records for the last 26 years (Table 3.7), the storm occurring once in a year, i.e. 26 times in 26 years, the time-intensity values for this frequency are obtained by interpolation and are as follows;

Intensity, i mm/hr	30	35	40	45	50	60
Duration, t minute	44	36	28.5	22.5	13.5	9.75

II) The generalised formula adopted for intensity and duration is

$$i = \frac{a}{t^n}$$

Where,

i : Intensity of rainfall in mm/hr

t : Duration in minutes and a and *n* are constants

A graph Figure A3.1-3 is plotted for one year storm using the values i and t from the above table on a log-log paper. From the line of best fit the values of a and n are found out. From the plotted line values of a and n are 160 and 0.4 respectively.

iii) Now using equation $i = (160 / t^{0.4})$, i.e. after substituting the values of a and n for different values of i for various values of t are calculated and tabulated as below and a curve (Figure A3.1-4) is plotted on an ordinary graph paper.

The table for intensity-duration curve for one year storm is given in Table A 3.1-1

Table A3.1-1

t_{min}	5	10	15	20	25	30	35	40	45	60	80	100	120
$i = a/t^n$	84.2	64.0	54.0	48.5	44.2	41.2	38.6	36.8	34.8	31.0	27.8	25.4	23.6

iv) Another graph (Figure A3.1-5) of runoff-coefficient C vs. duration time t is plotted as per values given in Table 3.8 (Hornet's Table).

v) From the above two graphs (Figure A3.1-4 and Figure A3.1-5) the values of C and i for the same duration time t are determined and the curves for 10 C i vs t for the various values of imperviousness are plotted (Figure A3.1-6). The value of 10 C i gives the rate of runoff in m^3/hr per hectare of the tributary area. These curves are ultimately used in calculating the runoff from the tributary areas for a given time of concentration and imperviousness factor.

2 DESIGN OF STORM SEWER SYSTEM

Table A3.1-2 gives the various components of the storm sewer system design.

Column 1-4 identify the location of drain, street and manholes.

Columns 5-6 record the increment in tributary area with the given imperviousness factors.

Column 7 gives the tributary area increment with equivalent 100 percent imperviousness factor.

Column 8 records the total area served by each drain.

Column 9 records the time of concentration at each upper end of line (drain).

The time of concentration is found by taking the weighted average of the two areas. i.e.,

$$t_c = \frac{A_1 t_0 + A_2 t_1}{A_1 + A_2}$$

Where,

A_1: Built up area
A_2 : Area of lawns

Column 10 records the time of flow in each drain. For example the time of flow in line 1 is calculated to be 70 / (60 ×1.0) = 1.17 min.

Column 11 is the total time of concentration for each drain.

Column 12 is the value of runoff as 10 C i read from the Figure A3.1-6 for the corresponding time of concentration.

Column 13 gives the total runoff from each tributary area.

Column 14 gives the runoff in lps from each tributary area.

Columns 15-18 record the chosen size, required grade resulting capacity, velocity of flow for each drain or line. These designs of storm sewers are computed from the Manning's chart for each required flow and maintaining a minimum velocity.

Columns 19-23 identify the profile of the drain.

Column 19 is taken form the plan

Column 20 = Col.19 ×Col.16

Column 21 the required drop in manholes is obtained directly from the recommended values in section 3.17.1.

Column 22 gives invert elevation at the upper end with minimum cover of 0.6m at starting manhole.

Table A3.1-2

Line Number	Location of drain			Tributary area (hectares) increment				t_c Time of Concentration			Runoff m^3/hr		Flow Q	Design						Profile		
	Street	Manhole from	Manhole to	0.7 Imp factor	0.2 Imp factor	Eq 100% Imp factor	Total Area	Time of (t_i) inlet to upper end	Time of flow in drain t_t	TOTAL t_c= t_i+t_t	Per hectare (10CI)	Total	lps	Dia mm	Slope m/1000	Capacity lps	Velocity mps	Length m	fall m	Drop in Manhole	Invert Elevation Upper end	Invert Elevation Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	South st	5	4	0.286	0.366	0.274	0.274	12.0	-	12.0	345	94.5	26.6	200	10.0	32	1.0	70	0.7	0.000	37.400	36.700
2		4	3	0.167	0.488	0.214	0.488	13.3	1.17	14.47	335	164.0	46.0	250	6.65	50	1.0	125	0.83	0.025	36.875	35.845
3	North south St.2	R 32	R 3.1	0.415	0.312	0.352	0.352	11.0	-	11.0	348	123.0	35.0	250	6.65	50	1.0	70	0.47	0.000	36.700	36.230
4		R 31	3	0.358	0.36	0.324	0.676	11.5	1.17	12.67	340	264.0	74.0	350	4.55	98	1.0	70	0.32	0.050	36.180	35.860
5	South St.	3	2	0.256	0.466	0.274	1.438	12.5	3.27	15.77	335	480.0	135.0	450	3.14	160	1.0	125	0.40	0.066	35.779	35.379
6	North south St.3	R2.2	R 2.1	0.230	0.492	0.260	0.260	12.8	-	12.8	340	87.5	25.0	200	10.0	32	1.0	70	0.70	0.000	38.000	37.300
7		R 21	2	0.410	0.310	0.348	0.608	11.0	1.17	12.17	342	208.0	59.0	300	5.55	70.	1.0	70	0.39	0.050	37.250	36.860
8	South St.	R 2	1	0.256	0.466	0.274	2.320	12.5	5.37	17.87	330	765.0	214.0	600	2.22	280	1.0	160	0.36	0.200	35.179	34.819
9	North south St.4	R 12	R1.1	0.660	0.282	0.517	0.517	10.2	-	10.2	350	182.0	51.0	250	10.0	60	1.25	70	0.70	0.000	36.800	36.100
10		R 11	1	0.580	0.362	0.479	0.996	10.8	0.94	11.74	344	330.0	92.0	350	5.0	100	1.1	70	0.35	0.050	36.050	35.700
11	South St.	1	Pump house	0.670	0.330	0.494	3.810	10.4	8.05	18.45	325	1240.0	345.0	700	1.67	400	1.0	25	0.42	0.234	34.585	34.165

Thus for lines 1, 3, 6 and 9, the invert elevations are respectively 37.400, 36.700, 38.000 and 36.000. In case a manhole having more than one inlet, the drop in the manhole is considered with respect to the lowest invert level of the inlets to fix the invert level of the outlet.

Column 23 = Col.22 - Col.20 = invert elevation at the lower end of the line.

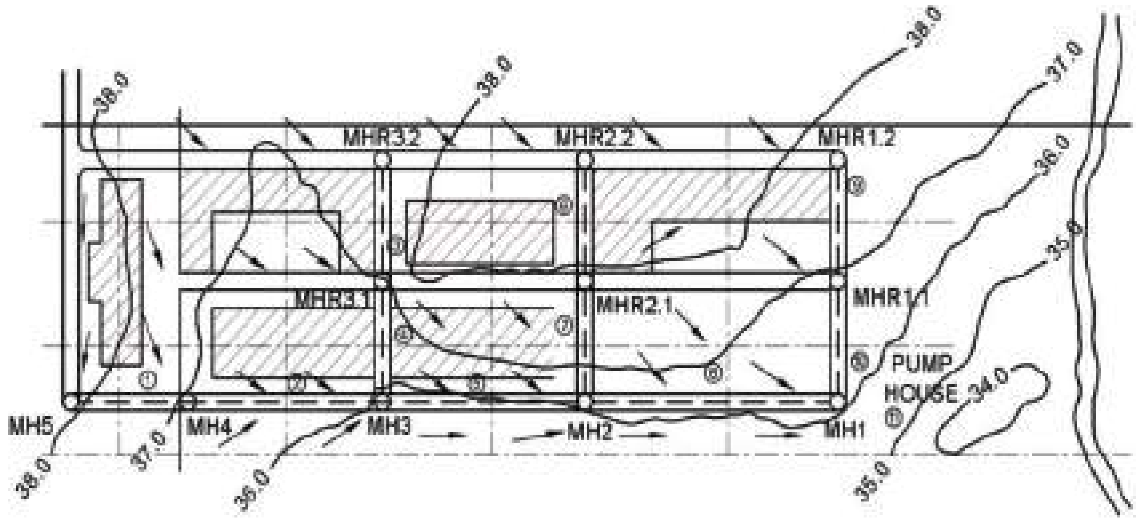


Figure A3.1-2

INTENSITY "i" (mm / hr)	30	35	40	45	50	60
DURATION "t" (minutes)	44	36	28.5	22.5	13.5	9.75

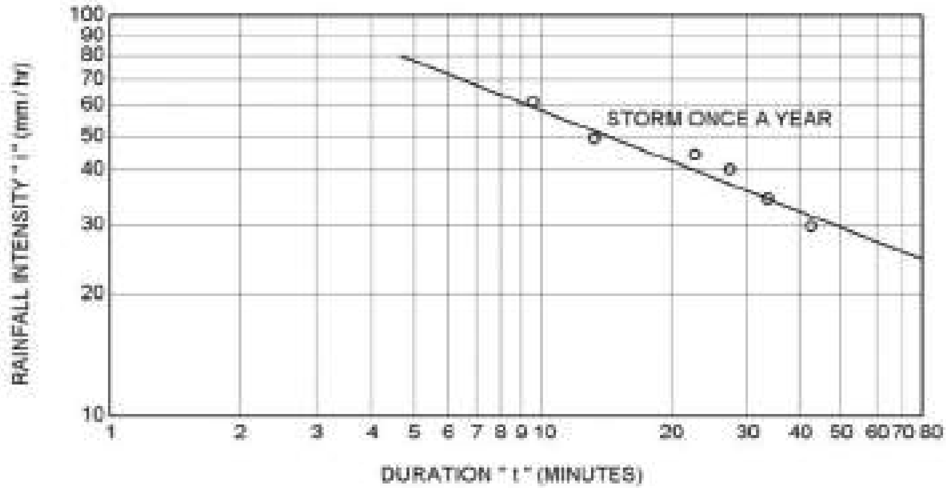


Figure A3.1-3

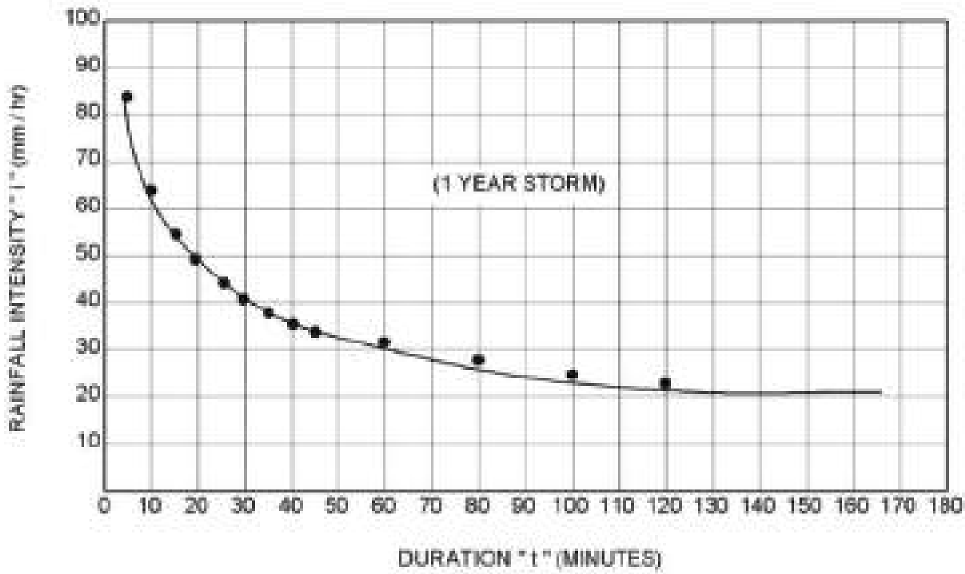


Figure A3.1-4

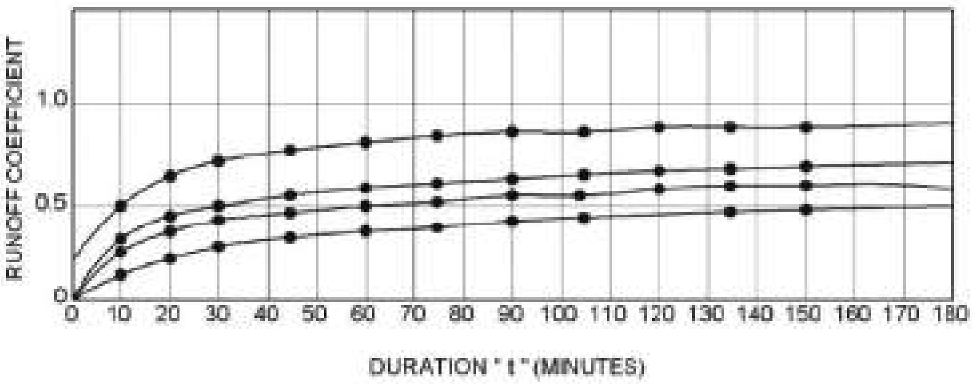


Figure A3.1-5 After horner area rectangle

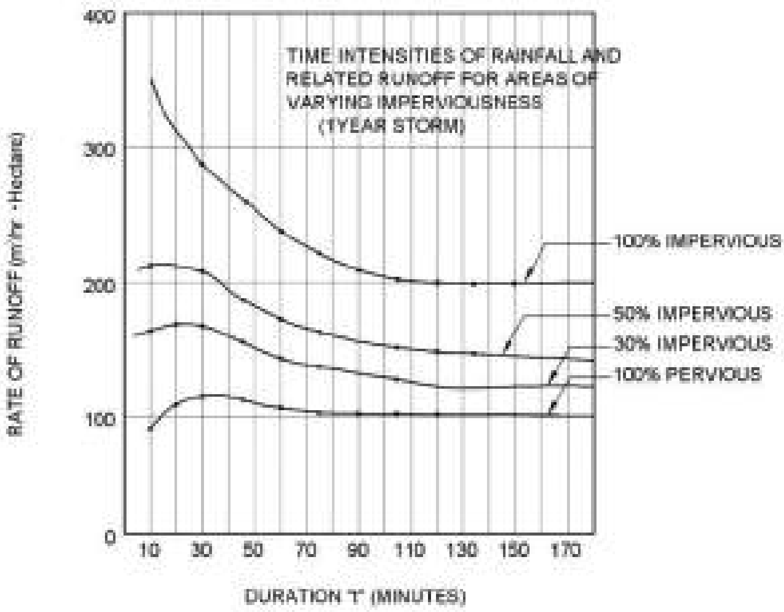


Figure A3.1-6

APPENDIX A 3.2 A
NOMOGRAM FOR MANNING'S FORMULA FOR GRAVITY SEWERS FLOWING FULL AND MANNING'S N VALUE OF 0.013.
(For discharges from 100 lpm to 100000 lpm)

For other values of Manning's n, the velocity and discharge will be inversely proportional.
Example-Find the discharge and velocity of a sewer flowing full of diameter 200 mm, slope of 1 in 200 and a Manning's n value of 0.0125.

Answer-From the nomogram, V = 0.75 m/s and discharge = 1,300 lpm. For n value of 0.0125,
V = 0.75 × 0.013/0.0125 = 0.78 m/s & discharge = 1,300 × 0.013/0.0125 = 1,352 lpm

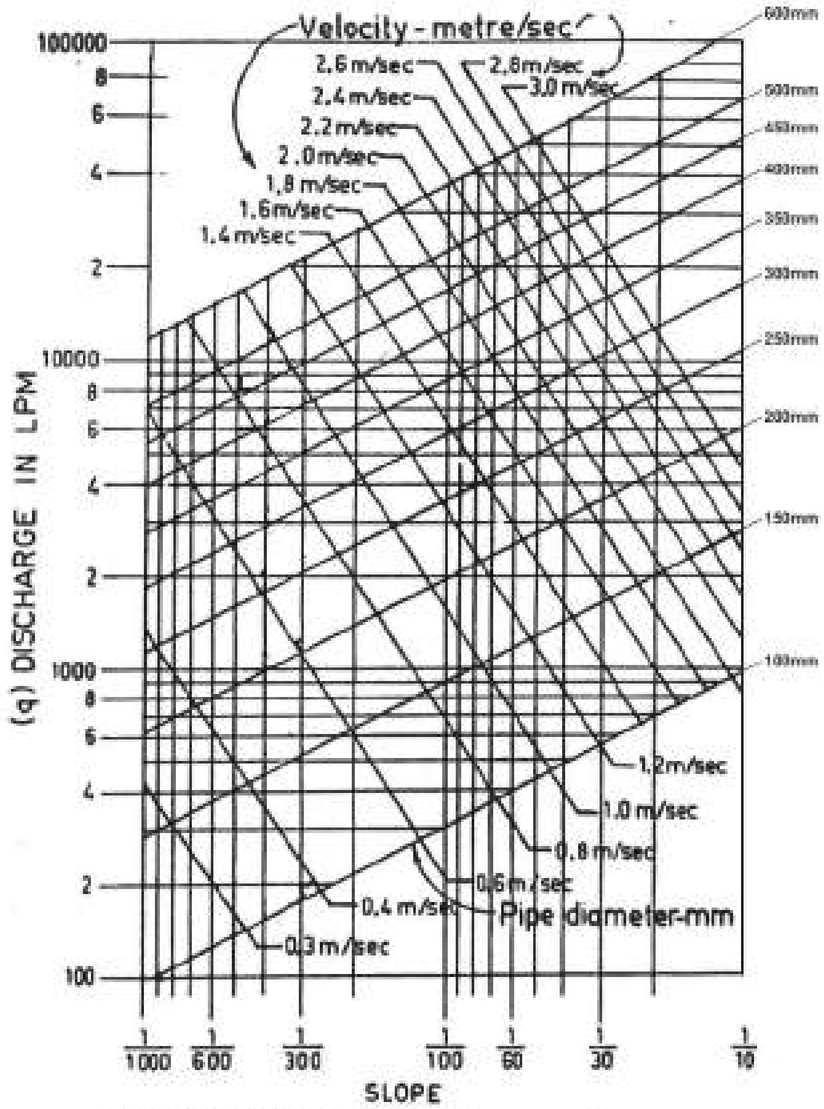


FIG 6 NOMOGRAM CHART FOR MANNING'S FORMULA (n = 0.013) FOR q = 100 lpm TO 100000 lpm

Figure A3.2A-1

Source-"Data Matrix for Public Health Engineers", TWAD Board, 1991

APPENDIX A 3.2 B

NOMOGRAM FOR MANNING'S FORMULA FOR GRAVITY SEWERS FLOWING FULL AND MANNING'S N VALUE OF 0.013

(For discharges from 1000 lpm to 1000000 lpm)

For other values of Manning's n the velocity and discharge will be inversely proportional.
Example-Find the discharge and velocity of a sewer flowing full of diameter 900 mm, slope of 1 in 1,000 and a Manning's n value of 0.012.

Answer-From the nomogram, V = 0.90 m/s and discharge = 35,000 lpm. For n value of 0.0125,
V = 0.90 × 0.013/0.0125 = 0.94 m/s & discharge = 35,000 × 0.013/0.0125 = 36,400 lpm

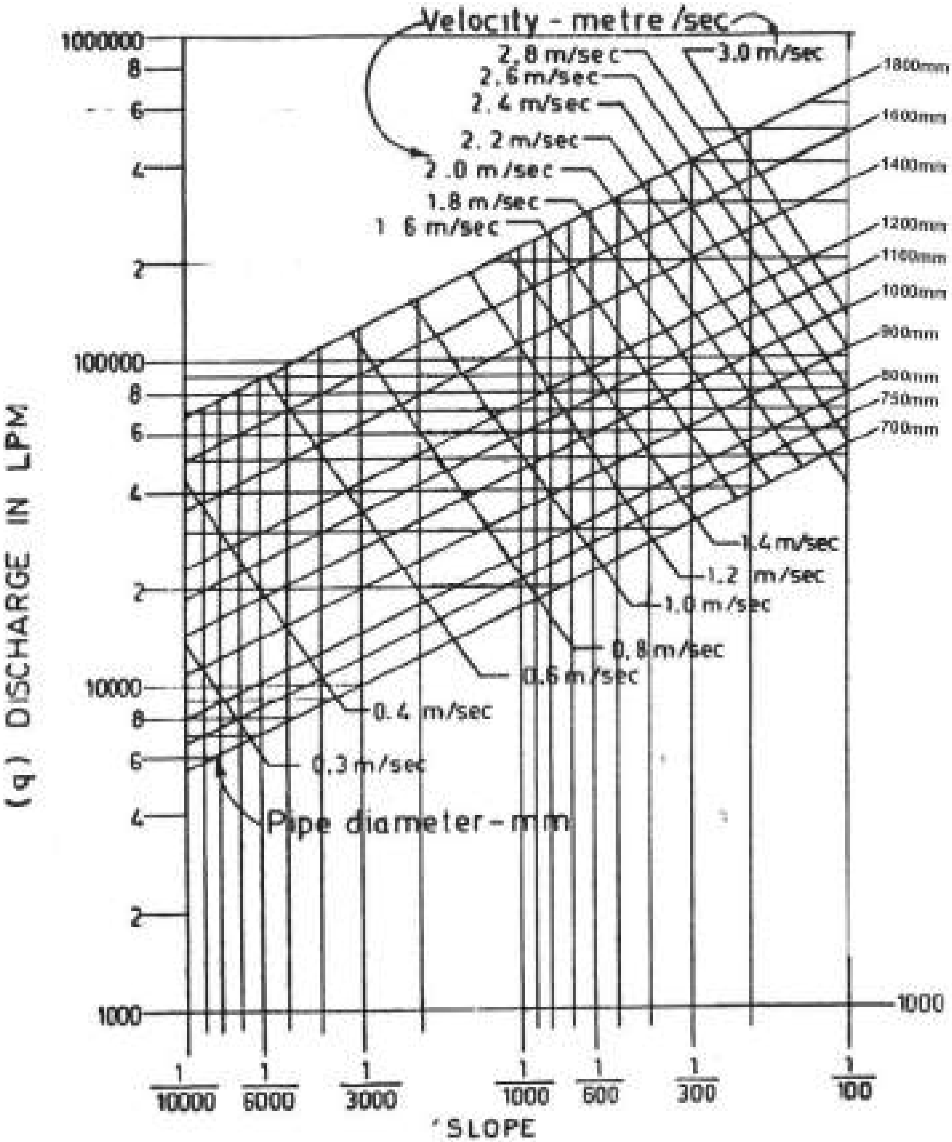


FIG 7 NOMOGRAM CHART FOR MANNING'S FORMULA

Figure A3.2B-1

Source-"Data Matrix for Public Health Engineers", TWAD Board, 1991

APPENDIX A 3.3
MS EXCEL FOR MANNING FORMULA FOR CIRCULAR GRAVITY PIPES FLOWING FULL

No	A	B	C
1	Appendix 3.3		
2	MS Excel For Manning Formula for Circular Gravity Pipes Flowing Full		
3	Given diameter and slope of sewer, find discharge and velocity		
4	This is left blank for the designer to enter his design notes		
5	This is left blank for the designer to enter his design notes		
6	This is left blank for the designer to enter his design notes		
7	diameter, mm	200	Enter by Designer
8	slope 1 in	250	Enter by Designer
9	value of Manning's n	0.013	Enter from Table 3.11
10	diameter power 0.67	34.81	POWER(B7,0.67)
11	slope power 0.5	0.06	POWER((1/B8),0.5)
12	Velocity, m/sec	0.67	(1/B9)*(3.968/1000)*B10*B11
13	diameter power 2.67	1392355	POWER(B7,2.67)
14	Flow rate l/s	21.1	(1/B9)*(3.118/1000000)*B13*B11
15	Flow in MLD	1.82	B14*3600*24/1000000

APPENDIX A 3.4 A
COMPUTATIONS OF DIAMETER, SLOPES, DISCHARGES IN GRAVITY SEWERS BY USING APPENDIX A 3.3

1 Given a sewer diameter, slope & depth of flow, find velocity and discharge

Diameter of sewer (D) = 200 mm
Depth of flow (d) = 160 mm
Sewer Material: Cement concrete pipe in “Good” condition (with collar joints)
Slope of the sewer line = 1 in 250
Manning’s co-efficient is uniform across sewer section
Use Manning value as 0.013 from Table 3.11
From Appendix A 3.2 A and A 3.2 B,
Velocity is 0.67 m/s and Discharge is 1260 lpm
Ratio of d/D is 160 / 200 = 0.8
Ratio of v/V from Table 3.12 is 1.14
Hence velocity is = 0.67 × 1.14 = 0.76 m/s
Ratio of q/Q from Table 3.12 is 0.968
Hence discharge is = 1260 × 0.968 = 1220 lpm

2 Given a sewer diameter, slope & discharge needed, find depth of flow

Diameter of sewer (D) = 200 mm
Discharge required = 1260 lpm
Sewer Material: Cement concrete pipe in “Good” condition (with collar joints)
Slope of the sewer line = 1 in 250
Manning’s co-efficient is uniform across sewer section
Use Manning value as 0.013 from Table 3.11
From Appendix A 3.2 A and A 3.2 B,
Velocity is 0.67 m/s and Discharge is 21.1 lps
Ratio of discharge is = 21.1 / 21.4 = 0.968
Corresponding ratio from Table 3.12 is d/D is 0.8
Hence, depth of flow is = 200 × 0.8 = 160 mm

3 Given discharge needed and available slope, find the diameter

Discharge needed = 612 lpm
Sewer Material: Cement concrete pipe in “Good” condition (with collar joints)
Available slope of the sewer line = 1 in 180
d/D ratio should not exceed 0.8
From Table 3.12, for d/D of 0.8, q/Q is 0.968
Discharge at full depth is = 612 / 0.968 = 632 lpm
From Appendix A 3.4 A, by entering the discharge and slope
Required diameter is 143 mm & velocity is 0.63 m/s as below
Use nearest higher diameter

APPENDIX A 3.4 B
MS EXCEL FOR MANNING FORMULA FOR CIRCULAR GRAVITY PIPES FLOWING FULL

No	A	B	C
1	Appendix 3.3		
2	MS Excel For Manning Formula for Circular Gravity Pipes Flowing Full		
3	Given discharge and slope of sewer, find diameter and velocity		
4	This is left blank for the designer to enter his design notes		
5	This is left blank for the designer to enter his design notes		
6	This is left blank for the designer to enter his design notes		
7	Discharge, l/s	10.20	Enter by Designer
8	slope 1 in	180	Enter by Designer
9	slope power 0.5	0.07	POWER((1/B8), 0.5)
10	value of Manning's n	0.013	Enter from Table 3.11
11	diameter power 2.67	570563	B7*B10/(3.118/1000000)/B9
12	diameter, mm	143	POWER(B11,(1/2.67)
13	diameter power 0.67	27.83	POWER(B12,0.67)
14	velocity m/s	0.63	POWER(B12,0.67)

APPENDIX A 3.5 A
NOMOGRAM FOR HAZEN WILLIAMS FORMULA FOR MAINS FLOWING FULL AND C VALUE OF 100
(For discharges from 100 to 100000 lpm)

For other values of C, the velocity and discharge will be directly proportional.

Example-Find the discharge and velocity of a sewer of diameter 300 mm flowing full slope of 1 in 100 and a Hazen Williams C value of 130

Answer-From the nomogram, V = 0.75 m/s and discharge = 5,700 lpm. For C value of 130,
V = 0.75 × 130 / 100 = 0.98 m/s & discharge = 5,700 × 130 / 100 = 7,400 lpm

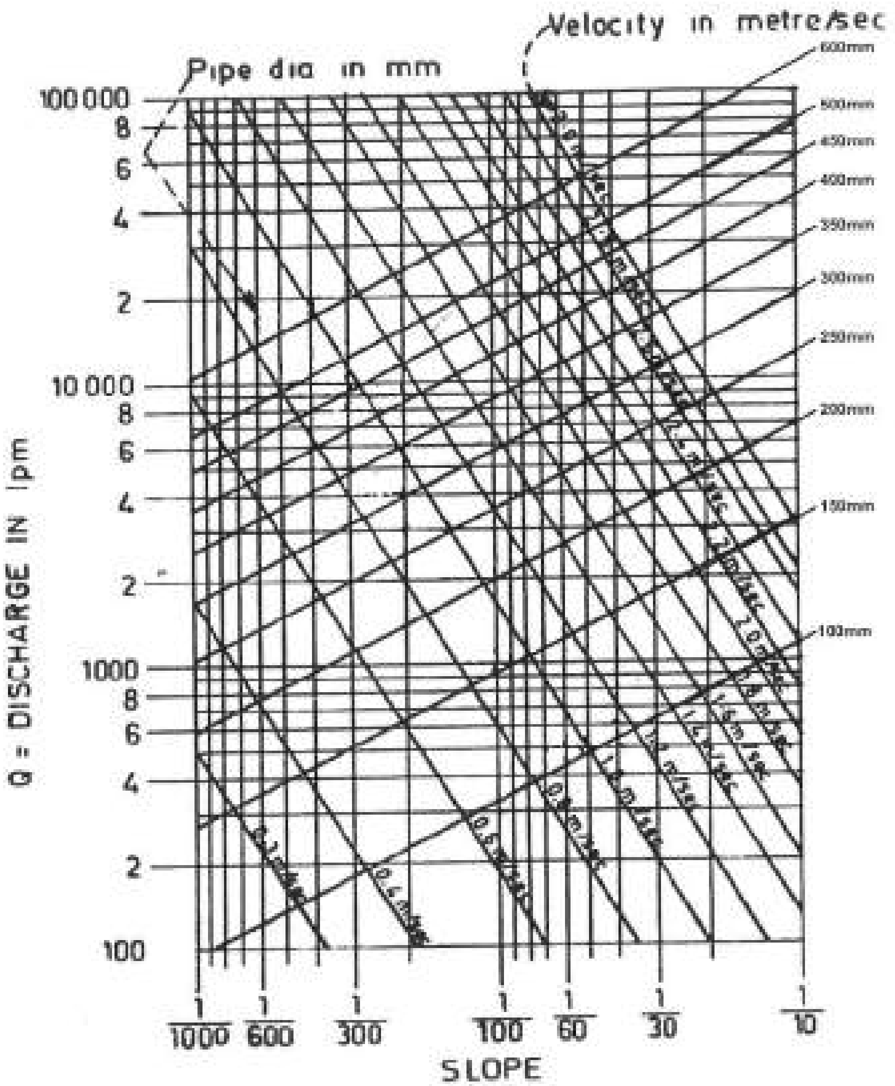


FIG 3 NOMOGRAM CHART FOR HAZEN-WILLIAMS FORMULA
(C = 100) FOR Q = 100lpm TO 100000lpm

Figure A3.5A-1

Source-“Data Matrix for Public Health Engineers”, TWAD Board, 1991

APPENDIX A 3.5 B
NOMOGRAM FOR HAZEN WILLIAMS FORMULA FOR MAINS FLOWING FULL AND C VALUE OF 100
(For discharges from 1000 to 1000000 lpm)

For other values of C the velocity and discharge will increase pro rata.
Example-Find the discharge and velocity of a sewer flowing full of diameter 1,200 mm, slope of 1 in 1,000 and a Hazen Williams C value of 130

Answer-From the nomogram, V = 0.95 m/s and discharge = 63,000 lpm. For C value of 130,
V = 0.95 × 130 / 100 = 1.24 m/s & discharge = 63,000 × 130 / 100 = 81,900 lpm

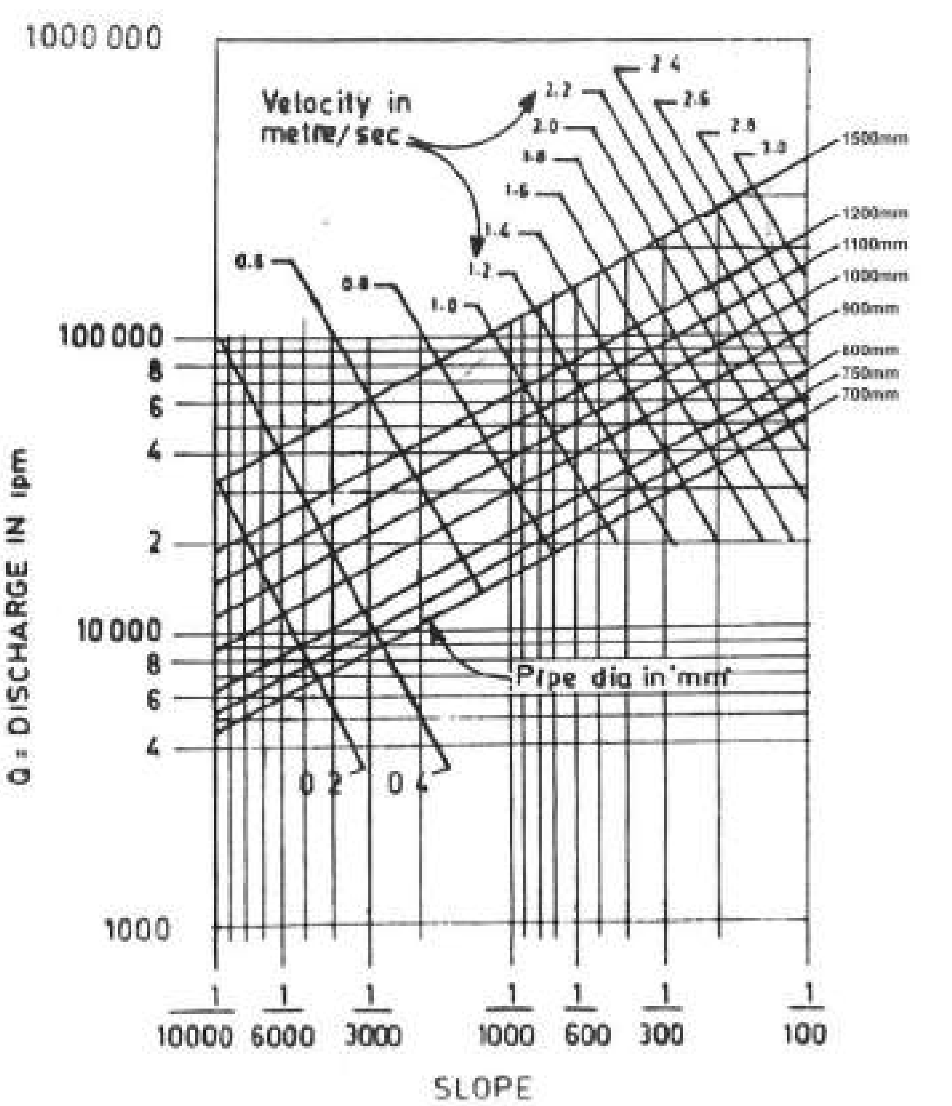


FIG 4 NOMOGRAM CHART FOR HAZEN-WILLIAMS FORMULA
(C : 100) FOR Q= 1000 lpm TO 1000000 lpm

Figure A3.5B-1

Source-“Data Matrix for Public Health Engineers”, TWAD Board, 1991

APPENDIX A 3.6
HAZEN WILLIAMS FORMULA FOR CIRCULAR SEWERS FLOWING UNDER PRESSURE

No	A	B	C
1	Appendix 3.6		
2	Hazen Williams Formula for Circular Sewers Flowing Under Pressure		
3	Given diameter and slope, find velocity and flow rate		
4	This is left blank for the designer to enter his design notes		
5	This is left blank for the designer to enter his design notes		
6	This is left blank for the designer to enter his design notes		
7	diameter, mm	300	Enter by Designer
8	slope 1 in	210	Enter by Designer
9	Hazen Williams C	140	Enter from Table 3.11
10	diameter power 0.63	36.36	POWER(B7,0.63)
11	diameter power 2.63	3272118	POWER(B7,2.63)
12	Slope power 0.54	0.056	POWER((1/B8),0.54)
13	Velocity m/s	1.295	(4.567/1000)*B9*B10*B12
14	Discharge cum / day	7913	(3.1/10000)*B11*B12*B9
15	Discharge in MLD	7.91	B14/1000
16			
17	Given flow rate and slope of sewer, find diameter and velocity		
18	Discharge cum / day	7913	Enter by Designer
19	Slope 1 in	210	Enter by Designer
20	Slope power 0.54	0.056	POWER(1/B19,0.54)
21	Hazen Williams C	140	Enter by Designer
22	diameter power 2.63	3272275	B18/(3.1/10000)/B20/B21
23	diameter, mm	300	POWER(B22,(1/2.63))
24	diameter power 0.63	36	Power(B23,0.63)
25	velocity, m/s	1.295	(4.567/1000)*B20*B21*B24

APPENDIX A 3.7
DESIGN OF SANITARY SEWER SYSTEM
(Retained as in 2nd edition)

1 PROBLEM

Design a system of sanitary sewers for the given area shown in the Figure A3.7-1 with the following details:

1. Population Density - 300 persons/hect.
2. Water Supply - 250 lpd/head (ultimate).
3. Maximum rate of infiltration - 20,000 lpd/hect.
4. Minimum depth of cover to be provided over the crown of the sewer - 1 m
5. Minimum velocity in sewer at peak flow - 0.6mps
6. Maximum velocity in sewer - 2.0 mps
7. Minimum size of the sewer - 150 mm
8. Waste water reaching sewers - 90% of W/S
9. Peak flow - 3.5 × Ave. flow

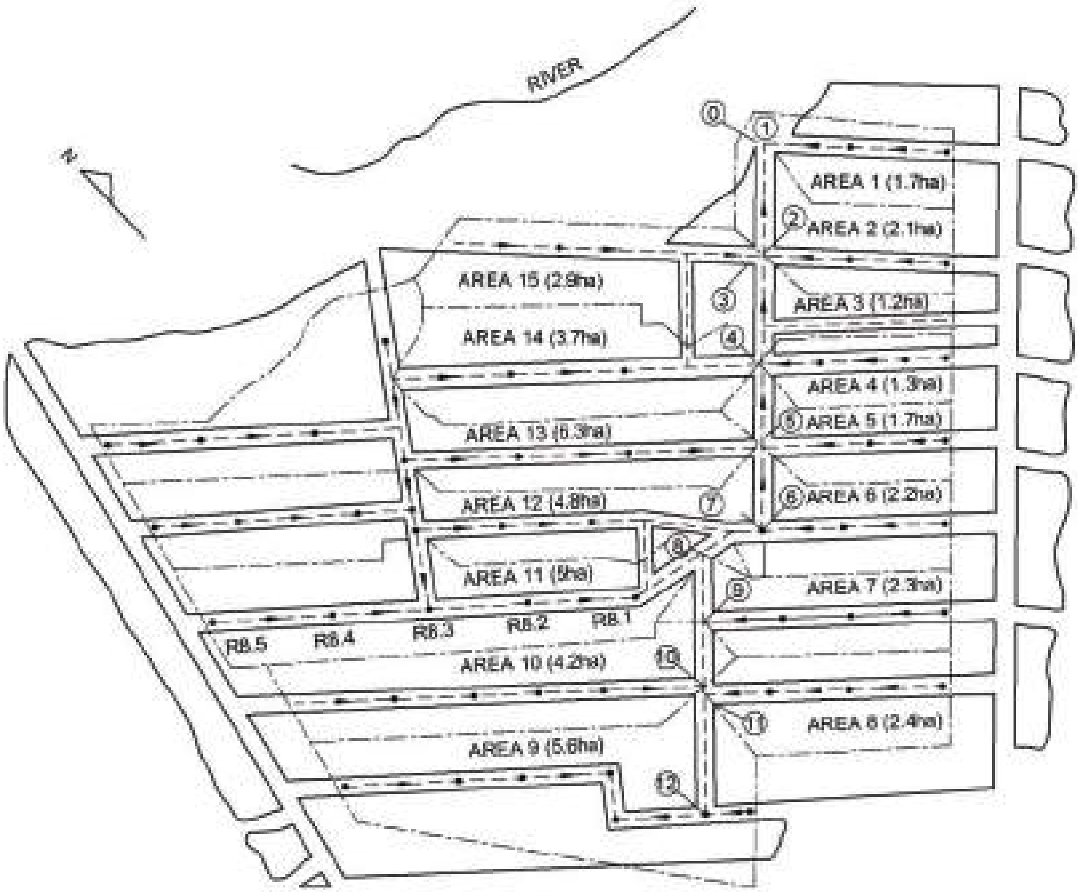


Figure A3.7-1

2 SOLUTION

1. Draw a line to represent the proposed sewer in each street or valley to be served. Near the line indicate by an arrow the direction in which sewage is to flow.

2. Locate the manhole, giving each an identification number.

3. Sketch the limits of the service areas for each lateral.

4. Measure the areas (ha) of the several service areas.

5. Prepare a table as shown in Table A3.7-1 with the columns for the different steps in computation and a line for each section of sewer between manholes.

Columns 1-6 for the line manhole, location of the manhole, manhole numbers, ground level at starting manhole and length of line between the manholes.

Columns 7-8 the corresponding area for the next street of sewer and in Col.8 the sum of the areas are entered.

Column 9 the population served by each corresponding line is entered.

Column 10 shows the sewage flow (mld) through each line. The sewage flow is assumed as 90% of the per capita water supply.

Column 11 shows the ground water infiltration for each area = $20{,}000 \times 10^{-6} \times$ Col.8.

Column 12 gives the peak flow i.e. Col.10 × 3 + Col.11.

Column 13 gives the peak flow in lps.

Column 14-15 indicate the diameter and slope of the pipes determined from the Manning's chart.

Columns 16-17 indicate the discharge through pipe flowing full and the actual discharge through the pipes i.e. as Col. 13.

Column 18 also determined from the Manning's chart when pipe following full.

Column 19 calculated from the hydraulic elements curve for the circular pipes.

Column 20 gives Col.6 × Col.15.

Columns 21-22 invert levels of the lines are calculated.

Table A3.7-1 Design of a sewer system

Line	Location	Manhole		Ground level at start manhole	Length	Area Served (ha)		Population	Sewage flow	Ground water infiltration	Peak flow		Diameter	Slope	Discharge lps		Velocity mps		Total fall	Invert Elevation m	
		From	To		m	Increment	Total		mld	mld	mld	lps	mm		Q Full	Q Actual	V Full	V Actual	m	Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
1	Street	R.8.5	R.8.4	38.275	120	0.80	0.80	240	0.054	0.016	0.205	2.37	150	0.008	14	2.37	0.75	0.57*	0.96	37.125	36.165
2	Street	R.8.4	R.8.3	37.960	116	1.20	2.00	600	0.135	0.040	0.512	5.92	150	0.008	14	5.92	0.75	0.72	0.93	36.135**	35.205
3	Street	R.8.3	R.8.2	36.873	114	1.40	3.40	1020	0.230	0.068	0.873	10.10	150	0.008	14	10.10	0.75	0.82	0.91	35.175	34.265
4	Street	R.8.2	R.8.1	36.895	116	0.90	4.30	1290	0.290	0.066	1.10	12.73	150	0.008	14	12.73	0.75	0.86	0.93	34.235	33.305
5	Street	R.8.1	8	36.420	75	0.70	5.0	1500	0.34	0.10	1.29	14.92	200	0.005	24	14.92	0.70	0.74	0.38	34.275	33.895
6	Street	8	7	36.117	41	14.5	19.5	5850	1.32	0.39	5.01	57.96	300	0.005	70	57.96	1.0	1.13	0.21	33.845	33.635
7	Street	7	6	35.830	26	4.8	24.3	7300	1.64	0.48	6.22	71.96	350	0.005	100	71.96	1.2	1.32	0.13	33.605	33.475
8	Main St.	6	5	35.105	88	2.2	26.5	7950	1.80	0.53	6.83	79.02	350	0.005	100	79.02	1.2	1.32	0.44	33.445	33.005
9	-do-	5	4	34.412	86	7.8	34.3	10300	2.31	0.68	8.76	101.35	400	0.0033	125	101.35	1.0	1.12	0.29	32.975	32.685
10	-do-	4	3	34.181	36	5.0	39.3	11800	2.65	0.70	10.05	116.28	400	0.0033	125	116.28	1.0	1.14	0.12	32.655	32.535
11	-do-	3	2	34.105	77	1.2	40.5	12150	2.73	0.80	10.35	119.75	400	0.0033	125	119.75	1.0	1.14	0.26	32.505	32.245
12	-do-	2	1	34.905	117	5.0	45.5	13650	3.07	3.91	11.65	134.79	450	0.0033	160	134.79	1.0	1.12	0.39	32.208	31.811
13	-do-	1	0	33.250	41	1.7	47.2	14200	3.2	0.94	12.14	140.46	450	0.0033	160	140.46	1.0	1.12	0.14	31.788	31.641

* Since velocity is less than 0.6mps, flushing once a day is necessary; ** A minimum level difference of 30 mm is provided between incoming and outgoing sewers to provide necessary slope in the manhole.

APPENDIX A 3.8
ILLUSTRATIVE EXAMPLES FOR STRUCTURAL DESIGN OF BURIED CONDUITS
(Retained as in 2nd edition)

General assumptions

The general assumptions relating to the characteristics of soil and other factors for the examples are given below:

i) Saturated density of fill(w)= 2,000 kg/m^3

ii) Kμ = Kμ' = 0.130, ordinary maximum tor clay (thoroughly wet)

iii) r_{sd} for rigid conduit on ordinary bedding= 0.7 for positive projection and -0.3 for negative projection

iv) Projection ratio = 1

v) Concentrated surcharge corresponding to wheel load for Class AA wheel loading=6.25T

vi) Impact factor = 1.5

vii) Factor of Safety for safe supporting strength = 1.1

viii) The design also provides for accidental surcharge of drains and accounts for a water load of 75% as per standard practice, based on the assumption that the sewage flow is 3/4 full.

Determination of fill loads over pipes

EXAMPLE I

1 PROBLEM

Determine the fill load on a 1,200 mm dia. NP2 Class concrete pipe installed in a trench of width of 2.3 m and depth of 4.00 m.

2 SOLUTION

Pipe thickness 't' = 65mm for D of 1,200mm

B_c = D + 2t = 12,00 + 130 = 1,330mm = 1.33m

B_d = 2.3m

H = 4.00 – 1.33 = 2.67 m

H/ B_d = (2.67 / 2.3) = 1.16

B_d is < 2B_c Hence trench formula is applicable.

C_d = 0.9965 or 1.00 (from Table3.19) for ordinary maximum for clay.

From Equation (3.29)

$W_c = C_d\, w\, B_d^{\,2}$ = 1.00 × 2,000 × 2.32 = 10,580 kg/m

EXAMPLE II

1 PROBLEM

Determine the fill load on 900mm dia NP2 Class concrete pipe installed in a trench of width 2.1 m and depth 6.0 m.

2 SOLUTION

Pipe thickness 't' 50mm for D of 900mrn

Bc = D + 2t = 900 + 100 = 1,000mm =1m

w = 2,000kg/m^2

H = 6.0 – 1.0 = 5.0m

B_d = 2.1m

(H / B_d) = (5.0 / 2.1) = 2.38

$2B_c < B_d < 3B_c$ Hence either the trench or embankment formula can be used,

From Table 3.19

Cd = 1.77188 or say 1.8

From Equation (3.29)

$W_c = C_d\, w\, B_d^2$ = 1.8 × 2,000 × 2.1^2 = 15,876 kg/m or say 16,000 kg/m.

EXAMPLE III

1 PROBLEM

Determine the fill load on a 1,200mm dia NP2 Class concrete pipe installed as a positive projecting conduit under a fill of 7 m height above the top of pipe. The pipe wall thickness is 65mm and the fill weight 2,000 kg/m^3.

2 SOLUTION

Assume r_{sd} = 0.7 and p = 1.0

H = 7 m

B_c = 1,200 + 130 = 1,330mm = 1.33m

H/B_c = 7/1.33 = 5.26

rsd × p = 0.7 × 1 = 0.7

C_c = 9 (from Figure 3.34)

Using Equation (3.25)

$W_c = C_c\, w\, B_c^2$ = 9 × 2,000 × 1.332 = 31,850 kg/m

EXAMPLE IV

1 PROBLEM

Determine the fill load on a 1,200mm dia NP2 Class pipe installed as a negative projection conduit in a trench the depth of which is such that the top of the pipe is 2 m below the surface of natural ground in which the trench is dug. The height of the fill over the top of the pipe is 10m.

2 SOLUTION

Assume the width of the trench as 2 m and fill weight, w = 2,000 kg/m^3
Assume r_{sd} = – 0.3 and p' = 1.0
H = 10m, B_d = 2.00m H/B_d = 10/2 = 5.00
For values of p' = 1.0 r_{sd} = – 0.3 and H/B_d = 5.00
C_n = 3.2 (from Figure 3.36)
Using Equation (3.26)
$W_c = C_n\, w\, B_d^2 = 3.2 \times 2{,}000 \times 2.02 = 25{,}600$ kg/m

EXAMPLE V

1 PROBLEM

Determine the load on 1,500mm dia conduit in tunnel condition 15 m deep in a soil of silty sand.

2 SOLUTION

The maximum width of excavation (Bt) may be assumed as 1,950mm; and the cohesion coefficient (C) of the soil as 500 kg/m^2

Kμ = 0.15 and w = 1,800 kg/m^3
H =15 m; Bt = 1.95 m
H/Bt = 15/1.95 = 7.7
Ct = 3.00 (from Figure 3.42)
Using Equation (3.31)

$$W_t = C_t.\ B_t.(w\ B_t - 2C) = 3.00 \times 1.95\ (1{,}800 \times 1.95 - 2 \times 500)$$
$$= 3.00 \times 1.95 \times 2{,}510 = 14{,}680 \text{ kg/m}$$

EXAMPLE VI

1 PROBLEM

Determine the load on a 600 mm dia NP2 Class pipe (t = 40 mm) under 1 m cover caused by 6.25 Tonnes Wheel load applied directly above the centre of pipe.

2 SOLUTION

L = 1 m (since standard length of conduit 1 m)
H = 1 m
B_c = 600+80 = 680 mm = 0.68 m
(L/2H) = (1.0/2× 1) = 0.50
(B_c /2H) = (0.68/2 × 1) = 0.34
From Table 3.21 for values of (L/2H) = 0.50
And (B_c /2H) = 0.34
C_s = 0.248
Using Equation (3.33)
$W_{sc} = C_s\ (PF / L) = (0.248 \times 6250 \times 1.5/1.0) = 2{,}325$ kg/m

EXAMPLE VII

1 PROBLEM

Determine the load on a 1,200 mm dia concrete pipe under 2 m of cover resulting from a broad gauge railway track loading;

2 SOLUTION

Assumed thickness of pipe = 100 mm
Axle load P = 22.5 tonnes
Impact factor F = 1.75
Length of sleeper 2A = D = 2.7 m
Assume 4 axles spaced 1.84 m on the locomotive (2B)
M = 4 × 2B = 4 × 1.84 = 7.36 m; H = 2 m
Weight of track structure =wt = 0.3 T/m
Using equation (3.37)

$$U = \frac{PF + 2W_t B}{4AB} = \frac{PF}{4AB} + \frac{W_t}{2A}$$
$$= \frac{22.5 \times 1.75}{2.7 \times 1.84} + \frac{0.3}{2.7} \quad T/m^2$$
$$= 7.925 + 0.111 = 8.036 \text{ tonnes/m}^2$$

Bc = 1,200 + 200 = 1,400mm = 1.4 m

$$\frac{D}{2H} = \frac{2.7}{2 \times 2} = 0.675$$
$$\frac{M}{2H} = \frac{4 \times 1.84}{2 \times 2} = 1.84$$

From Table 3.21
Influence Coefficient C_s = 0.652
Using Equation (3.36)
$W = 4\ C_s\ U\ B_c = 4 \times 0.652 \times 8.036 \times 1.4 = 29.34$ tonnes/m = 29,340 kg/m
(Since it has been given that it is a broad gauge track, the formula $W = 32.14\ C_s\ B_c$, could be used directly without calculating the value of U).
Using the formula $W = 32.14\ C_s\ B_c$
$W = 32.14 \times 0.652 \times 1.4 = 29.337$ t/m. or 29,337 kg/m

EXAMPLE VIII

1 PROBLEM

Design the structural requirement tor a 900mm dia. NP3 class sewer pipe which is to be laid in 6m deep trench of 2.0 m width assuming that the total vertical load will account tor concentrated surcharge of 6.25 T applied at the centre of the pipe. The water load should also be considered.

2 SOLUTION

The type of bedding for the purpose of this example may be assumed as Ab class with load factor of 2.8.

B_c = 900 + 2 × 50 = 1,000 mm = 1.0 m
H = 6 – 1 = 5 m
B_d = 2.0
H/B_d = 5/2.0 = 2.50
Cd = 1.764 (from table 3.13 for saturated top soil)
Using equation (3.29) ……. $W_c = C_d\, w\, B_d^2$
Wc = 1.764 × 2,000 × 22 = 14,110 kg/m.
L = 1m, H = 5m

$$\frac{L}{2H} = \frac{1}{10} = 0.1$$

and

$$\frac{B_c}{2H} = \frac{1}{10} = 0.1$$

From Table 3.21 $C_s = 0.019$
Using Equation (3.33)
$W_{sc} = C_s$ (PF/L) = (0.019 × 6,250 × 1.5) / 1 = 178 kg/m

$$\text{Water Load } W_w = \frac{22}{7} \times \frac{9}{10} \times \frac{9}{10} \times \frac{1}{4} \times 1{,}000 \times \frac{75}{100} = 471\ kg/m$$

$W_t = W_c + W_{sc} + W_w$ = 14,110 + 178 + 471 = 14,759 or say 14,800 kg/m

Safe supporting strength of 900 mm NP_2 pipe with class A_b bedding = [(3,750 × 2.8) / 1.5] = 7,000 kgs/m which is less than the total load on the pipe i.e. 14,800 kgs/m.

Safe supporting strength of 900 mm NP_3 pipe with class A_b bedding = [(10,140 × 2.8) / 1.5] = 18,928 kgs/m. which is more than the total load on the pipe i.e. 14,800 kgs/m.

Design of anti flotation blocks

EXAMPLE IX

1 PROBLEM

A RCC pipeline of internal dia 2,000mm and barrel thickness of 115mm is to be laid below Ground level. Each pipe is 2.5 metre long and weighs 2 Tonnes. The minimum overburden required to prevent the pipe from upliftment is to be determined. Where there is no over-burden the size of RCC of antiflotation block required to prevent it from flotation is to be determined.

2 SOLUTION

Depth of cover to prevent flotation of an empty pipeline.

$$H_{min}\ B_c\ (w_s - w_o) + W_c = \left(\frac{\pi}{4}\right) B_o^{\,2}\ w_o$$

Where,

H_{min} : Minimum depth of fill required to prevent flotation of empty pipe
B_c : O.D. of pipe, meters
W_s : Density of (soil) fill material = 1,800 kg/m^3
W_o : Density of water = 1,000 kg/m^3

To show that the pipe gets lifted up if there is no over burden
Weight of empty pipe W_c = 2,000 kg/metre
B_c = 2.00 + 0.23 = 2.23 metre (O.D. of pipe)
When there is no over burden weight of water
displaced = (π / 4) $B_c^2\ w_o$
(π / 4) (2.23)2 1,000 = 3,910 kg or 3.91 tonnes

Since the weight of Empty pipe (2 tonnes) is less than the upward weight of water (3.91 tonnes) the pipe will float.

Depth of minimum overburden required to prevent flotation with a factor of safety 1.2

$H_{min}\ B_c$ (ws – wo) + W_c = (π / 4) × $B_c^2\ w_o$
H_{min} × 2.23 (1.8 – 1.00) + 2 = [(π/ 4)] × 2.232 × 1 × (Factor of safety of 1.2)
H_{min} = 1.5 metres

Hence it is desirable to provide a cover of 1.5 metres to prevent flotation of pipeline.

Where it is not possible to provide the above minimum over burden anti flotation blocks can be provided for each pipe to prevent flotation of pipeline.

The Anchoring force required to be created is equal to the 1st term of the Equation (3.32).

$H_{min}\ B_c\ (w_s - w_o)$
H_{min} = 1.5m with a factor of safety of 1.2
B_c = 2.23m
w_s = 1,800 kg/m^3
w_o = 1,000 kg/m^3
1.5 × 2.23 × (1.8 – 1.00) = 2,680 kg/metre length of pipe
Anchoring force required for each pipe of 2.5 metre long
= 2.68 × 2.5 = 6.7 tonnes per pipe

Volume of concrete to be provided;

Submerged weight of concrete: (2,400 – 1,000) = 1,400 kg/m3 or 1.4 tonnes/m^3

Volume = (6.7 / 1.4) = 4.78 m^3

Provide anti flotation block of size 2.85 × 1.5 × 1.20 m for each pipe of 2.5 m long (Figure A3.8-1).

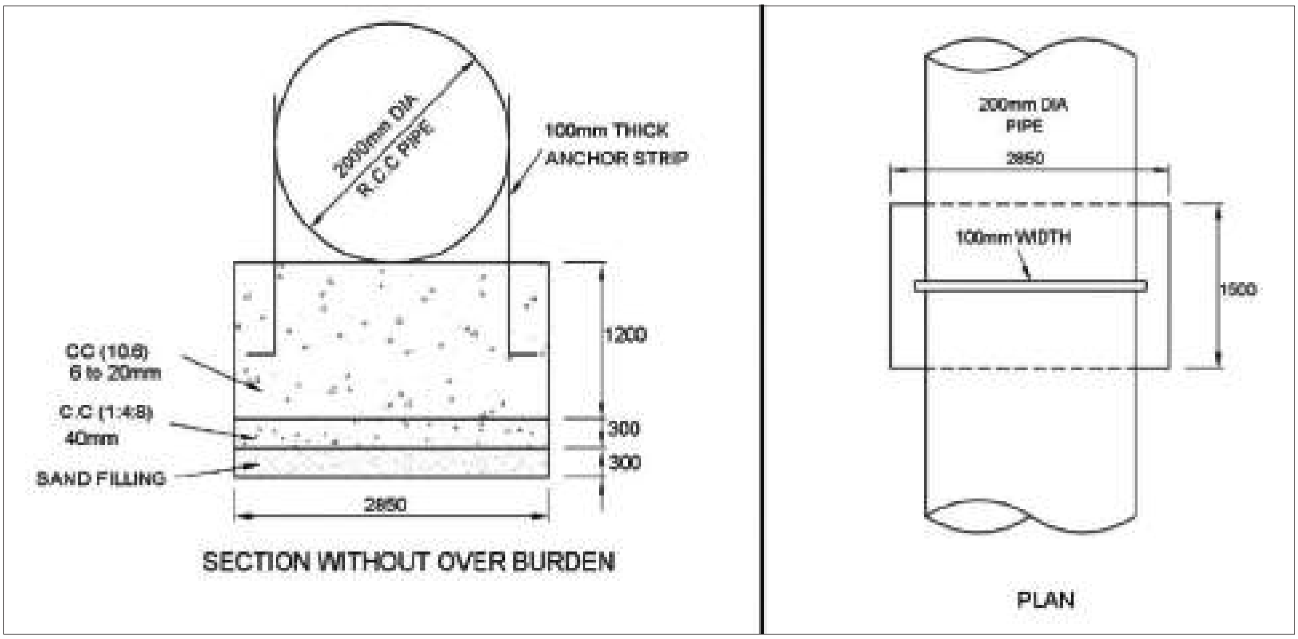


Figure A3.8.1 Anti Floatation Blocks

APPENDIX A 3.9
THREE EDGE BEARING TESTS FOR PIPE STRENGTH
(Retained as in 2nd edition)

The load which the pipe must withstand without failure is termed three-edge bearing strength. For non reinforced concrete pipes, the point of load at which the pipe cracks and fails is the termination of a three-edge bearing test.

For reinforced concrete pipes, these specifications provide two criteria for passing the three-edge bearing test; first, there is an intermediate load based on the appearance of a crack 0.25 mm wide and 0.3 m long. The final requirement for reinforced pipe is the ultimate three-edge bearing strength at the final failure of the pipe where no further load increase can be supported.

In conducting this test, the pipe is placed horizontally on two parallel wooden rails resting on 15cm × 15cm bearing block or other solid support that extends the length of the pipe. An upper bearing block is placed on the top of the pipe. Next, a rigid I-beam or other structural member is placed on the upper bearing block to apply the load to the block.

Table A3.9-1 Three edge bearing strengths of concrete pipes are given below

	Load to produce 0.25 crack (kg/liner meter)				Ultimate load (kg/liner meter)		
Dia of pipe mm	Concrete				Concrete		
	NP_2 P_1 P_2 & P_3	NP_3	NP_4	NP_1	NP_2 P_1 P_2 & P_3	NP_3	NP_4
1	2	3	4	5	6	7	8
80	1,040	-	-	-	1,560	-	-
100	1,040	-	-	1,560	1,560	-	-
125	-	-	-	-	-	-	-
150	1,040	-	-	1,560	1,560	-	-
200	-	-	-	-	-	-	-
250	1,140	-	-	1,670	1,710	-	-
300	1,200	-	-	1,790	1,800	-	-
350	1,260	3,040	-	1,880	1,890	4,363	-
400	1,360	3,460	3,460	2,020	2,040	5,190	5,190
450	1,480	3,730	-	2,220	2,220	5,640	-
500	1,660	4,160	4,160	-	2,490	6,240	6,240
600	1,900	4,720	4,720	-	2,850	7,980	7,980
700	2,100	5,320	5,120	-	3,150	7,980	7,980
800	2,300	6,060	6,060	-	3,430	9,090	9,090
900	2,500	6,760	6,760	-	3,750	10,140	10,140
1,000	2,680	7,400	7,400	-	4,020	11,100	11,100
1,100	2,780	8,200	8,200	-	4,170	12,300	12,300
1,200	2,880	9,000	9,000	-	4,320	13,500	13,500
1,400	2,900	-	10,610	-	4,470	-	17,950
1,600	2,980	-	12,800	-	4,470	-	18,300
1,800	2,980	-	13,800	-	4,470	-	20,700

APPENDIX A 3-10
RELATIVE LIMITATIONS ON USE OF PIPE MATERIALS (IN ALPHABETICAL ORDER) IN SPECIFIC LOCATIONS
(Pipe materials listed in alphabetical order)

Asbestos Cement (AC) Pipes	
AC-1	Not suitable for high sulphate levels in sewage / soil water, unless good lining such as HDPE/GRP/PVC/PE/CM with sulphate resistant cement lining is provided or pipe manufactured with sulphate resistant cement or Alumina cement. The quality of the lining shall be ensured by the competent authority.
AC-2	Not in aggressive soils/groundwater or tidal zone unless sulphate resistant cement or high alumina cement is used in manufacture.
AC-3	Not in locations where live load from vehicular traffic will occur on the laid pipeline
Cast Iron or Ductile Iron (CIDI) Pipes	
CIDI-1	Never to be used near buried electricity transmission high tension cables
CIDI-2	Wherever used above ground supports at each pipe length shall be ensured without any subsidence.
CIDI-3	Pipes with external synthetic coatings not to be used in marine coastal environments to prevent leaching of constituent chemicals into the environment
Glass Fiber Reinforced (GRP) Pipes	
GRP - 1	Not in area where future works may affect the pipe side support.
GRP - 2	Not in ground contaminated or possibly contaminated by certain chemicals in concentrations deleterious to the resin of the pipe.
GRP - 3	Do not use pipes/couplings with chips, cracks, crazing, layer delamination or exposed fibres or ends of pipes not sealed with resin
GRP - 4	Do not use pipe and couplings, stored unprotected from sunlight for more than 9 months.
GRP - 5	Do not use in ground conditions having low stiffness, e.g. tidal zone.
GRP - 6	Not in location subjected to vehicular load and has insufficient cover.
GRP - 7	Not in areas subject to excavations by other service providers within 2m radial distance of pipeline.
GRP - 8	Not in ground subject to differential settlement or extreme movement
GRP - 9	Not in ground offering low side support strength to the pipe.
GRP - 10	Do not use when control of construction practices is not adequate to ensure quality of embedment for flexible pipes.

GRP - 11	Not suitable for uncertainties in geotechnical analysis to determine if flexible pipe structurally suitable
GRP - 12	Uplift precaution in locations where high groundwater table and empty pipe may be encountered
Mild Steel (MS) Pipes	
MS - 1	Never to be used near electricity transmission cables
MS - 2	Never to be used below ground unless proper protection against corrosion from soil and soil water is ensured
MS - 3	Can be considered with appropriate lining and protection for gravity flow under exceptional circumstances. The quality of the lining shall be ensured by the competent authority.
MS - 4	Choice of spiral welded Vs or horizontal welded pipes shall be evaluated with respect to overburden
Reinforced Cement Concrete (RCC) Pipes	
RCC - 1	Not suitable for high sulphate levels in sewage / soil water, unless good lining such as HDPE/GRP/PVC/PE/CM with sulphate resistant cement lining is provided or pipe manufactured with sulphate resistant cement or Alumina cement. The quality of the lining shall be ensured by the competent authority.
RCC - 2	Not in aggressive soils/groundwater or tidal zone unless sulphate resistant cement or high alumina cement is used in manufacture.
Stoneware Pipes with hemp yarn and cement mortar (SWCM) packing joints	
SWCM - 1	Not in unstable ground, i.e. refilled ground, tidal zone.
SWCM - 2	Not suitable for above ground installation.
SWCM - 3	Not in the vicinity of trees with aggressive root systems.
SWCM - 4	Not to be used for crossing beneath water courses.
Stoneware Pipes with O ring (SQOR) joints	
SWOR - 1	Not in unstable ground, i.e. refilled ground, tidal zone.
SWOR - 2	Not suitable for above ground installation.
SWOR - 3	Not to be used for crossing beneath water courses.
Synthetic Pipes like Profile Wall, Double walled corrugated, PE, Solid Wall, HDPE, UPVC (SP) Pipes	
SP - 1	Not in location subjected to vehicular load and has insufficient cover.
SP - 2	Not in areas subjected to third party interference, e.g. excavations within 2m of pipeline by other parties.
SP - 3	Not in ground offering low side support strength to the pipe

SP - 4	Not in ground which allows migration of pipe embedment material into it.
SP - 5	Not in ground contaminated with deleterious chemicals
SP - 6	Not suitable for above ground installation
SP - 7	Not suitable as reticulations systems except for special applications

Note: Wherever special circumstances are encountered the above limitations can be overcome by appropriate precautions with the documented approval of the competent authority

APPENDIX A 4.1
COMPUTATION OF FRICTION FACTOR IN PUMPING MAINS

1	B	C	D
2	Appendix A 4-1		
3	Friction factor for Fittings in Pressure Mains		
4	Given the type and numbers of fittings, find the total friction factor		
5	This is left blank for the designer to enter his design notes		
6	Number of Sudden contractions	Enter by Designer	2
7	Friction Factor for each	From Table 4.2	0.5
8	Friction factor for all	D6*D7	1
9	Entrance shape well rounded	Enter by Designer	1
10	Friction Factor for each	From Table 4.2	0.5
11	Friction factor for all	D9*D10	0.5
12	Elbow 90 degrees	Enter by Designer	4
13	Friction Factor for each	From Table 4.2	1
14	Friction factor for all	D12*D13	4
15	Elbow 45 degrees	Enter by Designer	4
16	Friction Factor for each	From Table 4.2	0.75
17	Friction factor for all	D15*D16	3
18	Elbow 22 degrees	Enter by Designer	2
19	Friction Factor for each	From Table 4.2	0.5
20	Friction factor for all	D18*D19	1
21	Tee 90 degrees	Enter by Designer	4
22	Friction Factor for each	From Table 4.2	1.5
23	Friction factor for all	D21*D22	6
24	Tee in straight pipe	Enter by Designer	1
25	Friction Factor for each	From Table 4.2	0.3
26	Friction factor for all	D24*D25	0.3
27	Gate valve open	Enter by Designer	4

28	Friction Factor for each	From Table 4.2	0.4
29	Friction factor for all	D27*D28	1.6
30	Valve with reducer and increaser	Enter by Designer	3
31	Friction Factor for each	From Table 4.2	0.5
32	Friction factor for all	D30*D31	1.5
33	Globe valve	Enter by Designer	4
34	Friction Factor for each	From Table 4.2	10
35	Friction factor for all	D33*D34	40
36	Angle	Enter by Designer	2
37	Friction Factor for each	From Table 4.2	5
38	Friction factor for all	D36*D37	10
39	Swing check	Enter by Designer	1
40	Friction Factor for each	From Table 4.2	2.5
41	Friction factor for all	D39*D40	2.5
42	Venturi meter	Enter by Designer	1
43	Friction Factor for each	From Table 4.2	0.3
44	Friction factor for all	D42*D43	0.3
45	Orifice	Enter by Designer	1
46	Friction Factor for each	From Table 4.2	1
47	Friction factor for all	D45*D46	1
48	Total friction factor	D8+D11+D14+D17+D20+D23+D26+D29+D32+ D35+D38+D41+D44+D47	72.7

APPENDIX A 4.2
CALCULATION OF KW NEEDED FOR PUMPING

Given

Low sewage level	= 3.5 m
Delivery level	= 32.5 m
Velocity	= 0.883 m/s
Discharge	= 7,913 cum / day
Slope	= 210
Length of pumping main	= 4,200 m
Top of goose neck before delivery	= 34.0 m

Friction factor due to fittings as in Appendix 4-1

Type of pump is submersible

Efficiency assumed	= 0.65
Safety factor for estimation	=1 / 0.9

Answer

Goose neck is treated as well rounded entrance	= factor is 1
Hence friction loss due to fittings is	= 70 + 2 = 72
Discharge	= 7,913 × 1,000 / 24 / 3,600 = 91.6 lps
Velocity head	= 0.883 × 0.883 / 2 / 9.81 = 0.04
Loss of head in fittings	= 0.04 × 72 = 2.87 m
Top elevation of goose neck at delivery	= 34.0 m
Add factor of safety against cavitation	= 1 m
Static lift	= (34+1) – 3.5 = 31.5 m
Friction loss	= 4,200 / 210 = 20 m
Total system head	= 2.87 + 31.5 + 20 = 54.37 m
Pump efficiency	= 0.65
Actual kW needed	= 91.6 × 54.37 / 100.5/ 0.65 / 0.9 = 85 kW

APPENDIX A 4.3

EFFECTS OF SILTING AND EROSION SEWAGE PUMPING MAIN PERFORMANCE

The following is an extract from the "Master Plan for Water and Sewerage" by the WHO and UNDP for the Chennai Metropolitan Area prepared dated 1977 and which was the basis for the country's first birth of an exclusive Water Supply and Sewerage Board at Chennai.

Five numbers of differently aged sewage pumping mains of Chennai sewerage system were selected and the hydraulics were evaluated using Rhodamine B Fluorescent tracer dyes injected into the pump deliveries and the concentrations were measured with a fluorometer at the start and discharge of the pumping mains and related to the measured flows, diameter and length of the pipe lines which were all Cast Iron mains with spigot socket lead joints. The study was aimed to establish the velocity profiles, siltation in the pumping mains and the resulting friction co-efficient. All the pumping mains were low lift pumping stations and hence the Manning's n was selected as the criteria of friction factor evaluation.

1 The Physical Setting and Methodology

The sandy soils in the area coupled with the local practice of scouring cooking pots with sand contribute to the inordinate quantities of grit and silt found in Madras sewage. This phenomenon was recognized by Mr. J. W. Madeley, and his investigations conducted in the early 1900's led to the construction of degritting wells ahead of the major pump stations which however were mostly non-operational. As part of the present investigations, tests were conducted at five of the pump stations. Static head and friction losses were measured at several flows with dual pressure gauges immediately downstream from the pump station manifold. Flows were measured by using a Turner fluorometer and standardized solutions of Rhodamine "B" dye. The dye was fed at a constant rate into the pump bell mouth by a calibrated peristaltic pump and samples were collected at a point far enough downstream from the pump to ensure complete mixing. Flow rates were computed from the observed dilution of the standard fluorescent dye solution.

The results of these tests are presented in Table A4.3-1. The measurements of flow, total head and static head are given in columns (1) to (3). The dynamic head column (4) is the difference between the total and static heads. Column (5) shows the calculated flow velocities. The "K" value in column (6) is equal to Δhd/Q2. The Manning's roughness coefficient, n, was calculated from the Manning equation:

$$Q = (A/n)R^{2/3}(\Delta h_d / L)^{1/2}$$

Examination of Table A4.3-1 shows that the calculated K and n values change with flow and velocity. Higher apparent n values are associated with lower velocities. Hydraulic theory dictates that there are only two possible causes for this phenomenon, either the basic relation changes or the cross sectional area of the pipe and / or the roughness of its interior change. Dr. Walter L Moore discussed the first of these possibilities in his 1959 ASCE paper entitled "Relationships between pipe resistance formulas", where it was shown that the friction loss constant K, does increase slightly as the flow is reduced towards the point where the laminar flow regime begins. To account for this phenomenon, Dr Moore developed a procedure for varying the exponent "m" as a function of the Reynolds Number and, thereby, allowing K to remain constant.

Table A4.3-1 Force Main Friction Loss Analysis

Force main	Q flow	Δh_T	Δh_S	Δh_d	Velocity	Measured K $\Delta h_d/Q^2$	Computed Roughness Coefficient
	m^3/s	Total Head meters of water	Static Head metres of water	Dynamic Head metres of water	m/s	S^2/m^5	"n"
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Greams road	0.095	4.92	2.81	2.11	0.83 / 0.57	234	0.0104
625 m of 380 mm	0.136	7.03	2.81	4.22	1.19 / 0.82	228	0.0102
1,595 m of 460 mm	0.195	11.96	2.81	9.15	1.71 / 1.19	241	0.0106
New Purasawalkam	0.387	2.39	1.4	1.0	0.43 / 0.32	6.68	0.0154
3,017 m of 1,065 mm	0.449	2.67	1.4	1.27	0.50 / 0.38	6.30	0.0150
1,723 m of 1,200 mm	0.544	3.16	1.4	1.76	0.61 / 0.46	5.95	0.0145
	0.66	7.38	1.4	5.98	0.74 / 0.57	13.7	0.0221
Langs Garden	0.0665	2.8	nil	2.8	0.91	633	0.0134
603 m of 305 mm	0.074	3.5	nil	3.5	1.01	639	0.0135
Chetput	0.0296	5.62	nil	5.62	0.58	6414	0.0205
990 m of 225 mm	0.043	5.975	nil	5.97	0.85	3229	0.0146
	0.0296	5.45	nil	5.45	0.58	6220	0.0202
Law College	0.071			1.8	0.32	357	0.0240
2,107 m of 535 mm	0.085			3.2	0.38	443	0.0267
	0.111			5.49	0.50	446	0.0268
	0.15			4.45	0.67	198	0.0179
	0.17			3.4	0.76	118	0.0138
	0.202			4.45	0.90	109	0.0133

Modified K values were computed using Dr Moore's method in an attempt to explain the severe variations noted in column (6), but this refinement did little to stabilise the K values, and therefore, the variations must be attributed to the silt buildup changing the cross sectional are of the lines.

Data developed from the Law College tests were subjected to further analysis with the results shown in Table A4.3-2. Assuming that the variations in K values were in fact due to the partial clogging of the pipe, equivalent pipe sizes and areas were computed as in columns (4) and (5) of Table A4.3-2. The % shown in column (6) represent the unclogged area of a 535 mm diameter pipe which would have a cross sectional area equal to the equivalent pipe noted in column (5). Column (7) shows the apparent velocity of the flow, noted in column (1), travelling through a clean, 535 mm diameter line.

Considering these measured and computed data the bottom 40 to 45 percent of the Law College force main is evidently clogged with silt and remains so until the flow reaches an apparent velocity of approximately 0.55 m/s. at velocities greater than 0.6 m/s scouring begins to take place, and the pipe is essentially cleaned out when the apparent velocity exceeds 0.7 m/s. This phenomenon is illustrated graphically in Figure A4.3-1 is the measured flow versus head loss for the 535 mm force main is plotted.

Table A4.3-2 Analysis of Law College Force Main Losses.

Q	h	k	Equivalent pipe size (for n = 0.0135)	Equivalent pipe area	Equivalent portion of unclogged pipe	Apparent Velocity
m^3/s	m		mm	sqm	%	m/s
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.071	1.8	357	430	0.145	65	0.32
0.085	3.2	443	413	0.134	60	0.38
0.111	5.49	445	412	0.134	60	0.50
0.15	4.45	198	480	0.181	81	0.67
0.17	3.4	118	529	0.220	98	0.76
0.202	4.45	109	537	0.226	101	0.90

Four conclusions can be drawn from the results of the force main tests.

Firstly, the clogging of force mains with silt is a significant problem, and the current degritting methods are not effective. The Law College force main is not an exception because equally large concentrations of grit were observed in the flow of all force mains tested. Clogging takes place rapidly because the 40 percent reduction in the cross section of the Law College force main occurred within a period of twelve hours.

Secondly, no special tools or equipment are required to remove silt from a force main in as much as subjecting the line to velocities of greater than 0.8 m/s appears to remove essentially all the accumulations.

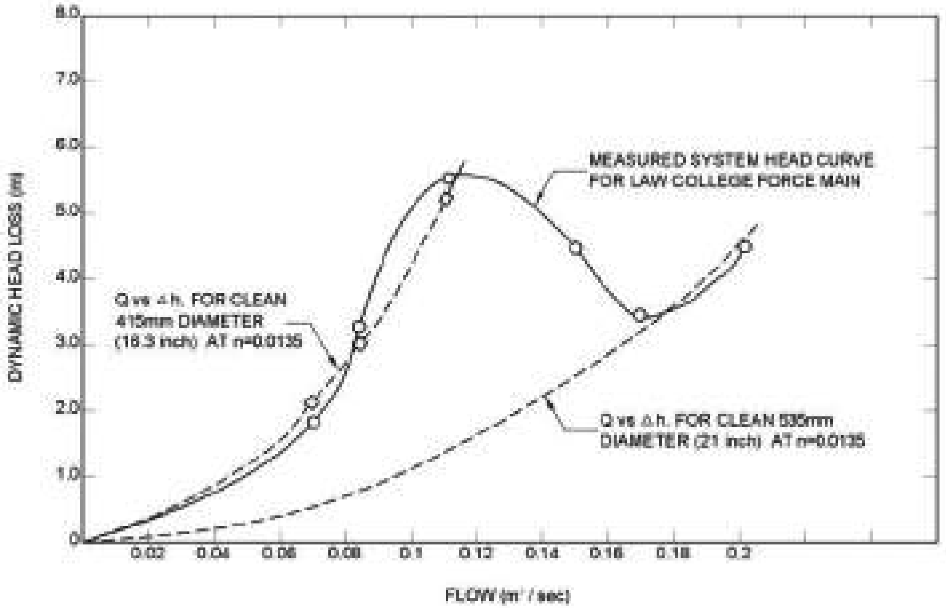


Figure A4.3-1 Measured Flow vs. head in the Law College Sewage Pumping Station

Thirdly, no economy in power cost is gained by designing the large force mains to operate at low velocities unless much more effective grit removal is practiced. Larger lines will fill; up with silt until a scour velocity is reached and then produce the friction losses commensurate with the smaller equivalent pipe.

Fourthly, force mains which have been operated at reasonably high velocities exhibit a friction loss coefficient “n” when clean similar to that of smooth wrought iron pipe. Evidently, the larger quantities of grit transported through these lines has polished their interiors. In fact Greams road force main was apparently so thoroughly polished that it responded hydraulically like a smooth brass or glass pipeline. It was therefore concluded that a friction loss co-efficient of 0.0125 is more representative of the Madras Force mains than the normally accepted values for new cast iron of 0.013.

This historical record establishes the need to ensure (a) adequate grit removal before pumping and adequate non silting velocities in the design stage itself as otherwise, like the mains reported herein, the siltation may be severely restraining the pumping main capacities in the years to come and especially in locations where the densification of population is very slow in newer habitations and eventually when the population does pick up to designed capacity, the pumping mains would have been severely choked calling for radical “cut and cure” techniques.

By a similar argument, the same can also happen to the gravity sewer collection pipes also and it will not be easy to “cut and cure” such gravity sewers.

The study also advances a case effectively on the need for incremental sewerage and non conventional options as in chapter-3 especially in the case of newer layouts.

APPENDIX A 4.4

EVALUATING OPTIONS FOR SIZING THE SEWAGE PUMPING MAIN AND PUMP SETS

1	Appendix A 4-4										
2	B	C	D	E	F	G	H	I	J	K	L
3	Condition of flow		Immediate flows			Intermediate flows			Ultimate flows		
4	Stage		low	ave	peak	low	ave	peak	low	ave	peak
5	Average flow, cum / day	Enter by Designer		1500			3800			7900	
6	Proportion	Enter by Designer	0.6	1	2.2	0.6	1	2.2	0.6	1	2.2
7	Design flow, cum / day	E5*D6, E5*E6, E5*F6	900	1500	3300	2280	3800	8360	4740	7900	17380
8	Hazen Williams C	Enter by Designer	100	100	100	100	100	100	100	100	100
9	Desired velocity, m/s	0.8, D9*E6/D6, D9*F6/D6	0.8	1.3	2.9	0.8	1.3	2.9	0.8	1.3	2.9
10	Area needed, sqm	D7/24/3600/D9	0.013	0.013	0.013	0.033	0.033	0.033	0.069	0.069	0.069
11	Dia needed, m	SQRT(D10*4/3.14)	0.129	0.129	0.129	0.205	0.205	0.205	0.296	0.296	0.296
12	Dia needed, mm	D11*1000	129	129	129	205	205	205	296	296	296
13	Radius, m	D11/2	0.064	0.064	0.064	0.102	0.102	0.102	0.148	0.148	0.148
14	Radius power 0.63	POWER(D13,0.63)	0.178	0.178	0.178	0.238	0.238	0.238	0.300	0.300	0.300
15	S power 0.54	D9/0.848/D8/D14	0.053	0.089	0.195	0.040	0.066	0.145	0.031	0.052	0.115
16	S	POWER(D15,(1/0.54))	0.004	0.011	0.048	0.003	0.007	0.028	0.002	0.004	0.018
17	Slope 1 in	1/D16	229.6	89.1	20.7	394.8	153.3	35.6	605.1	234.9	54.6
18	length, m	Enter by Designer	960	960	960	960	960	960	960	960	960
19	Friction in pipeline, m	D18/D17	4.2	10.8	46.4	2.4	6.3	27.0	1.6	4.1	17.6
20	Velocity head, m	D9*D9/2/9.81	0.033	0.091	0.439	0.033	0.091	0.439	0.033	0.091	0.439
21	Frction factor in fittings	'Appendix-4-1'!C47	72.7	72.7	72.7	72.7	72.7	72.7	72.7	72.7	72.7
22	Friction in fittings, m	D20*D21	2.4	6.6	31.9	2.4	6.6	31.9	2.4	6.6	31.9
23	Static lift, m	Enter by designer	26.5	26.5	26.5	26.5	26.5	26.5	26.5	26.5	26.5
24	Total head, m	D22+D23	28.9	33.1	58.4	28.9	33.1	58.4	28.9	33.1	58.4
25	Efficiency of pumpset	Enter by designer	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
26	Discharge, lps	D7*1000/24/3600	10.4	17.4	38.2	26.4	44.0	96.8	54.9	91.4	201.2
27	Kw required	D26*D24/D25/100.5	3.7	7.1	27.7	9.5	18.1	70.3	19.7	37.6	146.1
28	**Option 1**										
29	Provide 150 mm pipeline at zero year and another 150 mm at 11th year and another 250 mm in 21st year and change pumpsets at 11th and 21st years										
30	The augmnentation at 21st year can be decided based on realistic conditions at that time. May be suitable for smaller systems										
31	**Option 2**										
32	Provide 200 mm pipeline at zero year and provide pump sets also as for the 11th year flowrate and use the storage volume in the wet well and sewer system										
33	The augmnentation at 21st year can be decided based on realistic conditions at that time. May be suitable for very large systems										
34	**Basis for Decision**										
35	Unlike water pumping, sewage varies widely and grit adds to the problem										
36	Hence, paramount importance is to ensure minimum of 0.8 m/s all times.										
37	Each system to be weighed carefully based on calculations for that system										

APPENDIX A 5.1
ODOUR CONTROL

1 Introduction

Raw sewage must not be allowed to stagnate. If this occurs, anaerobic process sets in as shown in Figure 5.2. This in turn can generate Hydrogen Sulphide gas. It is this gas which causes foul odour problems because it smells like rotten egg. The need for control of this foul odour is the fact that this gas can be harmful to human beings. This chapter deals with the effects of this gas on human health, the locations where this gas can form and the technologies of its control.

2. Mechanism of its Entry and Effects on Human Beings

- The gas enters the body through eyes or mucous membrane of breathing organs.
- Blood seeps out from the capillaries in cavities of the lungs, causes pulmonary oedema, leading to breathing difficulties and death by suffocation.
- In sewer facilities, it is generated in rising mains with no oxygen supply and in inverted siphons, etc., where sludge is likely to accumulate easily.
- It is generated in grit chamber, pumping well, sedimentation basin, and sludge thickening tank in sewage treatment plants.
- Hydrogen sulphide generated in sewage and deposited sludge is sealed within and in the static condition, so it does not disperse to the atmosphere easily. However, when agitated, it disperses all at once to the atmosphere.

The relationship between its concentration and its toxic effect is shown in Table A5.1-1.

Table A5.1-1 Relationship between concentration of hydrogen sulphide and its toxic effects

onc. (ppm) of H2S	Effects and reaction on organ by H2S		
 0.025 0.3 3 to 5	Sense of odour Sensitive persons can sense the odour (limit of sense of odour) Anybody can sense the odour Foul unpleasant odour of medium strength		
10		Permissible concentration (lower limit for irritation of the mucous membrane of the eye)	
20 to 30	Although bearable, after getting accustomed to the odour (olfactory fatigue), any higher concentration cannot be sensed.	Breathing organs Lowest limit for irritating the lungs	
50			Eyes
100 to 300	Olfactory nerve paralysis for 2 to 15 minutes; feels like unpleasant odour has reduced.	If exposed continuously for 8 to 48 hours, bronchitis, pneumonia, and death by suffocation due to pulmonary oedema	Conjunctivitis, itchiness, pain in the eyes, feeling

onc. (ppm) of H2S	Effects and reaction on organ by H2S		
170 to 300		Scorching pain in the mucous membrane of respiratory tract; if exposure is less than 1 hour (limit), serious symptoms may not occur	of sand in the eye, glare, bloodshot eyes and swelling, turbidity of cornea, corneal damage and separation, bending and haziness of field of vision, increase in pain due to light
350 to 400		Exposure for 1 hour or more may lead to loss of life	
600		Exposure for 30 minutes hour may lead to loss of life	
700	Cerebral nerves After excessive respiration for a short period, respiratory paralysis occurs immediately thereafter		
800 to 900	Loss of consciousness, respiratory arrest, death		
1,000	Swoon, respiratory arrest, death		
5,000	Instantaneous death		

Source: JSWA, 2003

3 Locations where the gas is formed

The factors causing foul odour are unnecessary sewage stagnation and anaerobic activity. The locations where these can occur are

a) Sewers that are choked and not flowing,

b) Sewage pumping station sumps where sewage is not pumped out then & there,

c) Primary clarifiers, sludge thickeners, digesters and sludge drying beds in STPs.

Immediately on forming, the gas is however in dissolved form. When the sewage gets agitated like flowing through sewers, this gas is released into the air. At this stage, its foul odour is troublesome. Even though ammonia is also present in sewage, it does not cause any odour problem because it is present as ammonium bicarbonate salt. It is split into ammonia only during biological treatment and gets nitrified if additional oxygen is supplied. Even if it is not nitrified, its concentration is too low to cause a foul odour problem. There can be stray gases like methyl sulphide, dimethyl sulphide and methyl mercaptan, but their concentrations are usually negligible for any human discomfort.

4 Control Technologies

Odour control processes are as follows.

4.1 Odour Prevention

The objective is to reduce the number of locations and volume of odour-generating substances.

i) Sealing of locations emitting odour

Some of the methods to seal odour can be through using air-tight manhole cover, air-tight door, trap seal, air curtain.

ii) Anti- septic

In this method, the odour is controlled by restraining the decomposition of organic matter through use of sterilizer and maintaining aerobic condition through use of air and ozone.

iii) Cleaning

Debris tends to be accumulated around screen and grit removal facility and consideration is required during design to make cleaning of the structure easy.

4.2 Ventilation

Generated odour is ventilated and discharged to air by dilution and dispersion.

4.3 Deodorisation

There are many kind of deodorisation system. Optimal deodorisation system should be selected in consideration of air flow, constituents and intensity of nuisance odour, target of deodorisation, ambient environment, manageability of O&M, and economic efficiency. Consideration should be made whether central deodorisation or individual deodorisation system should be adopted in each STP or pumping stations.

4.4 Odour Enclosure

Providing a cover over the units, which produce odour, helps in containing the odour to be removed. The head room in such cases shall be a minimum 4.5 m as per the industrial safety requirements. The material of the cover can be synthetic types mounted on a funicular polygon. All materials and fasteners shall be non-corrodible. Some installations are shown in Figure A5.1-1.

(A) Figure A5.1-1 Covers over sewage structure — Flat type and Dome type (Yokohama City)

4.5 Deodorisation Processes

4.5.1 Aeration Oxidation Process (Activated Sludge Basins)

Principle

Odour-generating gas is fed into aeration tank where it is oxidized and decomposed by the action of activated sludge.

Target substances

Sulphur compound

Salient Features

Both capital and O&M costs are low. Blower needs mist filter and dust filter for protection and corrosion resistance.

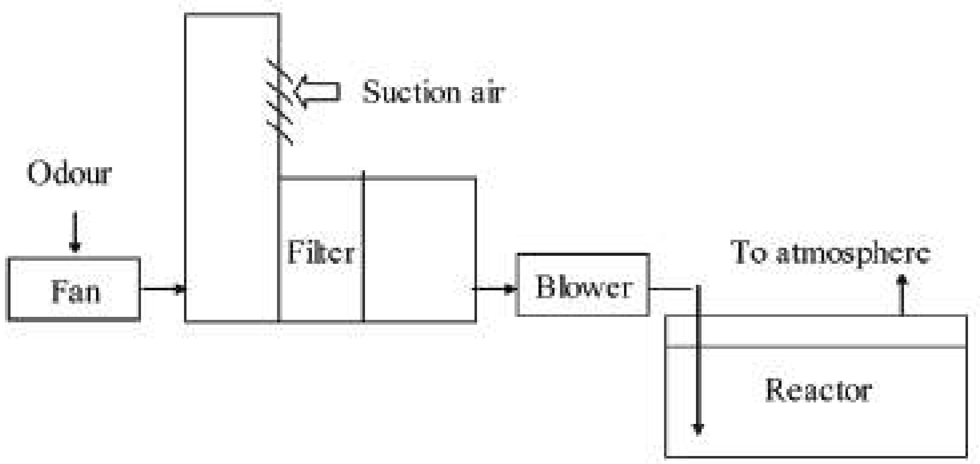


Figure A5.1-2 Schematic diagram of aeration oxidation process (activated sludge basins)

4.5.2 Soil Deodorisation System

i) Principle

Odour-generating substances are fed into soil where it is adsorbed, oxidized and decomposed by action of bacteria in soil.

Target substances

Organic substance, which is nutrient for bacteria

Salient Features

In this case, the capital cost is low but large footprint is required. Gradual consolidation of soil prevents permeability and discharging efficiency. Therefore, periodic maintenance of soil is required through ploughing and replacement.

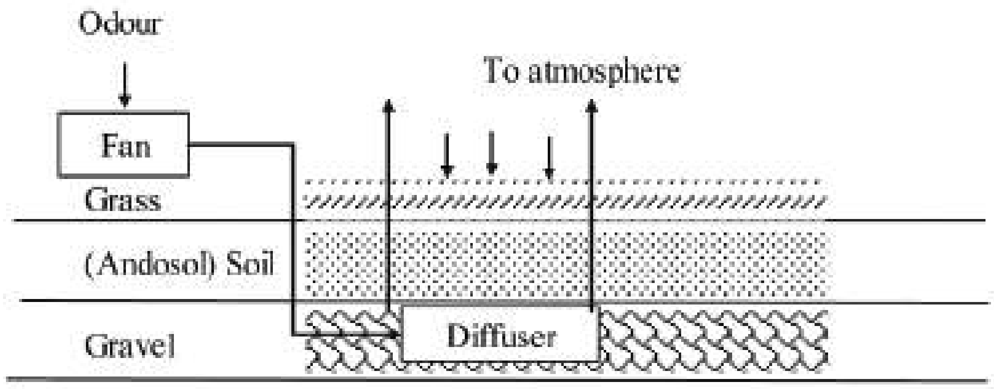


Figure A5.1-3 Schematic diagram of soil deodorisation system

4.5.3 Biofiltration

i) Principle

In this process, odour-generating substances fed to soil is adsorbed, oxidized and decomposed by action of bacteria.

ii) Target substances

Organic substance, which is nutrient for bacteria and hydrogen sulphide

iii) Salient Features

O&M cost is relatively low and footprint of equipment is small. The process is suitable for high strength of odour and acclimation period of bacteria is needed.

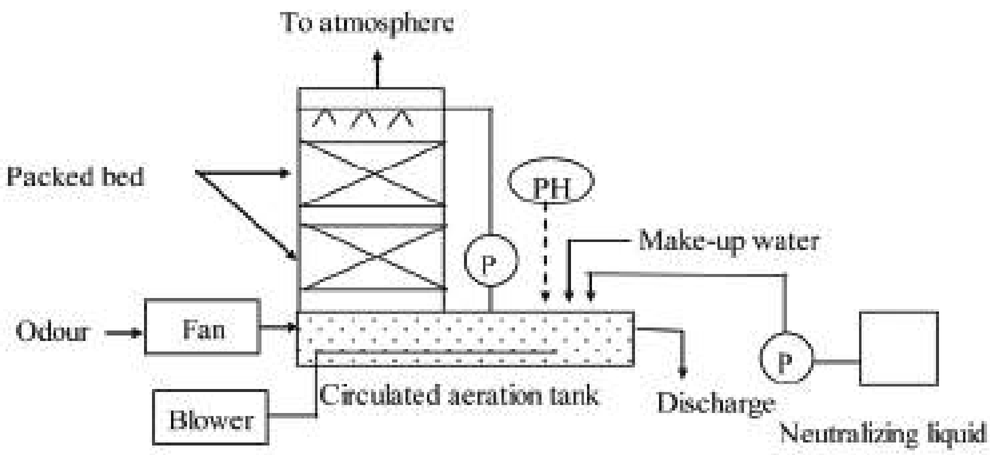


Figure A5.1-4 Schematic diagram of biofiltration

4.5.4 Water Scrubber Process

Principle

Odour is removed by contact of the odour-generating substances with water and dissolving odour.
Target substances

Ammonia, Amines and other water soluble substances

Salient Features

Both capital and O&M costs are low. This is generally used as the pre-treatment of following deodorisation process. When secondary treated wastewater is used as washing water, caution is needed because secondary treated water may emit odour.

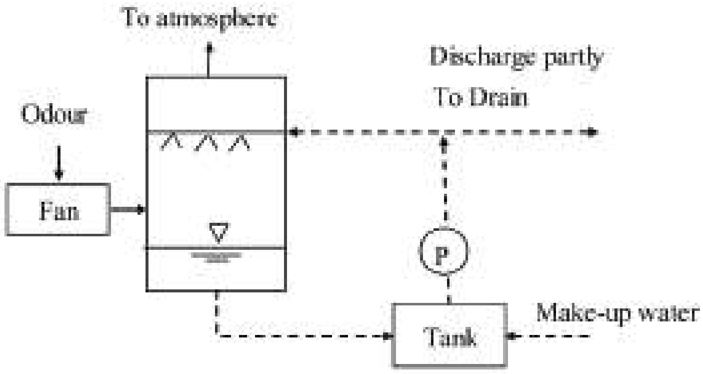


Figure A5.12-5 Schematic diagram of water scrubber process

4.5.5 Activated Carbon Process

i) Principle

Odour-generating substances are removed by adsorbing physically and chemically.

ii) Target substances

Hydrogen sulphide, Methyl sulphide, Ammonia, Trimethylamine

iii) Salient Features

Activated carbon is relatively expensive and characterised by high pressure loss. Periodically exchange or regeneration of activated carbon is necessary. Mist and dust in gas need to be removed. This system is suitable for low strength odour.

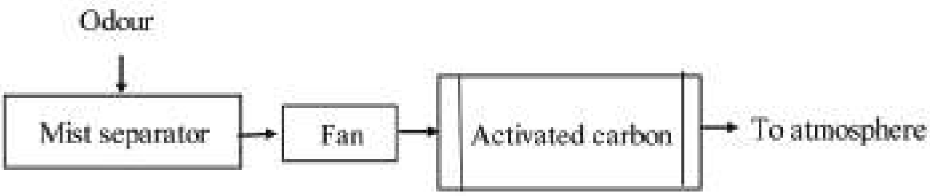


Figure A5.1-6 Schematic diagram of activated carbon process

4.5.6 Ion Exchange Resin

i) Principle

By passing odour-generating substance through ion exchange resin, odour-generating substances are removed by chemical adsorption of alkaline and acid substances and physical adsorption of neutral substances.

ii) Target substances

Almost all odour-generating substances

iii) Salient Features

Resin is relatively costly and pressure loss is large. Regeneration of resin is rather easy. Sometimes activated carbon process and ion exchange process are configured in series.

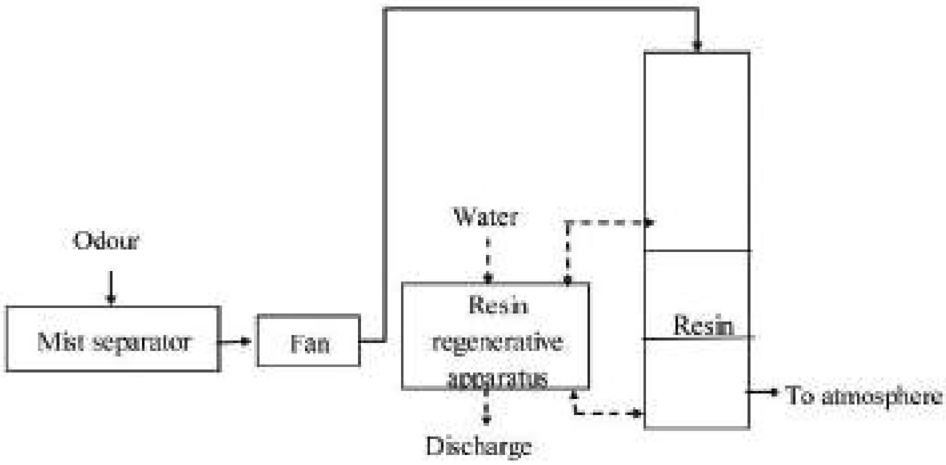


Figure A5.1-7 Schematic diagram of ion exchange resin

4.5.7 Chemical Oxidation Process

Principle

Odour-generating substances are removed by the oxidation action of oxidants such as sodium hypochlorite, and chlorine water.

Target substances

Oxidisable substances

Salient Features

In case if the exhausted gas contains chlorine, absorption equipment with alkaline solution is needed.

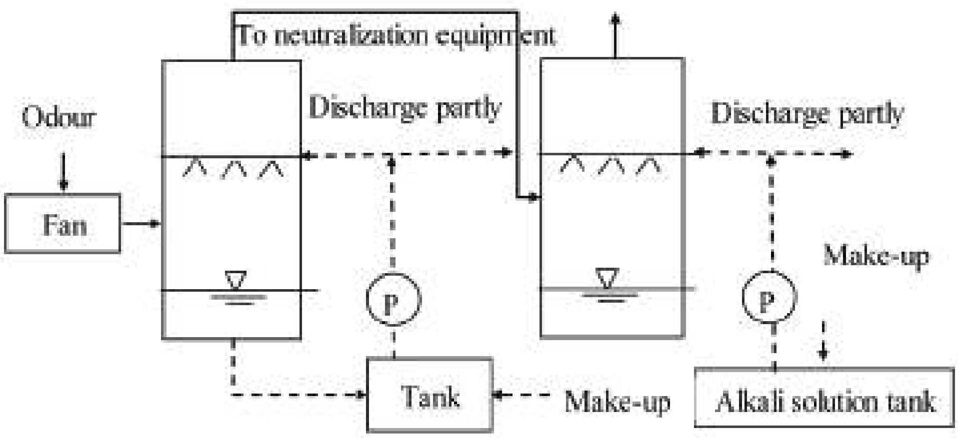


Figure A5.1-8 Schematic diagram of chemical oxidation process

4.5.8 Acid and Alkaline Scrubber Process

i) Principle

In acid scrubber process, odour-generating substances are kept in contact with hydrochloric acid or sulphuric acid, and are removed by neutralization reaction. In alkaline scrubber process, odour-generating substances are kept in contact with sodium hydroxide, and are removed by neutralization reaction.

ii) Target substances

Ammonia, Amines (acid scrubber process), Hydrogen sulphide, Methyl mercaptan (alkaline scrubber process)

iii) Salient Features

In this process, neutralization equipment is needed. Since there are many contact methods between chemical and odour-generating substances, close examination is needed in their selection. It is important to mention that pH of solvent influences the efficiency of deodorisation.

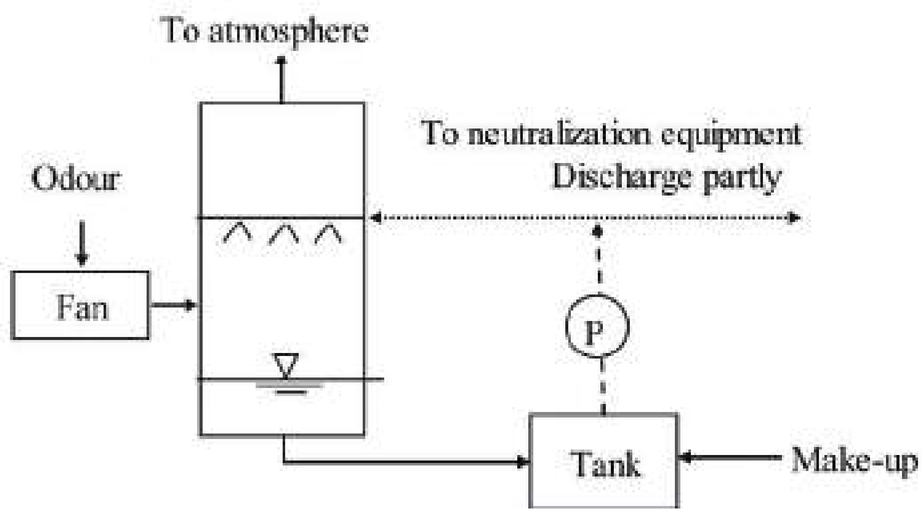


Figure A5.1-9 Schematic diagram of acid and alkaline scrubber process

4.5.9 Direct Combustion Process

i) Principle

Odour-generating substances are combusted and decomposed in incinerator at a very high temperature of approximately 800°C.

ii) Target substances

Almost all of odour-generating substances

iii) Salient Features

In case when ventilated odour-generating substances from various facilities are used as inflow to the incinerator, capital cost and operation cost would be economical. In case of individual combustion, capital cost and operation cost would be high. In this process, the temperature of air flow has to be raised high, whatever the concentration of the odour-generating substance may be. Therefore, small air flow with high concentration of odour-generating substance has advantage and higher removal efficiency. However, oxygen concentration in odour- generating gas should not be too low and SOx and NOx concentration in odour-generating gas should be examined.

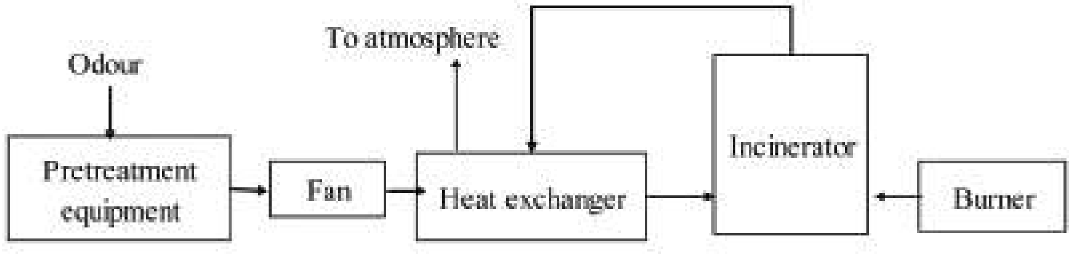


Figure A5.1-10 Schematic diagram of direct combustion process

4.5.10 Catalytic Combustion Process

i) Principle

In this process, odour-generating substances are heated up and destroyed at a temperature of approximately 350 °C by heat exchanger using incinerator in presence of catalyst such as platinum and vanadium.

ii) Target substances

Almost all the odour-generating substances

iii) Salient Features

Fuel consumption in this process is lower than that of the direct combustion process. This process is advantageous in case of high-concentration odour-generating substances below the explosion limit. If oily smoke is present, it sticks on the surface of catalyst, and reduces the activation, so it needs to be washed and removed once or twice a year.

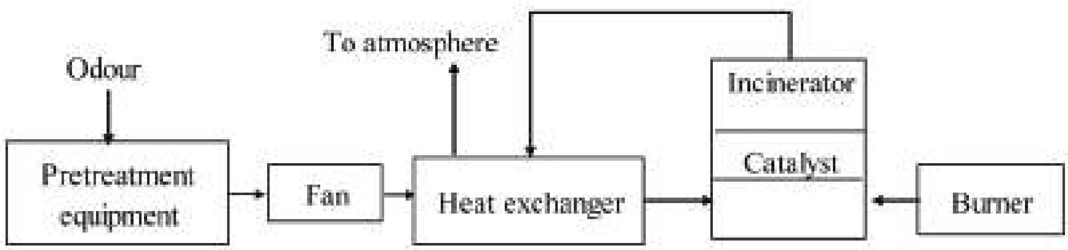


Figure A5.12-1 Schematic diagram of catalytic combustion process

4.5.11 Ozone Oxidation Process

i) Principle

Odour-generating substances are removed by the oxidation action of ozone.

ii) Target substances

Odour-generating substances with low concentration and a large volume (except ammonia)

iii) Salient Features

Ozone is harmful and with sharp smell. Monitoring of excessive ozone residue in treated gas is needed and if necessary, activated carbon system can be installed for removal of ozone. When the odour-generating substances are in wet situation, efficiency of removal will be higher.

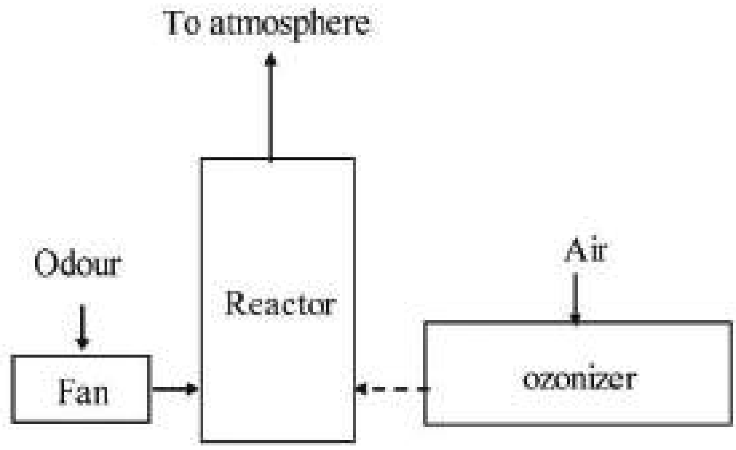


Figure A5.1-12 Schematic diagram of ozone oxidation process

4.6 Methods of Covers

1) Primary covers

Primary covers are installed near the water surface of tanks. These are well used for locations where odour is comparatively strong, such as primary sedimentation basin and sludge thickening tank, and in cases where upper parts are not used.

2) Secondary covers

Building is installed on tanks etc.

3) Double covers

This combines 1) and 2).

After collecting high-concentration odour with few quantity of air as much as possible, it is more economical to deodorize and effective. Therefore, non-working clearance of the water surface of tanks and covers, and working clearance of floors, ceilings, and walls should be necessary minimum in order to carry out operation and maintenance of the facilities.
Typical methods are shown in Figure A5.1-13.

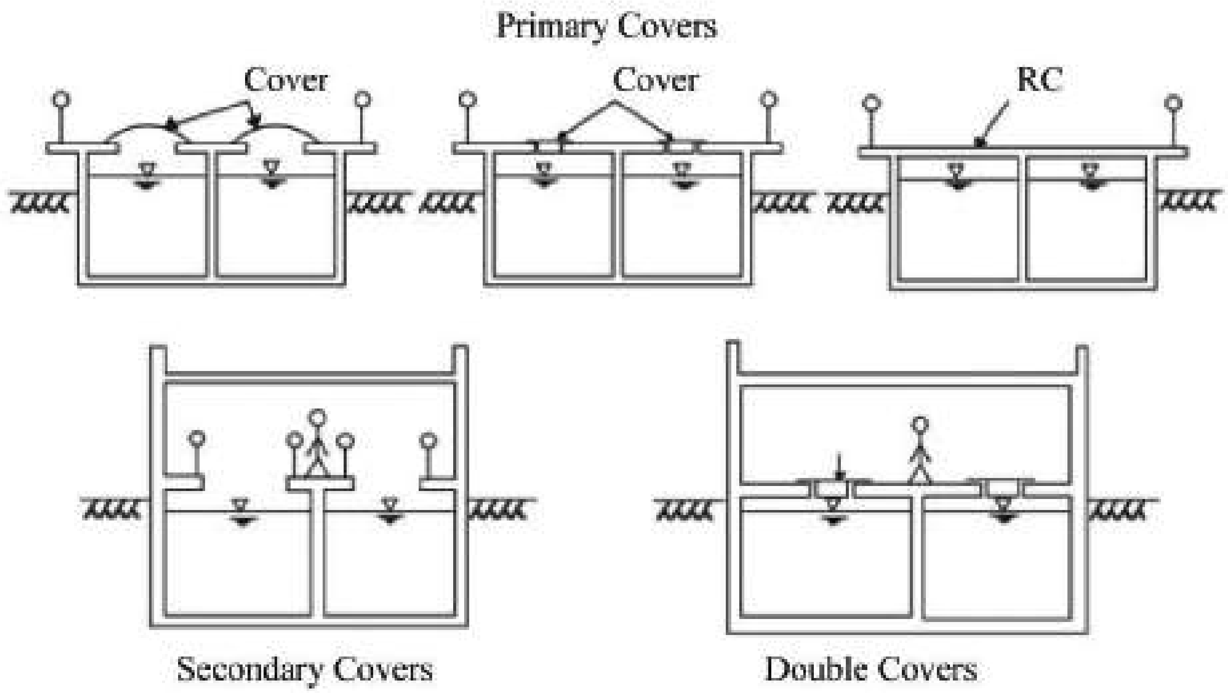
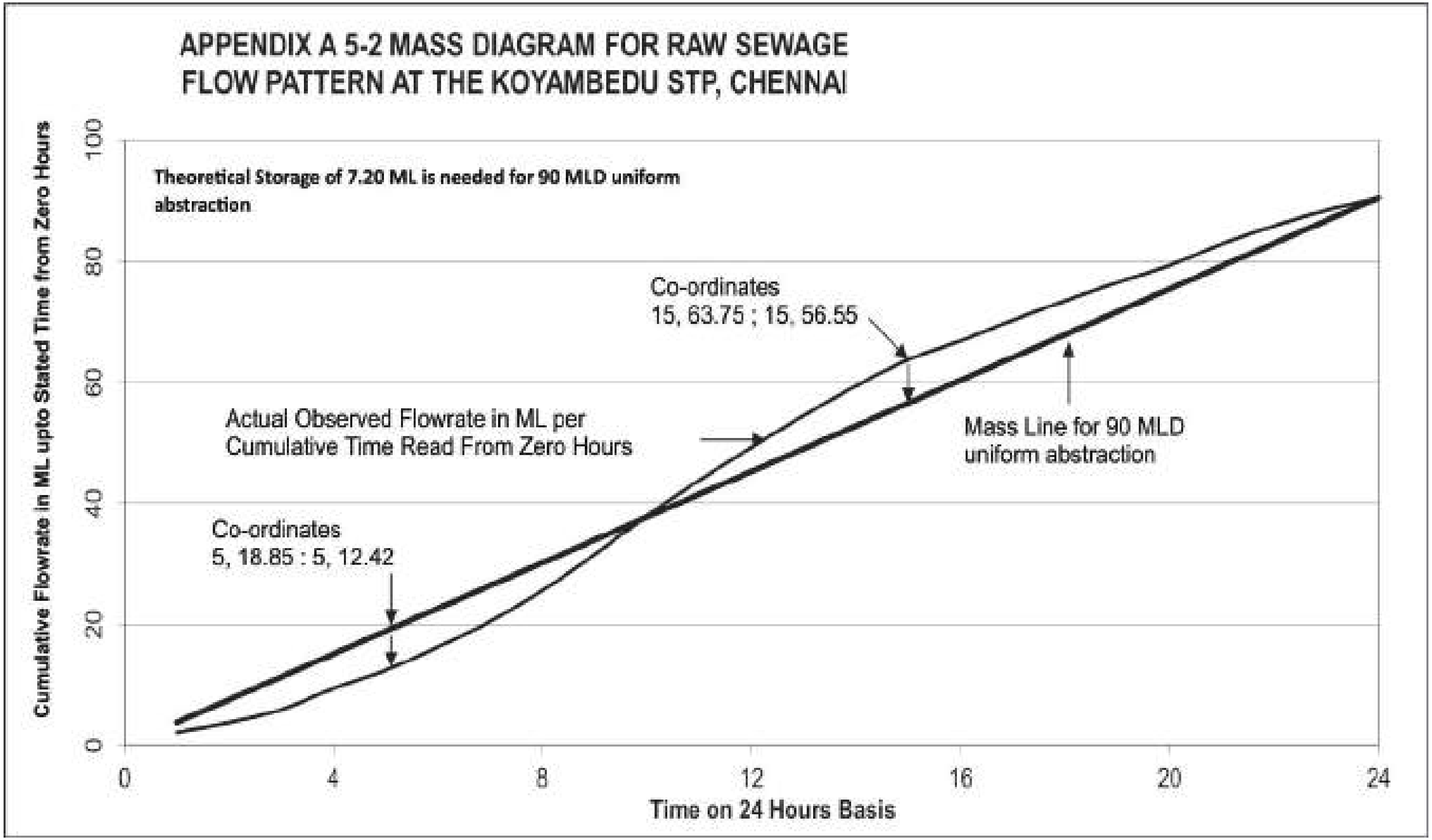


Figure A-5-1-13 Methods of providing domes over odour producing units in STPs

APPENDIX A 5.2
MASS DIAGRAM FOR RAW SEWAGE FLOW PATTERN AT THE KOYAMBEDU STP, CHENNAI



APPENDIX A 5.3
LABORATORY

5.3.1 General

A well designed and adequately equipped laboratory under a competent analyst is essential in all sewage treatment plants. Very small size plants such as stabilization ponds need not have their own laboratories if the facilities of a nearby laboratory are available. The results of the laboratory analysis will aid in the characterization of any waste water, pinpoint difficulties in the operation and indicate improvement measures, evaluate the composition of effluents and thus estimate the efficiency of operation and also measure the pollution effects of the discharge of such effluents upon the receiving water bodies.

The analytical data accumulated over a period to time is an important document in safeguarding the treatment plant from allegations of faulty operation. The laboratory should also engage in research and special studies for evolving improvements and innovations in the plant operation. The laboratory therefore, must form an integral part of the treatment plant.

5.3.2 Planning of Laboratory Facilities

5.3.2.1 Physical Facilities

The actual design of the laboratory depends on the size and type of treatment plants and type and volume of analytical work required to be carried out. Due consideration, therefore should be given to the space requirement for permanent installed equipments and smooth performances or analytical work by the personnel. Necessary provision for future expansions should also be incorporated in the laboratory design.

5.3.2.2 Size of the Laboratory

The size and equipments needed for the laboratory depends on the capacity of the STP. Even the smallest STP shall be provided with a laboratory, where at least a few simple analyses such as SS, pH, BOD and residual chlorine can be made. On the other hand large STP providing complete treatment may require a well planned laboratory building with facilities for physical, chemical, biological and bacteriological work.

A recommended layout for a STP control laboratory of about 25 mld treatment capacity is presented as Figure A 5.3-1. The total area of the laboratory is about 130 sqm with a small toilet hall and wash room. It includes the main laboratory hall of 75 sqm with work benches and smaller rooms of about 13 sqm each. One of these rooms can be used as the office and the other can be used as a balance room or instrument room etc. The laboratory should have a separate emergency exit.

5.3.2.3 Location

The laboratory should be easily accessible from any unit of the plant and so located as to provide adequate natural lighting (preferably north light) and ventilation, it should be away from pumps and other heavy operating machinery.

5.3.2.4 Floor Space

Minimum floor space required accommodating the equipment necessary to be installed in the room and to avoid interference in the work should be provided. The width of walkways between rows of tables or equipments should be not less than 1m but preferably 1.2m. Total floor space requirement of any work room should be arrived at by accounting for space requirement for all equipments and their placement and the number of staff utilizing the room.

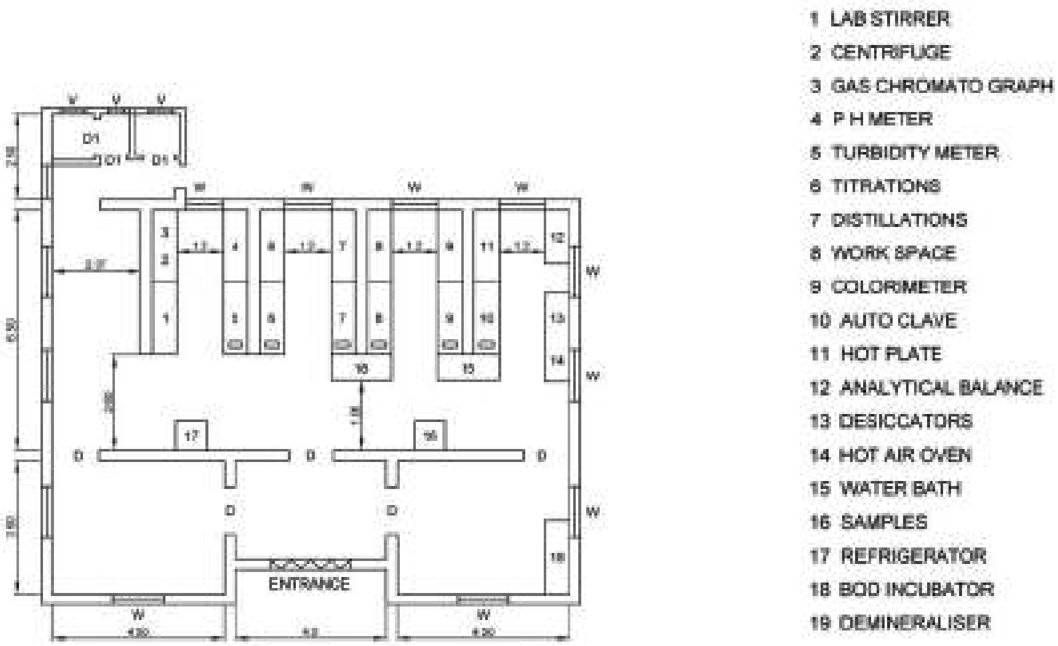


Figure A 5.3-1 Typical layout and list of equipments for sewage treatment plant control laboratory

5.3.2.5 Walls

Walls should be; finished smooth in light colours. The wall space and offsets should be convenient to locate cabinets, benches, hoods, incubators, alongside, without any loss of floor space.

5.3.2.6 Lighting

All work rooms in a laboratory including stairways and passages should be well lighted. The window areas in terms of floor area should not be less than 20 percent and all windows should be fitted with transparent glass panels. Long windows should be preferred to broad windows for greater depth of penetration of light into work rooms. North-South facing should be preferred for prevention of glare on work tables and benches. There must be adequate artificial lighting to supplement day-light, well distributed to provide uniform general lighting with minimum shadow effects. Spot lights should be provided to specific equipment and instruments such as weighing balances, hoods, etc. Adequate number of plug points should be provided for extra lighting and equipment when required,

5.3.2.7 Power Supply

Adequate electric power supply for at least 200 amps at L.T. voltage is required. Many laboratory equipments require higher voltage and provision for such exigencies should be made.

It is also desirable to provide suitable voltage stabilizers to protect sophisticated equipment from damage due to wide fluctuations in the line voltage. This may require consideration in terms of individual units or for the laboratory as a whole.

5.3.2.8 Floor

Floors should be of smooth finish but not slippery and should be easy to wash and keep clean. Concrete flooring with mosaic finish and dadooing up to window sill level is recommended.

5.3.2.9 Work Tables and Benches

A provision of 10 m space of work tables and benches per worker should be sufficient. These tables should be preferably located along the walls. Tables located in any other position should have a clear gangway of width not less than 1 m. between adjacent rows. Wall side tables are generally kept 60 to 75 cm wide and centre tables are designed 140 cm wide to allow work space on both sides. Height of tables should be 90 to 95 cm for working in a standing posture and 75 to 80 cm for working in a sitting posture. Table tops should be finished smooth with acid resistant tiles/sheets. A separate rigid table of size 120 cm x 60 cm with revolving adjustable stool should generally be provided for analytical balance. Adequate number of stools should be provided along with work tables and benches. Drains connected to table sinks should also be resistant to attack from corrosive substances.

5.3.2.10 Reagent Cabinets and Cupboards

These should be provided in adequate number and size for storing chemicals and reagents and stock solutions, etc. in a systematic order. Sliding glass panelled shutters should be preferred to hinge shutters in these cabinets. The laboratory tables could be provided with cupboards and open glass shelves on the top to provide additional space for storage of chemicals and stock solutions.

5.3.2.11 Sinks

Both table sinks and separate sinks with adequate water supply shall be provided. Table sinks are fitted with gooseneck taps extending high enough above the table to permit washing of litre cylinders. Separate sinks of sufficient size and depth located at suitable points shall also be provided for washing the glassware. Plumbing to sinks and wash basins shall be of proper design and of corrosive resistant materials like PVC or ceramic for waste water lines.

5.3.2.12 Fume Hoods and Chambers

Fume hoods and chambers are necessary to prevent spreading of toxic and irritant fumes and odours into other parts of the laboratory and also to prevent condensation of walls, windows and other fixtures causing corrosion. Some analytical work needs isolated fume chambers while other could he carried out under an exhaust hood. Positive ventilation with exhaust tans are generally provided for this purpose. Hoods are designed as per standard practice to provide a minimum air velocity of 30 linear m/min.

5.3.2.13 Gas Supply

The plant should provide its own gas supply to the laboratory by installing a gas plant.

Efforts should be made to use digester gas if sludge digesters are installed. Gas should be piped to main work tables with hoods with appropriate fixture outlets. Compressed cooking gas in cylinders can also be used.

5.3.2.14 Space for Analytical Balance

The analytical balance mounted on a small rigid table to be used in sitting position may be provided in a separate cubicle or enclosure in bigger laboratories. It may also be possible to provide a masonry platform with top surface of polished stone for mounting the balance.

5.3.2.15 Constant Temperature Room

In large plants, provision is sometimes made for constant temperature rooms maintained at 20 deg C for performances of BOD and other tests. If this is not available commercial type 20 deg C BOD incubator may be used.

5.3.2.16 Sample Preparation Room

In large plants employing both primary and secondary processes where number of samples handled daily is large and so a separate sample preparation room con be very useful. Such room should have refrigerators of suitable capacities. In addition, an attached cold room with storage facilities may also be necessary particularly where bacteriological work is done.

5.3.2.17 Media Preparation and Sterilization Room

In large STPs where continuous bacteriological analysis is done, additional facilities for media preparation, centrifuging sterilization by autoclaves, etc. are necessary and additional rooms for accommodating these facilities should also be included. Such rooms are usually attached to the laboratory and are located within easy reach of the analysts.

5.3.2.18 Space for Records

Space for keeping laboratory and plant records should be provided in the laboratory office or in the plant administrative block.

5.3.2.19 Wash and Toilet Facilities

Adequate toilets and wash basins should be provided separately for men and women. Emergency showers should also be provided which can be housed in the work room itself with a curtain to provide temporary privacy. Emergency foot operated spout type eyewash should also be installed in the workroom.

5.3.3 Equipment and Chemicals

5.3.3.1 Equipment Required

The type of equipment required for sewage treatment plant laboratory depends on the type of plant, the type of analytical work to be caused out and the frequency of each test to be performed. It is advisable to make initial decisions on the specific analysis to be undertaken, the number of samples, the frequency of sampling and the staff requirement to carry out these analysis, so as to avoid unnecessary purchases and keeping of equipment idle for an indefinite period. Equipment that is not used and is kept idle is often neglected and fall into disuse. Hence, selection of equipment for the plant laboratory requires most useful and careful planning, so that each equipment bought is specifically on the basis of anticipated function and availability of trained staff.

A list of important equipments required for carrying out several analytical works in a laboratory is given in Appendix A 5.4. The list is not exhaustive, but covers most of the requirements. The quantities required have to be decided as suggested above.

The estimates of essential consumable articles such as chemicals, glassware etc. and recurring replacement in the succeeding years of operation must be worked out with utmost care on the basis of the particular treatment processes to each plant. A list of important tests is given in Appendix A 5.5 which will serve as a guideline for choosing the required glassware and chemicals for a particular STP.

Refrigerators for reagents and deep freezes provided for preserving samples should be adequate in capacity and numbers.

All equipment needs a certain amount of maintenance care, particularly those that are electrically operated. Periodic servicing of equipment and checking for their efficiency will save the loss of equipment and prevent faulty analysts leading to work interruptions.

5.3.3.2 Storage

All glassware should be stored in an orderly way and used with care to minimize loss due to breakages in handling, Glassware should he cleaned thoroughly after their use and dried before placing in the cupboards and lockets.

Chemicals should be stated in proper shelves and lockets. Toxic chemicals such as arsenic, cyanide etc. should he kept under lock and key and should be under the direct charge of a senior analyst who issues and accounts for them, Acids, bulky glassware etc. which can cause accidents and burns by dropping on the floor should not be stored on high shelves., which need ladders or high stools to reach them.

Chemicals that have a limited life should be bought in such quantities as can be used before their potency is lost.

A stock register for all equipment, chemicals and glassware should be maintained in all laboratories and kept up-to-date.

APPENDIX A 5.4
MINIMUM LABORATORY EQUIPMENTS NEEDED FOR TESTS

Equipment	Type of Plant	
	5MLD	>5 MLD
Analytical Balance	x	x
Autoclave	--	x
Centrifuge	--	x
Chlorine comparator	x	x
Colony counters	--	x
Demineraliser	x	x
Dissolved Oxygen sampler	x	x
Drying oven (hot air)	x	x
Fume cupboards	x	x
Gas liquid chromatograph	--	x
Hot plates	x	x
Incubator 20°C (BOD)	x	x
Incubator 30°C (Bacteriological)	--	x
Kjeldahl Digester Unit	x	x
Magnetic stirrers	x	x
Microscope, binocular with oil immersion and movable stage counting cell	--	x
Membrane Filter Assembly	--	x
Muffle Furnace	x	x
Orsat or equivalent gas analysis apparatus		x
pH comparator (Colorimetric)	x	x
pH meter with reference & spare electrodes	--	x

Equipment	Type of Plant	
	5MLD	>5 MLD
pH meter portable	x	x
Refrigerator	x	x
Sedwick Rafter funnel	--	x
Sludge sampler	--	x
Soxhlet extraction unit	--	x
Spectrophotometer (atomic absorption)	--	x
Spectrophotometer with or without U-V rage or photo electric colorimeter	--	x
Total organic carbon analyser	--	x
Turbidimeter	x	x
Vacuum pump	x	x
Water bath (thermostat controlled)	x	x

APPENDIX A 5.5

TESTS RECOMMENDED TO BE CARRIED OUT ON UNITS OF SEWAGE TREATMENT PLANTS

	Treatment stage/Unit	Total Suspended Solids	Settleable Solids	Dissolved Solids	Mixed Liqueur Suspended Solids (ML SS)	SVI for ML	Turbidity	pH	Alkalinity	Volatile Acids	BOD	COD	DO	ORP	Total Kjeldahl Nitrogen
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	Raw sewage	x	x	x				x	x		x	x			x
2	Primary sedimentation tanks influent and effluent	x	x									x			
3	Attached/fluidized/immobilized media influent & effluent	x						x			x	x	x		x
4	Activated sludge aeration tank influent & effluent										x	x	x	x	
5	Above tank contents				x	x									
6	Effluent of secondary settling tank	x	x						x		x	x	x		x
7	Influent & effluent of septic tanks	x	x	x							x	x			
8	Above tank contents							x	x	x					
9	Digester contents							x	x	x					
10	Primary sludge														
11	Secondary settled sludge														
12	Digested sludge							x	x	x					
13	Sludge digester supernatant	x		x				x	x		x	x			x
14	Stabilization ponds influent & effluent	x					x				x	x	x		
15	Above pond contents							x	x				x	x	

APPENDIX A 5.6
EXAMPLE FOR HYDRAULIC DESIGN OF MECHANICALLY CLEANED BAR RACK AND SCREEN CHAMBER
(Retained as in 2nd edition)

1 PROBLEM STATEMENT

Design a bar rack and screen chamber for a peak design flow of 150 MLD (3×average sewage flow of 50 MLD) with the following data.

Peak design flow	=	1.736 m^3/s
Flow conditions in incoming trunk sewer		
Diameter of incoming sewer	=	1.40 m
Depth of flow in sewer at peak flow	=	1.05 m
Velocity in sewer at peak design flow	=	1.16 m/s
Drop of screen chamber floor to invert	=	0.08 m
Assumed width of rectangular bars	=	10 mm
Clear spacing between bars	=	25 mm

Sketch a hydraulic profile through bar rack under clean conditions as well as for 50 percent clogged conditions.

2 SOLUTION

2.1 DESIGN OF BAR RACK

Assume depth of flow in screen chamber	=	1.05 m
Assume velocity of flow through rack Openings	=	0.9 m/s

$$\textit{Clear area of openings through the rack} = \frac{Q}{V}$$

$$= \frac{1.736}{0.9} = 1.929\ m^2$$

$$\textit{Clear width of openings through the rack} = \frac{1.929}{1.05} = 1.84\ m$$

Provide 73 clear spacing of 25mm each

Number of bars = 72 of 10mm each

$$\textit{Total width of the screen chamber} = \frac{73\times 25}{1000} + 72\times\frac{10}{1000} = 2.545\ m$$

2.2 ACTUAL DEPTH OF FLOW IN SCREEN CHAMBER AT PEAK FLOW

The longitudinal section of the screen chamber is divided into four sections. The section 1 is at sewer, section 2 at screen chamber u/s of bar rack, section 3 at d/s of bar rack and section 4 u/s of the outlet of screen chamber. It is assumed that the outlet channel/sewer from screen chamber discharges freely into the sump well. The definition sketch is given in Figure A5.6-1.

Applying Bernoulli's theorem between sections 1 and 2

$$Z_1 + d_1 + \left(\frac{V_1^{\,2}}{2g}\right) = Z_2 + d_2 + \left(\frac{V_2^{\,2}}{2g}\right) + h_L$$

Where,

Z_1 & Z_2	=	datum heads
d_1 & d_2	=	depths of flow at sections 1 and 2
V_1 & V_2	=	velocities of flow at sections 1 and 2
h_L	=	head loss due to sudden expansion from sewer to screen chamber

$$= K_e\left[\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right]$$

Taking floor of the screen chamber as datum (Z_2 = 0) and assuming K_e = 0.3 for coefficient of expansion,

$$0.08 + 1.05 + \left(\frac{1.16^2}{2\times 9.81}\right) = 0 + d_2 + \frac{\left(\frac{1.736}{2.545 d_2}\right)^2}{2\times 9.81} + \frac{0.3}{2\times 9.81}\times\left[1.16^2 - \left(\frac{1.736}{2.545 d_2}\right)^2\right]$$

$$d_2^{\,3} - 1.178\, d_2^{\,2} + 0.0166 = 0$$

Solving by trial and error,

d_2 = 1.17 m

$$V_2 = \frac{1.736}{2.545\times 1.17} = 0.583\, m/s$$

2.3 VELOCITY THROUGH CLEAR OPENING OF BAR RACK

$$V = \frac{Flow}{Net\ area\ of\ opening\ through\ rack}$$

$$= \frac{1.736}{73\times 0.025\times 1.17} = 0.813\, m/s$$

The velocity through the bar rack was assumed to be 0.9 m/s but it is actually 0.81 m/s. If desired, the steps I, II and III can be revised to yield different values of number of bars, depth of flow and velocity of flow, etc. However as V is within range (0.6-1.2 m/s), these steps are not being revised, being acceptable.

2.4 HEAD LOSS THROUGH BAR RACK

$$h = 0.0728\left(V^2 - V_2^{\,2}\right) = 0.0728\times\left(0.813^2 - 0.583^2\right)$$

$$= 0.024\, m$$

Using Kirschmer's Formula

$$h = \beta \left(W / b\right)^{\frac{4}{3}} h_V \sin\theta$$
$$= 2.42 \times \left(\frac{72 \times 10}{73 \times 25}\right)^{\frac{4}{3}} \times \left(\frac{0.813^2}{2 \times 9.81}\right) \sin 75°$$
$$= 0.022\ m$$

2.5 DETERMINE DEPTH AND VELOCITY OF FLOW D/S OF BAR RACK

Applying energy equation between sections 2 and 3

$$Z_2 + d_2 + \left(\frac{V_2^{\ 2}}{2g}\right) = Z_3 + d_3 + \left(\frac{V_3^{\ 2}}{2g}\right) + h$$

when bar rack is clean

$$0 + 1.17 + \left(\frac{0.583^2}{2 \times 9.81}\right) = 0 + d_3 + \frac{\left(\frac{1.736}{2.545 \times d_3}\right)^2}{2 \times 9.81} + 0.024$$

$$d_3^{\ 3} - 1.163 d_3^{\ 2} + 0.0237 = 0$$

$$d_3 = 1.15\ m$$

$$V_3 = \frac{1.736}{2.545 \times 1.15} = 0.593\ m/s$$

2.6 HEAD LOSS THROUGH BAR RACK AT 50% CLOGGING

Assuming d_2^1 and V_2^1 as depth and velocity of flow at section 2 when bar rack is 50% clogged,

$$d_2^1 + \frac{\left(V_2^1\right)^2}{2g} = d_3 + \frac{V_3^{\ 2}}{2g} + h_{50\%}$$

$$h_{50\%} = 0.0728\left(\left(Velocity\ through\ c\log ged\ rack\right)^2 - V_2^{\ 2}\right)$$
$$= 0.0728\left[\left(\frac{1.736}{73 \times 0.025 \times 0.5 \times d_2^1}\right)^2 - \left(\frac{1.736}{2.545 d_2^1}\right)^2\right]$$
$$= \frac{0.23}{\left(d_2^1\right)^2}$$

Therefore,

$$d_2^1 + \frac{\left(\frac{1.736}{2.545 \times d_2^1}\right)^2}{2 \times 9.81} = 1.15 + \frac{0.593^2}{2 \times 9.81} + \frac{0.23}{\left(d_2^1\right)^2}$$

$$\left(d_2^1\right)^3 - 1.168\left(d_2^1\right)^2 - 0.206 = 0$$

$$V_3^1 = \frac{1.736}{1.30 \times 2.545} = 0.525\ m/s$$

Head loss under 50% clogging of bar rack,

$$h_{50\%} = \frac{0.23}{(1.30)^2} = 0.136\ m < 0.15\ m \quad hence\ \ OK$$

2.7 FLOOR RAISING REQUIRED IN CHANNEL BEFORE FREE FALL INTO SUMP WELL

If the flow d/s of bar rack has to be designed for free fall conditions into the adjoining sump well of pumping station, it is obvious that critical flow conditions will prevail near the outfall.

Depth of critical flow,

$$d_c = \left(\frac{Q^2}{gb^2}\right)^{\frac{1}{3}}$$

$$= \left[\frac{(1.736)^2}{9.81 \times (2.545)^2}\right]^{\frac{1}{3}} = 0.362\ m$$

Critical velocity,

$$V_c = \frac{1.736}{2.545 \times 0.362} = 1.88\ m/s$$

In order not to disturb the existing hydraulic profile at section 3 and beyond, the floor of the screen chamber has to be raised by an amount Z_c, which can be determined by applying Bernoulli's Theorem between sections 3 and 4.

$$Z_3 + d_3 + \left(\frac{V_3^{\,2}}{2g}\right) = Z_4 + Z_c + d_4 + \left(\frac{V_4^{\,2}}{2g}\right) + head\ \ loss$$

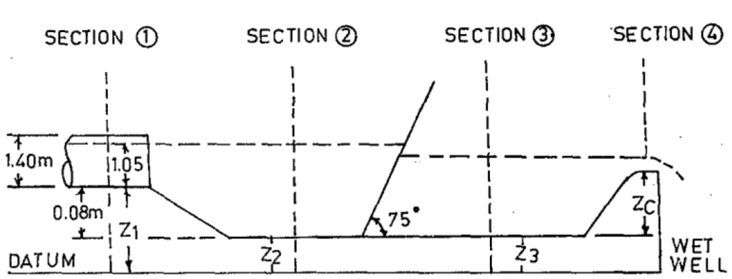
Since $Z_3 = Z_4$, $d_4 = d_c = 0.362$ m, $V_4 = V_c = 1.88$ m/s and neglecting head loss,

$$0 + 1.15 + \frac{0.593^2}{2 \times 9.81} = 0 + Z_c + 0.362 + \frac{1.88^2}{2 \times 9.81} + 0$$

$$Z_c = 0.625\ \text{m}$$

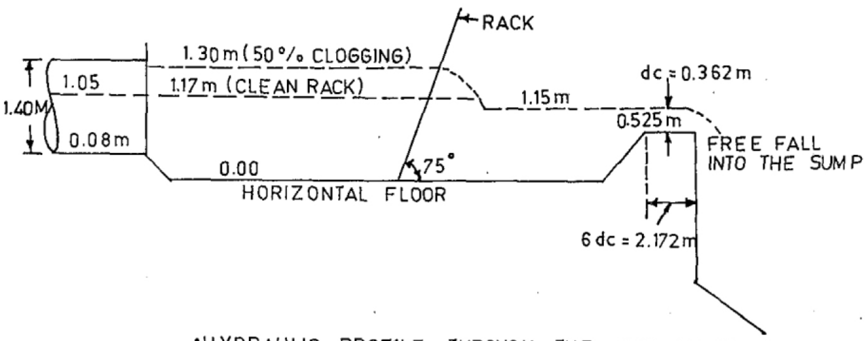
2.8 HYDRAULIC PROFILE

Hydraulic profile through the bar racks for clean conditions as well as for 50% clogged conditions is presented in the following Figure A 5.6-2.



:DEFINITION SKETCH FOR SECTIONS OF BAR SCREEN DESIGN

Figure A 5.6-1



:HYDRAULIC PROFILE THROUGH THE BAR RACK AT PEAK DESIGN FLOW WHEN RACK IS CLEAN & AT 50% CLOGGING

Figure A 5.6-2

APPENDIX A 5.7
DESIGN EXAMPLE FOR GRIT CHAMBER WITH PROPORTIONAL FLOW WEIR AS HYDRAULIC CONTROL DEVICE
(Retained as in 2nd edition)

1 PROBLEM STATEMENT

Design grit chamber to treat peak design flow of 150 MLD (3×Average wastewater flow of 50 MLD) of wastewater to remove grit particles up to a size of 0.15 mm and of specific gravity of 2.65. The minimum temperature is 15ºC. The grit chamber is equipped with proportional flow weir as control device.

2 SOLUTION

2.1 COMPUTATION OF SETTLING VELOCITY

Applying Stoke's Law,

$$V_s = \frac{g}{18}(S_s - 1)\frac{d^2}{v}$$

Given S_s = 2.65, d = 0.15×10^{-3} m

$$v = 1.14\times10^{-6}\ m^2/s \quad at\ 15\ ^\circ C$$

$$V_s = \frac{9.81}{18}\times(2.65-1)\times\frac{(0.15\times10^{-3})^2}{1.14\times10^{-6}} = 0.018\ m/s$$

Check for Reynold's Number, R

$$R = \frac{V_s d}{v} = \frac{0.018\times0.15\times10^{-3}}{1.14\times10^{-6}} = 2.37 > 0.5$$

Hence Stoke's law does not apply

Applying Transitions Law for $0.5 < R < 10^3$

$v_s = [\,0.7.07\ (S_s - 1)_d^{\,1.6}{}_v^{\,-0.6}\,]\ 0.714$

$= [\,0.707 \times (2.65-1) \times (0.15 \times 10^{-3})^{1.6} \times (1.14 \times 10^{-6})^{-0.6}\,]\ 0.714$

$= 0.0168$ m /s

2.2 COMPUTATION OF SURFACE OVERFLOW RATE, SOR

The surface overflow rate for 100% removal efficiency in an ideal grit chamber

= Settling velocity of the minimum size of particle to be removed

= 0.0168 m/s

= 1451.5 m3/m2/d

However, due to turbulence and short circuiting due to several factors as eddy, wind and density currents, the actual value to be adopted has to be reduced taking into account the performance of the basin and the desired efficiency of the particles removal. To determine the actual overflow rate, the following formula may be used.

$$\eta = 1 - \left(1 + n \frac{V_s}{\frac{Q}{A}}\right)^{-\frac{1}{n}}$$

where,

η = efficiency of removal of desired particles

n = measure of settling basin performance

= 1/8 for very good performance

Assuming η = 0.75, n = 1/8

$$\frac{Q}{A} = \frac{V_s n}{(1-\eta)^{-n} - 1}$$
$$= \frac{1451.5 \times (1/8)}{(1-0.75)^{-(1/8)} - 1} = 959\ m^3/m^2/d$$

2.3 DETERMINATION OF THE DIMENSIONS OF GRIT CHAMBER

$$\textit{Plan area of grit chamber} = \frac{Q}{(Q/A)}$$
$$= \frac{7.5 \times 10^3}{959} = 7.82\ m^2$$

Provide 4 channels of 0.5 m wide and 16 m long.

The critical displacement velocity to initiate re-suspension of grit is given by

$$V_c = \left[\frac{8K}{f}(S_s - 1)gd\right]^{0.5}$$

for K = 0.04, f = 0.03, S_s = 2.65, d = 0.15×10^{-3} m
V_c = 0.161 m/s

The horizontal velocity of flow V_h should be kept less than critical displacement velocity, V_c. Assuming a depth of 0.3 m,

$$V_h = \frac{7.5 \times 1000}{0.3 \times 4 \times 0.5 \times 24 \times 3600} = 0.15\ m/s < 0.161\ m/s \quad OK$$

The hydraulic residence time at peak flow is

$$HRT = \frac{Volume}{Peak\ discharge} = \frac{4 \times 0.5 \times 16 \times 0.3}{0.087} = 110\, seconds$$

Total depth of grit chamber = Water depth + free board + grit storage space

=0.3 + 0.25 + 0.25 = 0.8 m

Provide 4 channels of grit chamber, each 16m × 0.5m ×0.8m

2.4 DESIGN OF PROPORTIONAL FLOW WEIR

There will be four proportional flow weirs, each installed at the control section of each of the four grit chambers.

Peak flow for each weir = (0.087 / 4) = 0.022 m^3/s
Flow through a proportional flow weir is givers by

$$Q = Cb\sqrt{2ag}\left[h - \left(\frac{a}{3}\right)\right]$$

For symmetrical sharp-edged weir, C = 0.61

Assuming, a = 35 mm (usually between 25-50 mm)

h = 1.1m at peak flow

$$0.022 = 0.61 \times b\sqrt{2 \times 0.035 \times 9.81} \times \left[1.1 - \left(\frac{0.035}{3}\right)\right]$$

b = 0.04 say 4 cm

To determine the coordinates (x, y) of the curve forming the edge of the weir, assume suitable four values of y and compute corresponding values of x using equation 5.14, The coordinates for

$$x = \frac{b}{2}\left[1 - \frac{2}{\pi}\tan^{-1}\sqrt{\frac{y}{a} - 1}\right]$$

proportional flow weir are listed below:

Sl. No.	y, m	x, m
1.	a = 0.035	0.400
2.	10a = 0.35	0.082
3.	20a = 0.70	0.057
4.	30a = 1.05	0.047
5.	40a = 1.40	0.040

However, with a base width (b) of 4 cm, it is not possible to have such a weir. Hence, continue the channel section itself downstream by elevating the floor level above the grit storage space of 0.25 m for a length of at least 5 times the clear width and designing the floor slope such that a minimum velocity of 0.3 m/s is obtained at least in peak flow conditions.

APPENDIX A 5.8
DESIGN EXAMPLE FOR DETRITOR

Design a detritors for a peak flow of 150 MLD and Average flow of 60 MLD

Peak design flow	= 150 MLD
Average flow	= 60 MLD
Number of units	= 2 (designer to choose)
Peak design flow for each unit	= 75 MLD
Grit particle size to be removed	= 0.15 mm
Grit particle specific gravity	= 2.65
Settling velocity as per Table 5.6	= 0.018 m/sec
Surface overflow rate as per Table 5.6	= 1555 cum / sqm / day
Plan area of each detritors at peak flow	= 150 X 1000 / 1555 / 2 = 48 sqm
One side of a square detritor	= 7 m

Critical displacement velocity to initiate resuspension of grit is calculated as in Appendix A 5.7

Critical displacement velocity	= 0.161 m / sec
Depth of flow	= assumed as 0.85 m
Horizontal velocity	= 75 x 1000 / 24 / 3600 / 7 / 0.85 = 0.145 m / sec

Less than critical displacement velocity of 0.161and hence OK
Grit storage space

Rotary speed of scraper	= say 0.25 rpm
Time between scraping	= 1 / 0.25 = 4 minutes

Grit content for domestic sewage varies from 0.05m3 to 0.15m3 per ML

Assume	= 0.1m3 per ML
Quantity of grit between scrapings	= 60 x 0.1 x 4 / 1440 = 0.017 cum
Depth of detritor needed to hold this grit	= 0.017 / 48 / 2 = negligible

Normally this is a concurrent activity and separate depth is not needed and may affect velocities
Provide, 2 numbers of each 7 m x 7 m x 0.85m LD with freeboard as per equipment vendor

Appendix A 5-9
Calculating Size of Approach Channel for Parshall Plume

1	A	B	C	D	E	F	G
2							
3		choice 1		choice 2		choice 3	
4	Lower value of flow in MLD	Enter	45	Enter	45	Enter	45
5	Higher value of flow in MLD	Enter	170	Enter	170	Enter	170
6	Lower value of flow in lps	C4*1000000/24/3600	521	E4*1000000/24/3600	521	G4*1000000/24/3600	521
7	Higher value of flow in lps	C5*1000000/24/3600	1968	E5*1000000/24/3600	1968	G5*1000000/24/3600	1968
8	Throat width (W) m, from Table	Enter	0.3	Enter	0.6	Enter	0.9
9	Channel at (D) m, from Table	Enter	0.83	Enter	1.12	Enter	1.55
10	Liquid depth at low flow, m	POWER((C6/2264/C8),(1/1.5))	0.84	POWER((E6/2264/E8),(1/1.5))	0.53	POWER((G6/2264/G8),(1/1.5))	0.40
11	Liquid depth at high flow, m	POWER((C7/2264/C8),(1/1.5))	2.03	POWER((E7/2264/E8),(1/1.5))	1.28	POWER((G7/2264/G8),(1/1.5))	0.98
12	velocity at mouth, low flow, m/s	C4*1000/24/3600/C9/C10	0.75	E4*1000/24/3600/E9/C10	0.88	G4*1000/24/3600/G9/G10	0.83
13	velocity at mouth, high flow, m/s	C5*1000/24/3600/C9/C11	1.17	E5*1000/24/3600/E9/C11	1.37	G5*1000/24/3600/G9/G11	1.30
14	Approach channel width, m	C9	0.83	E9	1.12	G9	1.55
15	Approach channel, liquid depth, m	C11	2.03	E11	1.28	G11	0.98
16	Approach channel width, m	C14	0.83	E14	1.12	G14	1.55
17	Approach channel liquid depth, m	C11	2.03	E11	1.28	G11	0.98
18	Width of throat, m	C8	0.3	E8	0.60	G8	0.9

It may be seen that the velocities at low flow and high flow are met adequately in all three chosen channel widths. However, the ease of O&M also has to be considered. Choice 1 gives a very narrow width and very deep channel. This will be difficult to maintain. Between choice 2 and choice 3, both can be used. However choice 2 will give almost a square section and lesser width for easy maintenance.

APPENDIX A 5.10
DETENTION TIMES OF CLARIFIERS IN STPs EVALUATED BY NEERI

Name of STP	Q in	Primary				Secondary				
		Numbers	Diameter	SWD	HRT	Numbers	Diameter	SWD	HRT	Total HRT
	MLD		m	m	hrs		m	m	hrs	hrs
K&C Valley	163	3	39.6	3.04	1.65	3	46.5	3.6	2.70	
						1	39	3.6	0.63	3.33
Atladara	27	1	45.7	2.5	3.64	1	35	2.5	2.14	2.14
Howrah	45	2	38.1	3	3.65	2	38.1	3	3.65	3.65
Mahati	135	4	22.86	3.14	0.92	2	40	3.2	1.43	
						2	45	3.5	1.98	3.41
Jaspur	20	1	39.6	2.4	3.55	1	33.5	2.4	2.54	2.54
Kodungaiyur II	90	2	40	3	2.01	2	41.6	3	2.17	2.17
Koyambedu	34	2	36.6	2.74	4.07	2	33.5	2.44	3.03	3.03
Nesapakkam	23	2	21.2	2.4	1.77	2	24.4	3.1	3.02	3.02
Kalyani	10.8	1	23.6	4.2	4.08	1	28.3	1.6	2.24	2.24
Rithala	180	4	40	3.12	2.09	4	42.7	4.2	3.21	3.21
Kishopur	180	4	40	3	2.01	4	48	3.6	3.47	3.47
BHU	8	2	14.6	3	3.01	2	16	3.5	4.22	4.22
Maximum				4.20						4.22
Minimum				2.40						2.14

Source-Performance Evaluation of Sewage Treatment Plants in India-NEERI-February-1994

Appendix A 5-11
Illustrative Sizing of Clarifiers in Activated Sludge

1	A	B	C
2	Primary Clarifiers followed by secondary treatment		
3			
4	Average flow, MLD	Enter	23
5	Peak flow. MLD	Enter	60
6	Overflow rate for average flow from Table 5.8. m3/m2/day	Enter	35
7	Overflow rate for peak flow from Table 5.8, m3/m2/day	Enter	80
8	Surface area for average flow, sqm	C4*1000/C6	657
9	Surface area for peak flow, sqm	C5*1000/C7	750
10	Higher of the two areas, sqm	MAX(C8,C9)	750
11	Required diameter, m	SQRT(4*C10/3.14)	30.91
12	Resulting weir length, m	3.14*C10	97.06
13	Resulting weir loading rate, m3/m /day	C3*1000/C11	236.97
14	Weir loading rate in average flow from Table 5.8, m3/m / day	Enter	125.00
15	Usage of double sided weir is needed or not	Enter	Yes
16	Choose an annular space from inner sidewall of clarifier, m	Enter	0.20
17	Choose a launder width, m	Enter	0.60
18	Diameter of outer weir, m	C10-C15-C15	30.51
19	Diameter of inner weir, m	C17-C16-C16	29.31
20	Diameter of inner weir, m	3.14*(C17+C18)	187.83
21	Resulting weir loading rate, m3/m /day	C3*1000/C19	122.45
22	Weir loading is safe or not	Enter	Yes
23	Side water depth shall be to suit Table 5.8	Enter	Yes
24	In actual practice, the diameter and the size of launder will be not less than the above values		

25	Depending on the area and diameter, multiple number of these clarifiers may be designed		
26	Secondary Clarifiers for activated sludge		
27	Average flow, MLD	Enter	23
28	Peak flow. MLD	Enter	60
29	Return sludge flow, MLD	Enter	11.5
30	MLSS, mg/L	Enter	3500
31	Overflow rate for average flow from Table 5.8. m3/m2/day	Enter	25
32	Overflow rate for peak flow from Table 5.8, m3/m2/day	Enter	40
33	Surface area for peak flow, sqm	C5*1000/C7	750
34	Surface area for average flow for overflow rate, sqm	C27*1000/C31	920
35	Solids loading for average flow from Table 5.8. kg/m2/day	Enter	100
36	Solids loading for peak flow from Table 5.8, kg/m2/day	Enter	210
37	Surface area for average flow for solids loading, sqm	(C27+C29)*C30 /C35	1207.5
38	Surface area for peak flow for solids loading, sqm	(C28+C29)*C30/ C36	1192
39	Higher of the four areas, sqm	MAX(C33,C34, C37,C38)	1500
40	Required diameter, m	SQRT(4*C39/3.14)	43.71
41	Resulting weir length, m	3.14*C40	137.26
42	Resulting weir loading rate at average flow, m3/m /day	C27)*1000/C41	168
43	Weir loading rate in average flow from Table 5.8, m3/m / day	Enter	185
44	Usage of double sided weir is needed or not	Enter	No
45	Choose an annular space from inner sidewall of clarifier, m	Enter	0.2
46	Side water depth shall be to suit Table 5.8		
47	In actual practice, the diameter and side water depths will be not less than the above values		
48	Depending on the area and diameter, multiple number of these clarifiers may be designed		

APPENDIX A 5.12
ILLUSTRATIVE DESIGN OF CONVENTIONAL ASP AERATION

The MS Excel version is available in the CD version of the manual and can be easily used. In case of any difficulty, the reader may proceed as follows.

Leave 3 blank rows at the top and start from cell A4.
Copy column A as below
Paste it in column A of your computer
Then copy column C as below
Paste it in column C of your computer by prefixing =
Then wherever it says “Enter” in column C below, you can enter your choice in column B
Then copy column B as entered and paste in column D except in cell D6,
Then enter the winter temperature in cell D6

No	A	B	C	D
1				
2				
3				
4	Plant design flow, MLD	50	Enter	50
5	Elevation of site above MSL, m	30	Enter	30
6	Operating temperature in deg C	30	Enter	20
7	Primary clarifier effluent BOD, mg/l	230	Enter	230
8	Thickener overflow return as fraction of plant flow	0.15	Enter	0.15
9	Thickener overflow return, MLD	7.5	B4*B8	7.5
10	Thickener overflow return BOD, mg/l	500	Enter	500
11	Centrate from sludge dewatering as fraction of plant flow	0.006	Enter	0.006
12	Centrate from sludge dewatering return, MLD	0.3	B4*B11	0.3
13	Centrate from sludge dewatering return BOD, mg/l	380	Enter	380
14	Influent BOD to aeration tank, mg/l	266	((B4*B7)+(B9*B10)+ (B12*B13))/ (B4+B9+B12)	380
15	Effluent BOD	20	Enter	20

No	A	B	C	D
16	Weighted BOD to be removed in the aeration tank, mg/l	246	B14-B15	246
17	MLSS	3000	Enter	3000
18	F : M	0.35	Enter	0.35
19	F	14208	B16*(B4+B9+B12)	14208
20		40594	B19/B18	40594
21	Aeration tank volume calculated from F/M, cum	13531	(B20/B17)*1000	13531
22	Mean Cell Residence Time, Theta C, days	3	From Fig 5.31 in Chapter 5	5
23	Constant Y	0.500	Fixed value	0.500
24	constant Kd	0.060	Fixed value	0.060
25	Aeration tank volume calculated from Theta, cum	5208	B23*B4*1000*B16 *B22/(1+B24*B22)/ B17	0.006
26	HRT for average flow as per CPHEEO, hrs	5	4 to 6 in Table 5.8 in Chapter 5	5
27	Influent BOD to aeration tank, mg/l	266	((B4*B7)+(B9*B10)+ (B12*B13))/ (B4+B9+B12)	380
28	Aeration tank volume calculated from HRT, cum	10417	B4*1000*B26/24	10417
29	liquid depth, m (restrict to max of 6.5 m if air cooled)	5.50	Enter	5.50
30	BOD removed in aeration tank, kg/day	14208	(B4+B9+B12)*B16	14208
31	Kg oxygen / Kg of BOD removed	0.9	0.8 to 1.0 in Table 5.8 in Chapter 5	0.9
32	Kg of Oxygen needed per day	12787	B30*B31	12787
33	Residual D. O. in aeration	2.0	Fixed value	2.0
34				
35	**If surface aerators are used**		5412.4	

No	A	B	C	D
36	Alpha value	0.83	0.8 to 0.85 as in Equation 5.31 in Chapter 5	0.83
37	Beta value	0.95	As in clause 5.8.1.7.5.3	0.95
38	D O at operating temperature	7.43	14.42+(0.003*B6* B6)-(0.323*B6)	9.16
39	D O at operating elevation	7.41	(1-(B5/152)*0.017) *B38	9.13
40	Oxygen tension, mg/l	5.03	(B39*B37)-B33	6.67
41	Oxygen gradient, mg/l	0.55	B40/9.17	0.73
42	Temperature difference	10.0	B6-20	0.0
43	Temperature Co-efficient	1.03	Fixed Value	1.03
44	Temperature correction factor	1.3	Power(B43,B42)	1.0
45	Conversion factor to standard conditions	0.61	B41*B36*B44	0.60
46	Oxygen needed under standard conditions, kg / day	20879	B32/B45	21172
47	Provide factor of safety for intangibles	1.1	--	1.1
48	Oxygen needed after factor of safety , kg/ day	22967	B45*B46	23289
49	Oxygen transfer capacity of aerator Kg/Kwhr in aeration tank, kg/day	1.8	1.2 to 2.4 in clause 5.8.1.7.5.3	1.8
50	Kw of aerator needed	532	B48/24/B49	532
51				
52	If diffused aeration is used			
53	Factor for temperature power 3	0.027	POWER(B6,3)/ (POWER(10,6))	0.008
54	Factor for temperature power 2	0.001	(POWER(B6,2)/ (POWER(10,5))/7)	0.001

No	A	B	C	D
55	Factor for temperature power 1	0.090	0.003*B6	0.060
56	Density of air at operating temperature	1.221	1.285+B53-B54-B55	1.232
57	Content of oxygen in air	0.23	Fixed value	0.23
58	Kg of oxygen needed for residual D O per day	100	B4*B33	100
59	Total kg of oxygen needed per day	12887	B32+B58	12887
60	Air needed in cum / day	45505	B59/B56/B57	45072
61	Transfer efficiency of diffuser system per m depth (*)	0.05	Fixed	0.05
62	Transfer efficiency at design depth	0.28	B29*B61	0.28
63	diffuser fouling factor per year	0.04	Enter	0.04
64	Diffuser life cycle, years	3.00	--	3.00
65	Diffuser fouling factor for its life cycle	1.12	POWER((1+B63), B64)	1.12
66	Provide factor of safety for intangibles	1.10	B47	1.10
67	Air needed for oxygenation in cum / day	204747	B60*B65*B66/B62	163899
68	Air needed for oxygenation in cum / hour	8531	B67/24	6829
69	Air mixing criteria cum /minute / 1000 cum of tank	16	As in clause 5.8.1.7.5.6	16
70	Air needed for mixing as per manual cum / hr	12990	B28*B69*60/1000	12990
71	Air needed for mixing as per manual in cum / hr	2.7	1.8 to 2.7 as per US EPA, 625/8-85/0100, p 38	2.7
72	Surface area of aeration tank, sqm	2460	B28/B29	2460
73	Air needed for mixing as per US EPA guidelines in cum / hr	6643	B71*B72	6643
74	Maximum of air needed for mixing in cum / hr	12990	MAX(B70,B73)	12990
75	Air needed as higher of oxygenation and mixing cum / hr	12990	MAX(B68,B74)	12990

No	A	B	C	D
76	Air needed as under standard conditions, cum / hr	21210	B75/B45	21508
77	Friction and other losses as fraction of depth	0.2	Enter by Designer	0.2
78	Liquid depth as water column for air pressure	6.6	B29*(1+B77)	6.6
79	Kw of needed compressor at 1400 rpm	486.6	0.746*((0.03*B76)+16)	493.3
80	For DPR purpose, equation for Compressor Kw at 1400 rpm for 7 m water column can be taken as BHP = 0.03*(cum / hr)+16			
81	For DPR purpose, equation for Compressor Kw at 1400 rpm for 6 m water column can be taken as BHP = 0.025*(cum / hr)+13			
82	For DPR purpose, equation for Compressor Kw at 1400 rpm for 5 m water column can be taken as BHP = 0.02*(cum / hr)+14			
83	**Sludge Flows**			
84	Y0bs=Y/(1+Kd*ThetaC)	0.42	(B23)/(1+B24*B22)	0.38
85	Excess Sludge mass wasted Kg/day	5208	B84*B16*B4	4727
86	Kgd of excess sludge / Kg of BOD removed	20879	B32/B45	21172
87	Kg of excess sludge from thumb rule per day	4916	B4*B16*B86	4916
88	Excess sludge as higher of the two values, Kg/ day	5208	MAX(B85,B87, D85, D87)	5208
89	Concentration factor for MLSS in return / excess sludge	3.3	Enter by Designer	3
90	Return / excess sludge MLSS concentration, mg/l	9900	B17*B89	9900
91	Cells in aeration, kg	40594	B17*B28/1000	40594
92	Cells wasting from system kg / day	5208	B88	5208
93	Volume of excess sludge, cum / day	526	B92*1000000/B90/1000	526
94	Resulting Theta C	7.8	B91/B92	7.8

No	A	B	C	D
95	Least Theta C in design	0.3	MIN(B22,D22)	0.5
96	Volume of excess sludge for least Theta C	1367	B93*B94/B95	820
97	Excess sludge pump set duty as cum / day	1367	MAX(B93,B96,D93,D96)	1367
98	Recirculation ratio	0.8	0.25 to 0.8 as per Table 5.8	0.8
99	Return sludge pump set duty as MLD	40	B4*B98	40
100	Motors of both the return sludge and excess sludge pump sets will be provided with VFD to downsize actual pumpage as needed			

Note.

For incorporating anoxic tank before aeration tank, the design criteria of the Bangalore K&C Valley STP as given in Table 5.32 may be followed.

APPENDIX A 5.13
DESIGN EXAMPLE OF FACULTATIVE AERATED LAGOON
(Retained as in 2nd edition)

1 PROBLEM

Design a facultative aerated lagoon to serve 40,000 people. Sewage flow @ 180 lpcd = 7200 cum/ day, Raw BOD_5 = 50 gcd or 277 mg/l and final BOD_5 is not to exceed 30 mg/l in winter. Average ambient air temperature in January is 18ºC and in summer 37ºC.

2 SOLUTION

2.1 LAGOON SIZE

Assume detention time	=	5 days
Lagoon volume	=	7,200×5 = 36,000 cum.
Let Lagoon dimensions be		70 m×130 m×4 m deep

2.2 LAGOON WINTER TEMPERATURE

Use Eq. (5.34) to determine TL. Assume Ti = 23°C
Hence, Hence,

$$\frac{5\ days}{4\ m} = \frac{(23 - T_L)}{0.49(T_L - 18)}$$

TL = 21ºC

2.3 ESTIMATION OF K

Assume K at 20ºC	=	0.7 per day
Hence, K at 21°C	=	0.7×1.035 = 0.724/day,

2.4 D/UL ESTIMATION

Keep lagoon geometry such that flow conditions are plug-flow type (i.e. D/UL = 0.2 approx.). This will be possible if a long and narrow lagoon (23 m×390 m) is provided (see Table 5.12) or baffles are provided within the rectangular lagoon of 70 m×130 m to give a winding flow with the same effect. (See Figure 5.42).

2.5 BOD_5 REMOVAL EFFICIENCY (IN WINTER)

K×Θ = 0.724×5	=	3.62
See Figure (5.41) at K×Θ = 3.62 and D/UL	=	0.2
Soluble BOD removal efficiency	=	92%
Namely, soluble BOD in effluent	=	22 mg/l
SS likely to flow out in effluent	=	35 mg/l (say)

BOD of VSS = 0.77×(0.6×35) = 16 mg/l
Hence, BOD of effluent = 22 + 16 = 38 mg/l
Overall efficiency in winter = 86%

In other months of the year, the efficiency will be higher and effluent BOD will be less than the above value.

2.6 POWER REQUIREMENT

When efficiency = 86% and all BOD is removed aerobically,
O_2 required/day = 0.86 (1.4×2000 kg/d)
= 2,408 kg/d = 100kg/hr

$$Power\ needed = \frac{100\ kg/hr}{0.8 \times 2\ kgO_2/kWh}$$

$$= 62.5 \text{ kW (i.e. about 80 HP)}$$

$$Power\ level\ in\ Lagoon = \frac{62.5kW \times 1{,}000}{36{,}000}$$

= 1.7 W/cum (acceptable)

2.7 LAND REQUIREMENT

Net lagoon area = 9,000 sqm
Area including embankments and slopes = 13,500 sqm (approximately)
Area/person = 0.337 sqm/person

NOTE)

If the lagoon was kept as a square shaped unit or a rectangular unit with say W:L = 1:2, the D/UL value would have been between 3.0 and 4.0 (namely, approaching completely - mixed conditions) and soluble effluent BOD would have increased to 49 mg/l, thus giving a total final effluent of about 65 mg/l instead of 38 mg/l seen above. Thus, lagoon geometry plays an important part in determining efficiency.

APPENDIX A 5.14
DESIGN EXAMPLE OF FACULTATIVE STABILIZATION POND
(Retained as in 2nd edition)

1 PROBLEM

Design a facultative stabilization pond to treat 5,000 m3/d municipal wastewater, BOD5 230 mg/l from a town (population 25,000 persons) located in Central India, latitude 22ºN, elevation 100 m above sea level. The average temperature in January is 18°C. The effluent from the pond is to be used for irrigation.

2 SOLUTION

2.1 POND SIZE

Permissible organic load according to temperature correlation	= 20×18 – 120
	= 240 kg BOD/ha/d
Permissible organic load according to latitude and elevation	= 235/(1 + 0.003×1)
	= 180 kg BOD/ha/d
Adopt a conservative loading rate of 200 kg BOD/ha/d	
BOD load from the town	= 5,000×0.23 = 1,150 kg/d
Therefore pond area	= 1,150/200 = 5.75 ha
Adopt an average depth of 1.5 m	
Therefore pond detention time	= 5.75×104×1.5/5,000
	= 17.25 d

Provide three ponds of equal volume and surface area; two primary ponds in parallel and one secondary pond in series receiving the effluent of the two primary ponds. Use of multiple ponds improves performance from viewpoints of stability, efficiency of treatment and maintenance. However, it requires greater land area for the same pond surface area.

2.2 CHECK FOR DETENTION TIME

For 90% BOD reduction, the BOD reaction rate constant = 0.2/d for plug flow condition. The total overall detention time, ϴ, is given by:
0.1 = exp - 0.2 (2×ϴ/3 + ϴ/3), or ϴ = 11.5 d

For a conservative estimate, for completely mixed condition in all three ponds, the total overall detention time is given by:
0.1 = 1/(1+0.2×2ϴ/3) (1 + 0.2×ϴ/3), or ϴ = 22.5 d

In actual conditions, the hydraulic regime in the ponds is going to be between the two ideal conditions of plug flow and completely mixed flow. The detention time of 17.25 d is therefore acceptable.

2.3 CHECK FOR MICROBIAL QUALITY FOR IRRIGATION

WHO guidelines recommend sewage retention in stabilization ponds for 8-10 days for irrigation of cereal, fodder and industrial crops and trees. This assures removal of intestinal nematodes from sewage. The design meets this requirement.

For irrigation of crops likely to be eaten uncooked, the guidelines recommend a faecal coliform limit of 1,000 organisms/100 ml. For microbial reduction rate constant of 2.0/d at 20°C or 1.4 at 18°C, and influent faecal coliform concentration = 107/100 ml, the effluent concentration N is
given by

N = 107/ (1+1.4×2×17.25/3) (1+1.4×17.25/3)
or,
N = 64,600/100 ml

Therefore the design does not meet the criteria of irrigation water quality for crops likely to be eaten uncooked. If two maturation ponds, each of 17.25/3 d detention time are provided in series after the secondary pond, the effluent concentration is expected to be:

N = 107/ [(1 + 1 4×2×(17.25/3)] (1+1.4×17.25/3)3
= 788/100 ml

The above calculations are based on assumption of complete mixing. In actual condition the performance is likely to be better.

2.4 SLUDGE ACCUMULATION

Most of the sludge will accumulate in primary ponds. Assuming 0.75 m deep allowable sludge deposition, capacity available = 0.75 × (2/3) × 5.75 × 104 = 28,750 m3. For 0.07 m3/person/year sludge accumulation rate,

Desludging frequency = 28,750/(0.07 × 25,000) = 16 years.
Because of non-uniform deposition of sludge, a desludging frequency of once in 10 years is recommended.

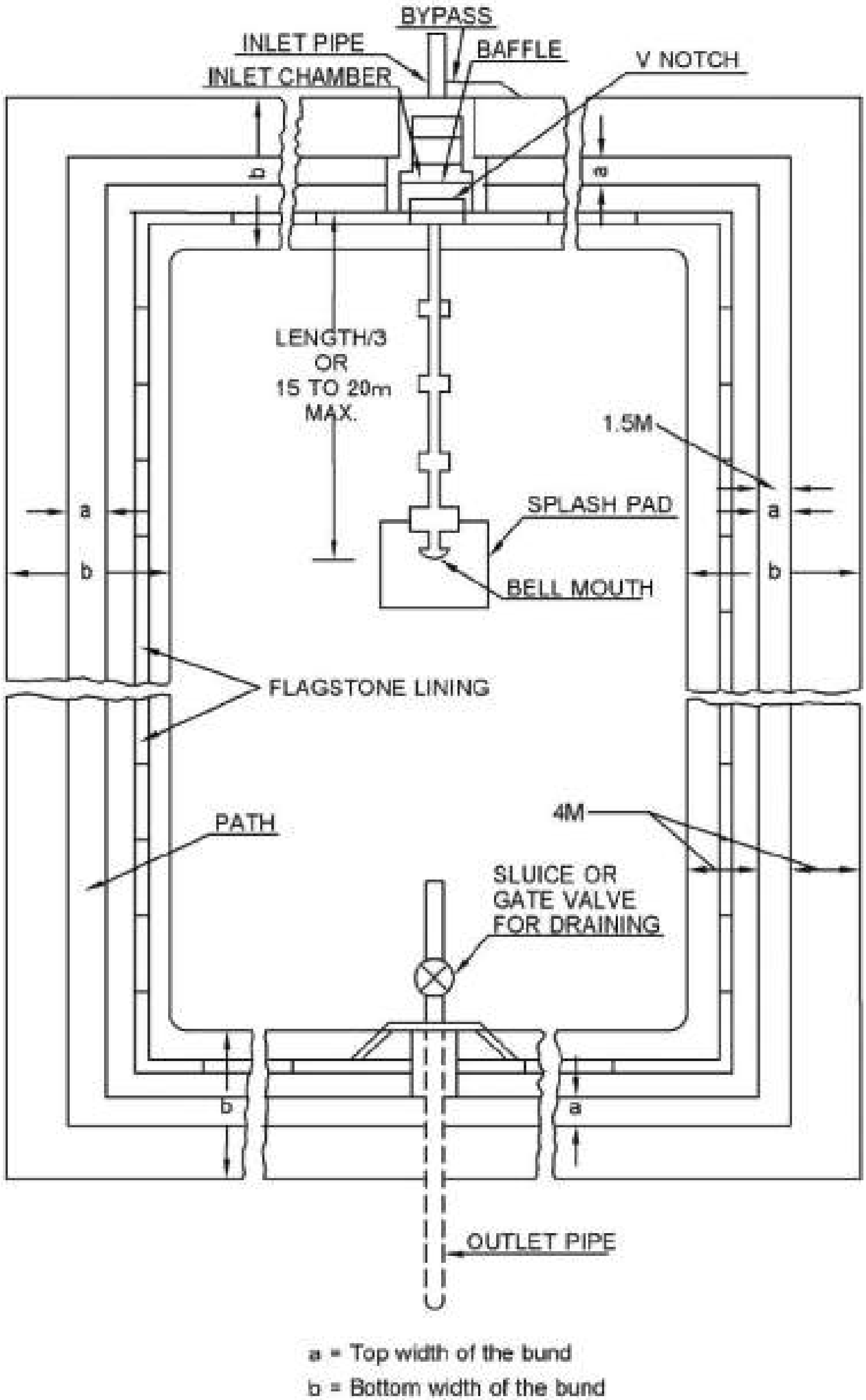


Figure A5.14-1 Typical plan of a waste stabilization pond

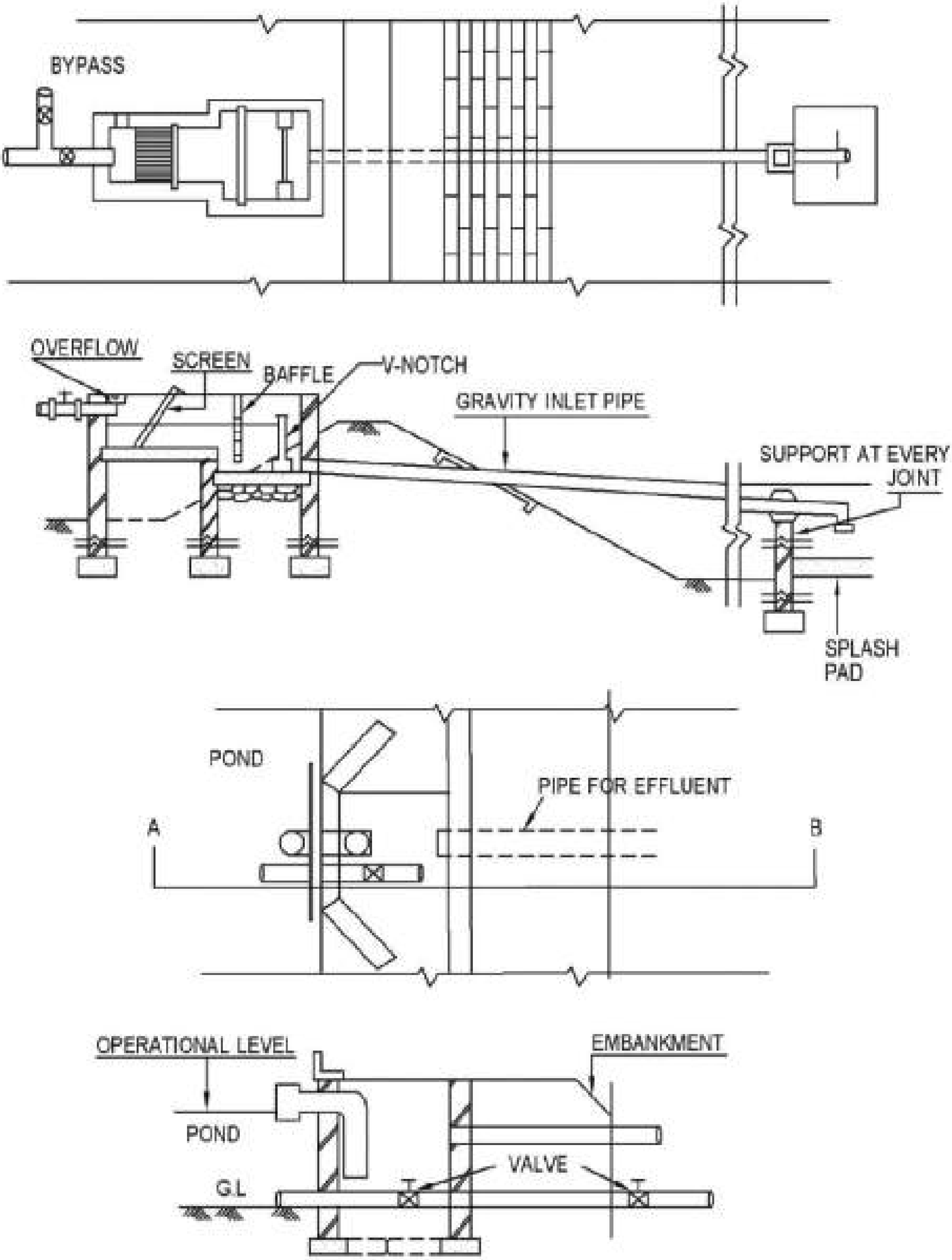


Figure A5.14-2 Typical details of inlet and outlet chamber for facultative waste stabilization pond

APPENDIX A 5.15
DESIGN EXAMPLE FOR UPFLOW ANAEROBIC SLUDGE BLANKET REACTOR
(Retained as in 2nd edition)

1 PROBLEM

Design an upflow sludge blanket reactor for an average flow of 5 MLD of wastewater with the following data:

1. COD of wastewater	=	400 mg/l
2. Design hydraulic residence time	=	6 hrs
3. Design COD loading	=	1 - 2 kg COD/m^3d
4. Velocity of rise of wastewater in the reactor through sludge bed	=	0.75 m/hr
5. Velocity of wastewater in settling chamber	=	< 1.5 m/hr
6. Flow area covered by each inlet	=	1 - 2 m^2

2 SOLUTION

2.1 DETERMINE THE DIMENSIONS OF UASBR

Volume of UASBR	= 5,000 × (6/24)	=	1,250 m^3
Actual volumetric organic loading	= [(5×400)/1,250]	kg COD/m^3d	
	= 1.6 [OK as it is between 1-2 kg COD/m^3d]		
Height of wastewater in reactor	= Rise velocity×HRT		
	= 0.75×6	=	4.5 m
Area of Reactor	= [1250/4.5]	=	277.8 m^2

Provide two reactors of 11.8 m×11.8 m×5.25 m (height)

2.2 NO. OF INLETS

Assume that each inlet can serve 2.0 m^2 of flow area

Number of inlets in each reactor = [138.9/2] = 70

2.3 AREA OF SETTLING CHAMBER

Assuming a velocity of 1.2 m/hr in the settling zone

Area of settling chamber in each reactor = [5,000/(2×24×1.2)] = 86.8 m^2

APPENDIX A 5.16
DESIGN EXAMPLE FOR ANAEROBIC FILTER
(Retained as in 2nd edition)

1 PROBLEM

Design anaerobic filters to treat an average flow of 5 MLD of wastewater with the following assumptions:

1.	COD of the wastewater	=	400 mg/l
2.	Design COD Loading	=	1.0 kg COD/m^3d
3.	Depth of media	=	1.2 m

2 SOLUTION

2.1 DIMENSIONS OF ANAEROBIC FILTER

Total COD load	= 5×400	=	2,000 kg COD/d
Volume of anaerobic filters for media	= [2,000/1.0]	=	2,000 m^3
Plan Area of filters	= [2,000/1.2]	=	1,666.7 m^2

Provide two filters of diameter 32.6 m and height 1.5 m including free board and bottom zone for dispersion of wastewater and supporting media.

2.2 HRT FOR FILTERS

HRT = [2,000/5,000] d
= 9.6 hrs

APPENDIX A 6.1
ILLUSTRATIVE COMPUTATION OF
SLUDGE WEIGHTS AND VOLUMES FROM ASP

No	A	B	C	D
1	Appendix 6-1			
2	Illustrative Computation of Sludge weights and volumes from ASP			
3	Raw sewage flow in MLD	Enter		10
4	Raw sewage BOD in mg/l	Enter		250
5	Raw sewage SS in mg/l	Enter		400
6	Raw sewage VSS in percent	Enter		60
7	Raw sewage ISS in percent		100-D6	40
8	Secondary clarifier effluent BOD in mg/l	Enter		10
9	Consider Primary Clarifier alone			
10	BOD removal in percent	Enter		30
11	Effluent BOD in mg/l		D4*(100-D10)/100	175
12	ISS removal in percent	Enter		60
13	VSS removal in percent	Enter		60
14	TSS removal in percent	Enter	100*((D6*D12/100)+(D7*D13/100))/ (D6+D7)	60
15	VSS removed in kg / day		D3*D5*D6*D13/100/100	1440
16	ISS removed in kg / day		D3*D5*D7*D12/100/100	960
17	TSS removed in kg / day		SUM(D15:D16)	2400
18	% solids in primary sludge	Enter		4
19	Specific gravity of primary sludge	Enter		1
20	Sludge removed in Kld		D17*100/D18/D19/1000	60
21	BOD removed in kg / day		D3*D4*D10/100	750
22	Kg VSS / kg BOD removed		D15/D21	1.9

No	A	B	C	D
23	Kg ISS / kg BOD removed		D16/D21	1.3
24	Kg TSS / kg BOD removed		D17/D21	3.2
25	ISS carry over in kg / day		(D3*D5*D7/100)-D16	640
26	Consider conventional secondary stage alone			
27	Clarifier effluent BOD in mg/l	Enter		10
28	Secondary clarifier VSS in percent	Enter		85
29	Secondary clarifier ISS in percent		100-D28	15
30	Aeration kd in per day	Enter		0.06
31	Aeration Y	Enter		0.5
32	Aeration SRT in days			5
33	Kg VSS / kg BOD removed, Y observed = Y/(1+ (kd)*(SRT))	Enter	D31/(1+(D30)*(D32))	0.38
34	BOD removed in kg / day		D3*(D11-D27)	1650
35	Kg VSS removed per day		D33*D34	635
36	SS removed in kg / day		D35*100/D28	747
37	Inert sludge carry over in kg/day		(D3*D5*D7/100)	1600
38	Kg TSS carried over per day		SUM(D36:D37)	2347
39	% solids in sludge		Enter	0.75
40	Specific gravity		Enter	1.00
41	Sludge removed in Kld		D38*100/D39/D40/1000	313
42	Kg VSS / kg BOD removed		D35/D34	0.4
43	Kg TSS / kg BOD removed		D38/D34	1.42
44	Consider extended aeration alone			
45	Clarifier effluent BOD in mg/l	Enter		10
45	Aeration kd in per day	Enter		0.06

No	A	B	C	D
46	Aeration kd in per day	Enter		0.06
47	Aeration Y	Enter		0.5
48	Aeration SRT in days	Enter		15
49	Kg VSS / kg BOD removed, Y observed = Y/(1+ (kd)*(SRT))	Enter	D47/(1+(D46)*(D48))	0.26
50	BOD removed in kg / day		D3*(D4-D45)	2400
51	Kg VSS removed per day		D49*D50	632
52	Inert sludge carry over in kg/day		D3*D5*(D7)/100	1600
53	Kg TSS removed per day		SUM(D51:D52)	2232
54	% solids in sludge		Enter	0.75
55	Specific gravity		Enter	1.00
56	Sludge removed in Kld		D53*100/D54/D55/1000	298
57	Kg VSS / kg BOD removed		D51/D50	0.26
58	Kg TSS / kg BOD removed			0.93
59				
60	Rule of Thumb for Design Purposes for parameters as above.			
61	For other parameters please rework in the M S excel			
62			Kg Sludge / Kg BOD removed	
63	Process	as VSS	as TSS	
64	Primary alone	1.92	3.20	
65	Primary & secondary as conventional ASP	0.86	1.31	
66	Extended aeration ASP	0.26	0.93	

APPENDIX A 6.2
ILLUSTRATIVE CALCULATION OF SLUDGE WEIGHTS

Calculation example of Figure 6.1.

1) Set influent solid as 100 and Design sludge generation, D as 90.
2) Assume solids recovery rate or reduction rate of each treatment stage as under.

Table A6.2-1

Solids recovery rate, etc.	Figure 6.1, a	Figure 6.1, b
Solid recovery rate in Thickening, γ^1	90 %	90 %
Solid reduction rate in Digestion, γ^2	-	40 %
Solid recovery rate in Dewatering, γ^3	95 %	90 %
Coagulant dosing rate in Dewatering, γ^4	0.8 %	1.0 %
Solid reduction rate in Incineration, γ^5	70 %	-
Solid recovery rate in Incineration, γ^6	80 %	-

3) In case of Figure 6.1, a

Solid of return flow from sludge treatment facilities, R, is calculated as under assuming all solids are returned to the sludge treatment facilities.

$$R = (D + R)\ \{(1 - \gamma^1) + \gamma^1(1 + \gamma^4)(1 - \gamma^3) + \gamma^1(1 + \gamma^4)\ \gamma^3\ (1 - \gamma^5)(1 - \gamma^6)$$
$$= (90 + R)\ \{(1 – 0.9) + 0.9(1 + 0.008)(1 – 0.95) + 0.9(1 + 0.008)\ 0.95\ (1 – 0.7)(1 – 0.8)$$
$$= (90 + R) \times 0.197$$
$$R = 22.1$$

Calculation of Solid in each treatment stage

Thickener: X1 = D + R = 90 + 22.1 = 112.1

Centrifugal dewatering machine: $X_2 = X_1 \times \gamma^1 = 112.1 \times 0.9 = 100.9$

Coagulant dosing: $C = X_2 \times \gamma_4 = 100.9 \times 0.008 = 0.8$

Incinerator: $X_3 = X_2 \times (1 + \gamma_4) \times \gamma_3 = 100.9 \times (1+0.008) \times 0.95 = 96.6$

Solid reduction in Incinerator: $G = X_3 \times \gamma_5 = 96.6 \times 0.7 = 67.6$

Incineration ash: $X_4 = (X_3 - G) \times \gamma^6 = (96.6 - 67.6) \times 0.8 = 23.2$

4) In case of Figure 6.1, b

Solid of return flow from sludge treatment facilities, R, is calculated as under assuming all solids are returned to the sludge treatment facilities as in case of Figure 6.1, a.

$$R = (D + R)\ \{(1 - \gamma^1) + \gamma^1(1 + \gamma^4)(1 - \gamma^3) + \gamma^1(1 + \gamma^4)\ \gamma^3\ (1 - \gamma^5)(1 - \gamma^6)$$
$$= (90 + R)\ \{(1 – 0.9) + 0.9(1 + 0.008)(1 – 0.95) + 0.9(1 + 0.008)\ 0.95\ (1 – 0.7)(1 – 0.8)$$

$= (90 + R) \times 0.197$

$R = 22.1$

Calculation of Solid in each treatment stage

Thickener: $X1 = D + R = 90 + 22.1 = 112.1$

Centrifugal dewatering machine: $X_2 = X_1 \times \gamma_1 = 112.1 \times 0.9 = 100.9$

Coagulant dosing: $C = X_2 \times \gamma_4 = 100.9 \times 0.008 = 0.8$

Incinerator: $X_3 = X_2 \times (1 + \gamma_4) \times \gamma_3 = 100.9 \times (1+0.008) \times 0.95 = 96.6$

Solid reduction in Incinerator: $G = X3 \times \gamma 5 = 96.6 \times 0.7 = 67.6$

Incineration ash: $X_4 = (X_3 - G) \times \gamma_6 = (96.6 - 67.6) \times 0.8 = 23.2$

APPENDIX A 6-3
FRICTION LOSSES IN SLUDGE PIPELINES UNDER GRAVITY AND PUMPED CONDITIONS

1	A	B	C
2	Calculation of Friction Losses in Sludge Pipelines		
3	for Gravity conditions and pumped conditions		
4	For Gravity Conditions		
5	Pipe material considered is UPVC		
6	value of Manning's n	Enter from Table 3.11	0.0100
7	Discharge rate m^3/day	Enter by Designer	500
8	Discharge rate in litres per sec	C7*1000/24/3600	5.79
9	Required velocity, m/sec	Enter by Designer	1.20
10	Required area in sqcm	(C8/C9/1000)*100*100	48.23
11	diameter in mm	10*SQRT(4*C10/3.14)	78.38
12	Hydraulic mean radius R in m	C11/4/1000	0.0196
13	diameter in mm power 0.67	POWER(C11,0.67)	18.5830
14	Gradient power 0.5	C9*C6*1000/3.968/C13	0.1627
15	Gradient	C14*C14	0.026
16	Length of pipeline, m	Enter by Designer	100
17	Friction loss, m	C15*C16	2.6484
18	Solids content of sludge in percent	Enter by Designer	4
19	Friction compounding factor	2.88+(0.176*C18*C18)-0.866*C18	2.2320
20	Total friction head, m	C17*C19	5.9113
21	Add safety factor at percent	Enter by Designer	10
22	Design friction head, m	C20*(1+(C21/100))	6.502
23			

24	For Pumped Conditions of raw sludge		
25	Pipe material considered is UPVC		
26	Solids in sludge as percent	Solids in sludge as percent	4
27	Value of Hazen C	Value of Hazen C	53
28	Discharge in cum / day	Discharge in cum / day	75
29	Discharge rate in litres per second	Discharge rate in liters per second	0.87
30	Required velocity, m/sec	Required velocity, m/sec	1.50
31	Required area in sqcm	Required area in sqcm	5.79
32	diameter in mm	diameter in mm	27.15
33	diameter in mm power 0.67	diameter in mm power 0.67	9.13
34	Gradient power 0.54	Gradient power 0.54	0.678
35	Gradient	Gradient	0.488
36	Length of pipeline, m	Length of pipeline, m	15
37	Friction loss, m	Friction loss, m	7.314
38	Friction compounding factor	Friction compounding factor	3
39	Total friction head, m	Total friction head, m	21.94
40	Add safety factor at percent	Add safety factor at percent	10
41	Design friction head, m	Design friction head, m	22.3
42	Number of tees	Number of tees	2
43	Friction factor for tee	Friction factor for tee	1.5
44	Number of valves	Number of valves	2
45	Friction factor for valve	Friction factor for valve	1.4
46	Total friction co-efficient	Total friction co-efficient	6
47	velocity head, m	velocity head, m	0.11
48	Friction loss in bends & tees	Friction loss in bends & tees	0.67
49	Other Losses, m	Other Losses, m	0.33
50	Total friction loss, m	Total friction loss, m	23.27

APPENDIX A 6.4
DESIGN EXAMPLE OF SLUDGE DIGESTERS
(Retained as in 2nd edition)

Design low rate and high rate digesters for digesting mixed primary and activated sludge from a 50,000 m3/day capacity activated sludge Wastewater Treatment Plant.

1 GIVEN

a)	Raw effluent suspended solids (SS) concentration	= 400 mg/l
b)	SS removal efficiency in the primary sedimentation tank	= 75 %
c)	Therefore, quantity of primary sludge generated (0.4 kg/m^3 × 50,000 m^3/day × 0.75)	= 15,000 kg/day
d)	At 4% consistency or 40 kg/m3 SS concentration, primary sludge volume (15,000 kg/day ÷ 40 kg/m^3)	= 375 m^3/day
e)	The excess activated sludge generated	= 2,630 kg/day
f)	At 1% consistency or SS concentration of 10 kg/m^3 the excess activated sludge volume (2,630 kg/day ÷ 10 kg/m^3)	= 263 m^3/day
g)	Total volume of the raw mixed sludge (375 + 263)	= 638 m^3/day
h)	Total quantity of the raw mixed sludge (15,000 ÷ 2,630)	= 17,630 kg/day
i)	SS concentration of the raw mixed sludge (17,630 kg/day ÷ 638 m^3/day)	= 27.6 kg/ m$^{3]}$
j)	The approximate percentage of volatile matters (VM) in the mixed sludge	= 70 %
k)	Quantity of VM in the raw mixed sludge (0.7 × 17,630)	= 12,341 kg/day
l)	Quantity of Non-VM or inorganic (0.3 × 17,630) * 1% consistency = 10,000 mg/l = 10 kg/m^3	= 5,289 kg/day

2 LOW RATE DIGESTER

a)	Approximate percentage destruction of VM (design value)	= 50%
b)	For achieving 50 % VM destruction, under mesophilic conditions, the HRT required (from Figure.6.5)	= 40 day
c)	Quantity of VM in the digested sludge (0.5 × 12,341)	= 6,170 kg/day
d)	Quantity of non-volatile matters or inorganic matters in the digested sludge	= 5,289 kg/day
e)	Total quantity of solids in the digested sludge (6,170 + 5,289)	= 11,459 kg/day
f)	Percentage of VM in the digested sludge (6,170 + 11,459)	= 53.80 %
g)	Percentage of inorganic matter in the digested sludge (5,289 + 11,459)	= 46.20 %

h) Depending on the frequency of sludge withdrawal the consistency of the digested sludge withdrawn from the low rate digester is expected to be in the range of 4 - 6 %.

i) For an average consistency of 5 % (or 50 kg/ m^3),
the volume of digested sludge (11,459 ÷ 50) = 229 m^3/day

j) Therefore the volume of digester
$V = [V_f – 2/3 (V_f - V_d)]$ T1
= [638 – 2/3 (638 – 229)] × 40
= 14,624 m^3
Check for volatile solids loading rate kg VSS/day/m^3
= 12,341 ÷ 14,624 = 0.84 kg VSS/day/m^3
(The VSS loading is within the permissible range - 0.6 to 1.6 kg VSS/day/m^3)

3 GAS GENERATION

a) Gas production per kg of VM destroyed = 0.9 m^3
b) Total gas generation (0.9m^3/kg VM × 6,170 kg VM/day) = 6,039 m^3
c) To avoid foaming, the minimum surface area required
to meet the condition - 9 m^3 of gas generated per
day per m2 surface area, will be (6,039 ÷ 91) = 617 m^2
d) For operational flexibility and constructional reasons, it is suggested
to install two digesters of the following dimensions.
e) Volume of each digester (14,624 m^3 ÷ 2) = 7,312 m^3
f) Minimum surface area of each digester (617 m2 ÷ 2) = 309 m^2
g) Choosing the digester shape as a low, vertical cylinder and for
a diameter of 34 m, the surface area of each digester will be = 908 m^2
h) Therefore the effective digester depth will be
(7,312 m3 ÷ 908 m^2) = 8.0 m

4 ADDITIONAL VOLUME

a) Volume for sludge storage during the monsoon period –when the sludge drying bed option
is used for sludge dewatering = Vd × T2
b) For a storage period of 12 days (229 m^3/day × 12 days) = 2,748 m^3
c) Equivalent to 2,748 m^3 ÷ 908 m^2 = 3.0 m
d) Additional allowance for grit and scum Accumulation = 0.6 m
e) Free board = 0.6 m
f) Therefore total additional depth = 4.2 m

Two digesters – each of 34 m diameters & 12.2 m depth

5 HIGH RATE DIGESTERS

a) For a sludge temperature of 24°C, the Solids Retention Time (SRT) required for 50% VSS
destruction (refer Table 6.10) = 20 days
b) Therefore the digester volume will be = 638 × 20
c) (Volume of fresh sludge × Retention time) = 12,760 m^3
d) Choosing two digesters, the capacity of each digester will be:
Volume (12,760 m3 ÷ 2) = 6,380 m^3

e) Choosing a diameter of 27 m, the effective depth will be = 11.2 m

f) Additional allowance for grit accumulation = 0.5 m

g) Free board = 0.6 m

h) Total additional depth = 1.1 m

Two digesters of 27 m diameter and 12.3 m depth

Additional, separate sludge holding facility for storage during monsoon period (when sludge drying bed option is used for dewatering) is to be computed as before.

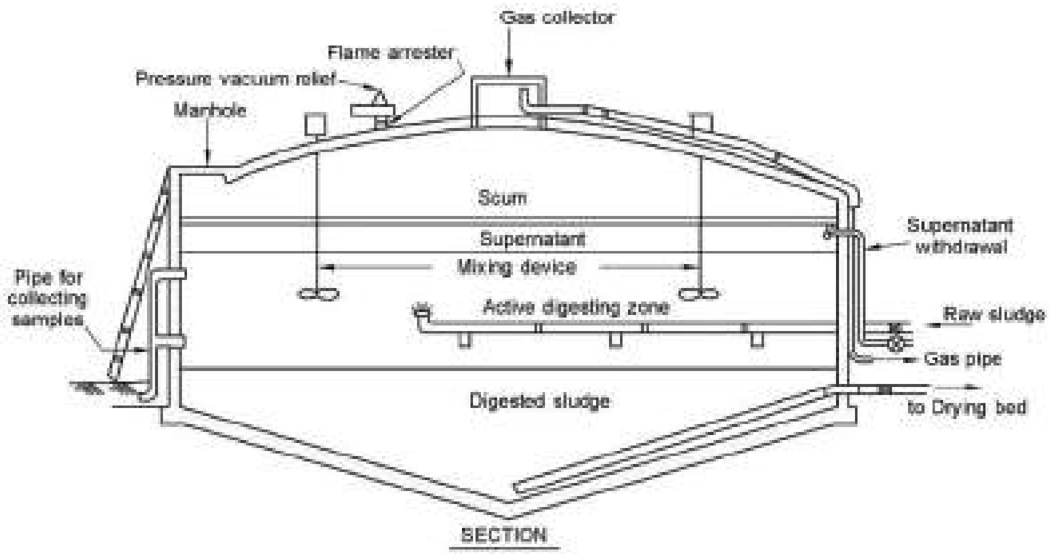


Figure A6.4-1 Typical Details of Low rate Sludge Digester

APPENDIX A 6.5
DESIGN EXAMPLE OF SLUDGE DRYING BEDS
(Retained as in 2nd edition)

1 PROBLEM STATEMENT

Design sludge drying beds tor digested sludge obtained from low rate anaerobic digesters for digesting a mixture of primary and excess activated sludge. The capacity of activated sludge plant is 50,000 m^3/d and following data is assumed:

i)	Volume of digested sludge (Refer to design example on low rate anaerobic digester in Appendix 6.4)	= 229 m3/day
ii)	Dewatering, drying and sludge removal cycle	= 10 d
iii)	Depth of application of sludge	= 0.3 m

2 SOLUTION

i) Total plan area of sludge drying $= \dfrac{229 \times 10}{0.3}\ m^2 = 7{,}633\ m^2$

ii) Number of beds is assumed to be = 30

iii) If per capita wastewater flow is assumed as 150 lpcd

Contributory design population $= \dfrac{50{,}000 \times 10^3}{150} = 3{,}33{,}333$

Plan area of sludge drying bed = 7,633 ÷ 3,33,333 = 0.023 m^2/capita

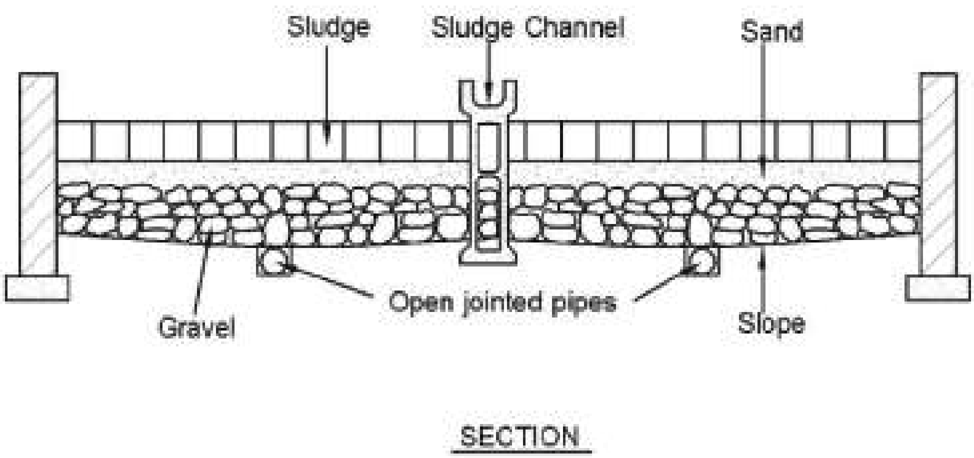


Figure A6.5-1 Typical details of sludge drying bed

Appendix A7-1

Table of Micro Nutrients to be added to biological systems per month till MLSS builds up to 3000 mg/l for continuous flow reactors (Source, Eckenfelder)

A	B	C	D	E	F	G	H	I	J	K
Sewage in MLD			4	BOD at Inlet	300					
		Atomic	mg per mg of BOD	Market Chemical	Chemical Formula	Molecular	mg per mg of BOD	Kg	Rs	Cost
		Weight	as element			Weight	as compound	per month	per Kg	per month
									(may vary)	
Calcium	Ca	40	0.0062	Calcium carbonate	CaCo3	100	0.0155	18.60	120	2232.00
Cobalt	Co	58	0.00013	Cobaltic chloride	CoCl2(6H2O)	238	0.000533448	0.64	3400	2176.47
Copper	Cu	64	0.00015	Cupric sulphate	Cu(SO4)	160	0.000375	0.45	340	153.00
Iron	Fe	56	0.012	Ferrous ammonium sulphate	FeSO4 (NH4)2(SO4) (6 H20)	392	0.084	100.80	260	26208.00
Magnesium	Mg	24	0.003	Magnesium chloride	MgCl2	95	0.011875	14.25	200	2850.00
Manganese	Mn	55	0.0001	Manganese chloride	MnCl2(4H2O)	198	0.00036	0.43	280	120.96
Molybdenum	Mo	96	0.00043	Molybdic acid	MoS2	106	0.000474792	0.57	3500	1994.13
Potassium	K	39	0.0045	Potassium chloride	KCl	75	0.008653846	10.38	240	2492.31
Selenium	Se	79	0.0014	Selenium chloride	SeCl4	228.825	0.004055127	4.87		
Sodium	Na	23	0.00005	Sodium chloride	NaCl	59	0.000128261	0.15	120	18.47
Zinc	Zn	66	0.00016	Zinc oxide	ZnO	82	0.000198788	0.24	300	71.56
									Rs/Kl	0.32

The values in column (I) are for the indicated feed raw sewage of 4 MLD and feed raw BOD of 300 mg / l, For other values, it will be prorate.
For example, if raw sewage is 20 MLD and raw BOD is 450 mg/l, Ca need will be 18.6 * (20*450) / (4*300) = 140
These can be added even in shorter intervals than a month if the need arises. Cost of chemicals as given is only for illustrative calculations.

Appendix A7-2

Table of Micro Nutrients to be added to biological systems per month till MLSS builds up to 3000 mg/l for batch flow reactors

A	B	C	D	E	F	G	H	I	J	K
volume of reactor in cum			1500	BOD of feed	200					
		Atomic	mg per mg of BOD	Market Chemical	Chemical Formula	Molecular	mg per mg of BOD	Kg	Rs	Cost
		Weight	as element			Weight	as compound	per month	per Kg	per month
									(may vary)	
Calcium	Ca	40	0.0062	Calcium carbonate	CaCo3	100	0.0155	4.65	120	558.00
Cobalt	Co	58	0.00013	Cobaltic chloride	CoCl2(6H2O)	238	0.00053344 8	0.16	3400	544.12
Copper	Cu	64	0.00015	Cupric sulphate	Cu(SO4)	160	0.000375	0.11	340	38.25
Iron	Fe	56	0.012	Ferrous ammonium sulphate	FeSO4 (NH4)2(SO4)(6 H20)	392	0.084	25.20	260	6552.0 0
Magnesiu m	Mg	24	0.003	Magnesium chloride	MgCl2	95	0.011875	3.56	200	712.50
Manganes e	Mn	55	0.0001	Manganese chloride	MnCl2(4H2O)	198	0.00036	0.11	280	30.24
Molybden um	Mo	96	0.00043	Molybdic acid	MoS2	106	0.00047479 2	0.14	3500	498.53
Potassium	K	39	0.0045	Potassium chloride	KCl	75	0.00865384 6	2.60	240	623.08
Selenium	Se	79	0.0014	Selenium chloride	SeCl4	228.825	0.00405512 7	1.22		
Sodium	Na	23	0.00005	Sodium chloride	NaCl	59	0.00012826 1	0.04	120	4.62
Zinc	Zn	66	0.00016	Zinc oxide	ZnO	82	0.00019878 8	0.06	300	17.89
									Rs/day	1368

The values in column (I) are for the indicated feed volume of reactor of 1500 cum & feed raw BOD of 200 mg / l. For other values, it will be prorate. For example, if volume of reactor is 2500 cum and raw BOD is 450 mg/l, Ca need will be 4.65*2500*450/1500 / 200 = 17.44
These can be added even in shorter intervals than a month if the need arises.

APPENDIX A 7.3
CASE STUDIES IN RECYCLING AND REUSE OF SEWAGE

A complete documentation of all the technologies, design basis and engineering practices will be not only voluminous but also not directly applicable in every other place and as such, only the key issues are brought out here and a reference is available in the end for a detailed learning of these comprehensively. The case studies are arranged in the following order:

1. Agriculture
2. Farm Forestry
3. Horticulture
4. Toilet flushing
5. Industrial and commercial
6. Fish culture
7. Groundwater recharge
8. Indirect recharge of impoundments

1 AGRICULTURE

The risk in using inadequately treated sewage in agriculture in general poses the health hazards as enunciated by the WHO and extracted hereunder.

Table A7.3- 1 Health hazards of inadequately treated sewage in agriculture

Type of pathogen / infection	Relative excess of frequency of infection or disease
Intestinal nematode infections (*Ascaris lumbricoidus, Trichuris trichiura*, hookworm)	High
Bacterial infections Bacterial diarrheas (e.g. cholera, typhoid)	Lower
Viruses Viral diarrheas Hepatitis A	Lowest

Source: World Health Organization 1989

These infections can virtually reduce the effective man days by as much as even 50 % especially in rural habitations where neither prophylaxis not cure is easily accessible and if at all, financially in out of reach unless subsidized. It is not that sewage cannot be used at all. It only requires appropriate treatment. The reported specific experiences are hereunder.

1.1 ISRAEL

There are approximately 200 numbers of deep surface reservoirs in operation throughout the country with a total storage capacity of 150 Mm^3 that are used to store treated sewage during the winter season and the water is then used during the summer season for irrigation crops and animal fodder thus reusing nearly 70% of the treated sewage which equates to nearly 400 Mm^3 per year. The Dan Region reuse system serves the Tel Aviv metropolitan area treats 120 Mm^3/year of Tel Aviv sewage which is stored in recharge aquifer basins and then pumped from recovery wells and conveyed to irrigation. The Kishon facilities treat 32 Mm^3/year of sewage from the Haifa metropolitan area and the treated sewage is conveyed beyond 30 km where it is blended with local storm water and stored in a 12 Mm^3 reservoir for irrigation of 15,000 ha of cotton and other non-edible crops. There are 3 other similar reuse projects in the Jeezrael Valley for 8 Mm^3/year, Gedera for 1.5 Mm^3/year and Getaot Kibbutz for 0.14 Mm^3/year. This emphasises the importance of reuse projects.

1.2 TALLAHASSEE, FLORIDA, US

The Tallahassee agricultural reuse system is a cooperative operation where the city owns and maintains the irrigation system, while the farming service is under contract to commercial enterprise. The reuse dates back to 1966 and until 1980, was limited to irrigation of 50 ha of land used for hay production. Based upon success of the early studies and experience, new spray field has been expanded to approximately 840 ha with an application rate of 8 cm per week on the soils of 95% sand and interspersed clay layer at a 10 m depth and the field is sloping from 21m to 6m above sea level. The treated sewage meets BOD of 20 mg/l for BOD and TSS and MPN of 200 per 100 ml for faecal Coliform and is pumped over 13 km and distributed via 16 centre-pivot irrigation units. Major crops are corn, soybeans, coastal Bermuda grass, and rye with the rye and Bermuda grass being grazed by cattle, though some of the Bermuda grass is harvested as hay and haylage.

1.3 AFRICA

Raw sewage farming for vegetables is used in Eritrea in Northeast Africa in an area of 124,320 km^2 and a 3.5 million population set at 1,700 m above sea level with water scarcity which forces the crudely treated sewage use for vegetables. The pathogens detected in the vegetables are shown hereunder:

Table A7.3- 2 Pathogen detected in vegetables grown from crudely treated sewage at Eritrea, Africa

Types of vegetables grown	Sampling sites	No. of sample	Faecal coliform		E. Coli		Giardia	
			No. + ve	%. + ve	No. + ve	%. + ve	No. + ve	%. + ve
Cabbage	4	8	6	75	6	75	4	50
Lettuce	4	12	11	96.6	6	96.6	6	50
G.vegetable	4	12	12	100	11	100	3	25
Tomatoes	3	9	9	100	12	100	2-	22.0
Carrots	3	9	6	66.6	9	66.6	ND	-
Cucurbits	3	6	6	100	6	100	ND	-
Total	16	62						

ND indicates not dctoctcd.
Note: No Salm onclla was detected, Shigclla was detocted in one let tuce sample.

The faecal Coliforms in the sewage used for irrigation was 4×10^4 to 13×10^9 per litre as compared to a limit of 1,000 per 100 ml and in the vegetables it was 2×10^3 to 4×10^6 per kg. Among the local farmers in the field, out of a sample of 75 persons, 34 had Giardia Lamblia. But there was surprisingly no detection of round worm or hook worm, Shigella or Salmonella. This might have been due to the combination of prophylactic methods in treading on farm soil and drinking water sources were not affected by sewage but eating green salad as a standard practice.

1.4 FINDINGS BY THE LONDON SCHOOL OF TROPICAL MEDICINE & INTERNATIONAL WATER MANAGEMENT INSTITUTE, HYDERABAD, INDIA

An evaluation of the situation at Hyderabad when the city sewage was not treated and used for irrigation of nearly 3,100 hectares of land downstream of the discharge river course and the quality issues are hereunder:

Table A7.3- 3 Effect of exposure to untreated sewage and partially treated sewage on Ascaris Lumbricoides, Hookworm, Heavy hookworm and Trichuris Trichiura infection at Hyderabad

Characteristic		Water quality, ova/liter	OR*1	95%CI	P
A. lumbricoides *2					
	River water	0	1.0		
	Partially treated wastewater	12	3.2	1.2-8.6	0.02
	Untreated wastewater	70	5.3	2.0-14	0.001
Hookworm*3					
	River water	0	1.0		
	Partially treated wastewater	15	0.7	0.4-1.1	0.11
	Untreated wastewater	76	3.5	2.2-5.5	<0.001
Hookworm (epg > 160)*4					
	River water	0	1.0		
	Partially treated wastewater	15	0.8	03-2.2	0.65
	Untreated wastewater	76	3.9	1.5-9.9	0.004
*T. trichiura**5					
	River water	0	1.0		
	Partially treated wastewater	0.3	0.6	02-2.5	0.53
	Untreated wastewater	4	5.6	1.8-18	0.003

*1 OR = odds ratio, CI = confidence interval: epg = eggs per gram (of feces).
*2 Controlled for age. sex. education, and household clustering,
*3 Controlled for age. education, caste, latrine presence, and household clustering.
*4 Controlled for age. sex. education, type of water supply, agricultural activities involved in. and household clustering.
*5 Controlled for sex. education, and household clustering.

Although this study was unable to provide conclusive evidence about the validity of the current WHO wastewater nematode guideline, the findings suggest that for A lumbricoides and T trichiura infection, the WHO nematode guideline of less than or equal to one ova per litre is appropriate but the nematode guideline of less than or equal to one per litre when exposed to children is too strict. No increased risk of hookworm infection was detected when wastewater with a mean concentration of

15 ova per litre was used, which would suggest that at least for the risk of hookworm infection, the WHO nematode guideline is too strict and a more lenient guideline can be set if hookworm species is the predominant ova in wastewater. However, more studies are needed to confirm this suggestion.

1.5 FINDINGS OF THE INSTITUTE OF PUBLIC HEALTH LAHORE AND INTERNATIONAL WATER MANAGEMENT INSTITUTE, LAHORE, PAKISTAN

The study was carried out around the town of Haroonabad a small town with about 80,000 inhabitants in the southern part of the Punjab Province in Pakistan. Around this town agricultural areas are irrigated with untreated urban wastewater for about 30 years for vegetables sold in the city. In the middle of this main site there is a colony and most farmers live here. There are two peri-urban settlements and all three settlements are connected with the municipal relative reliable water supply. There is an organized system for disposal of sewage water in a pond, either by drains or pipes or by carts. The farmers in the villages use canal or tube well water. This provides as ideal setting for a relative evaluation of farm workers from sewage farm and fresh water farm. The findings of incidence of diseases related to the practices are extracted hereunder:

Table A7.3- 4 Prevalence of diseases by exposure to wastewater with odds ratio and 95% confidence interval

Disease	Exposed		Unexposed		OR	96% CI
Diarrhea	11 7%	(23)	6.2%	(17)	2.00	1.04-3.85
Diarrtea complicated with fever	56%	(11)	1.8%	(5)	3.18	1. 09-9.31
Dysentery	0.5%	(1)	0		-	-
Skin problems	30%	(6)	5.8%	(16)	0.51	0.20-1.32
Nail problems	7 1%	(14)	2.2%	(6)	3.42	1.29-9.05
Typhoid	0		0.7%	(2)	-	-
Fever/cold	11 7%	(23)	11.7%	(32)	1.00	0.57 – 1.77
Total number of health questionnaires	197		274			

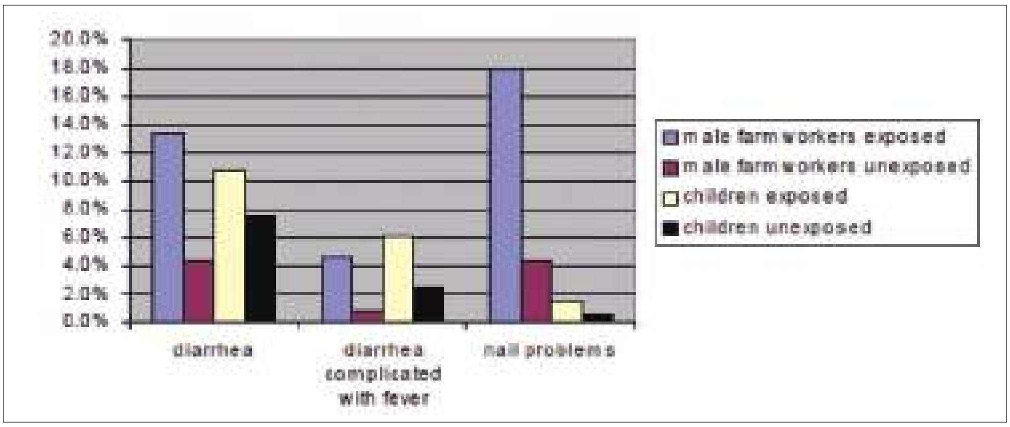


Figure A7.3- 1 Prevalence of diseases in male adult farm workers and children by exposure to wastewater

The classical finding of this study was the health guidelines for faecal coliform bacteria in wastewater from the WHO seem difficult to realize in the tropical climate of the Punjab Province, since canal water also exceeds the guidelines regularly. However, an appropriate form of treatment of wastewater for helminth eggs and faecal coliform bacteria before application to the fields is highly recommended. If treatment is not possible because of the high costs, other protective measures should be taken. Low cost interventions could include information on hygiene behaviour for farmers, wearing of shoes and gloves while working in wastewater irrigated fields, regular treatment of farmers and their families with anti-helminthic drugs and crop restrictions in wastewater irrigated fields.

1.6 FINDINGS OF THE UNIVERSITY OF JERUSALEM

Though this study covers an outdated period of early 20th century, the lessons there from are worth learning for posterity. Some the extracts from this study are shown hereunder:

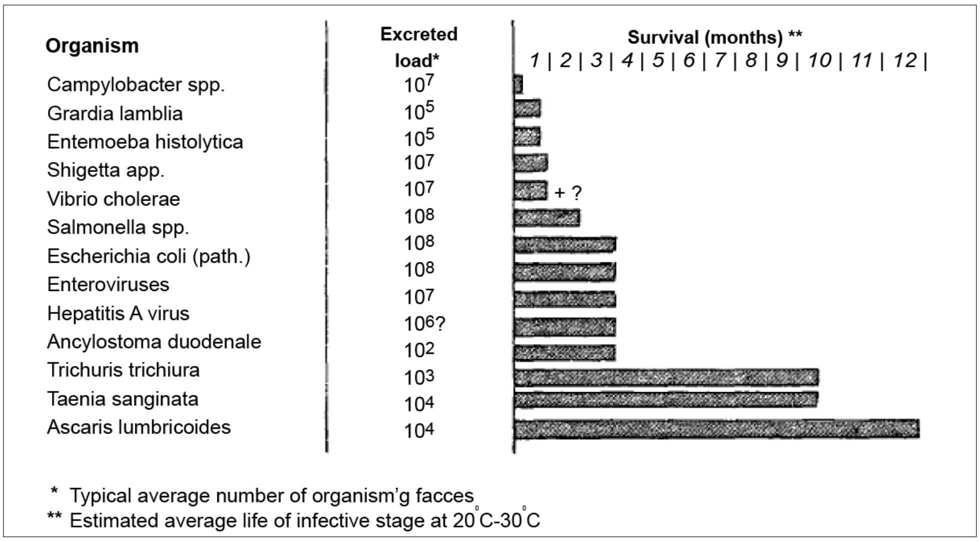


Figure A7.3- 2 Persistence of selected enteric pathogens in water, wastewater, soil and crops

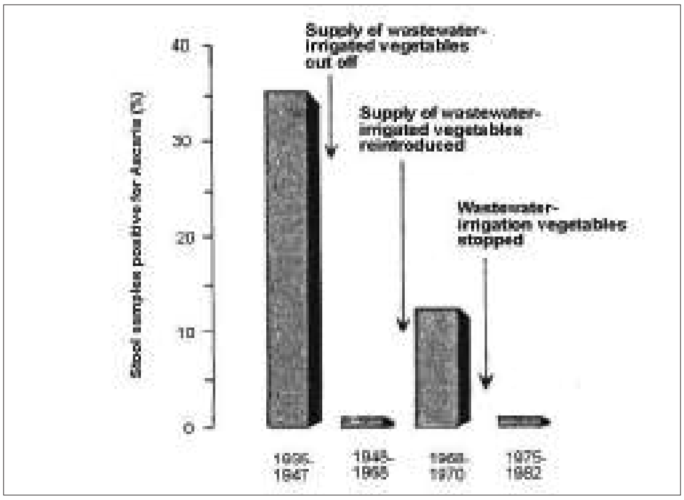


Figure A7.3- 3 Relationship between Ascaris-positive stool samples in the population of western Jerusalem and supply of vegetable and salad crops irrigated with raw wastewater in Jerusalem, 1935-1982

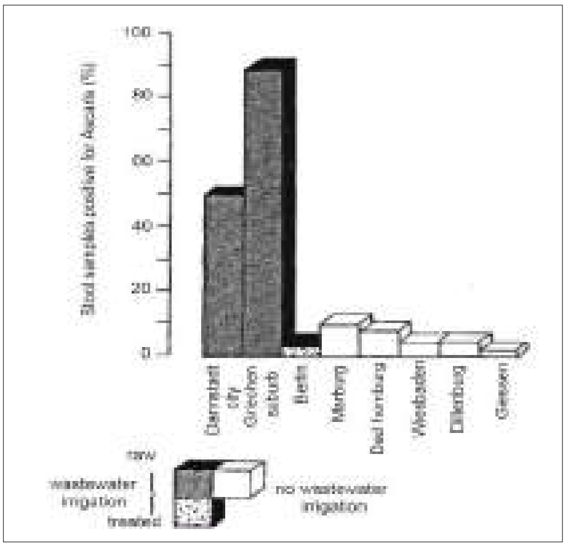


Figure A7.3- 4 Wastewater irrigation of vegetable and Ascaris prevalence in Darmstadt, Berlin and other German cities, 1949

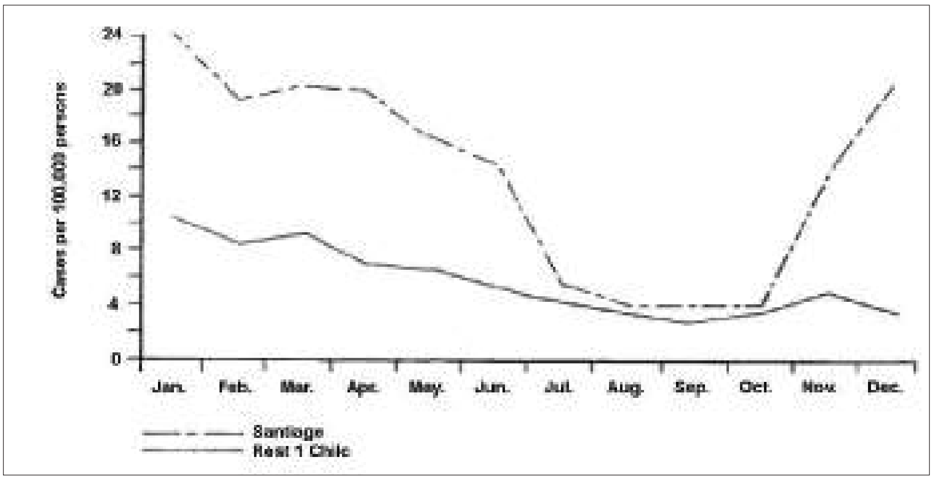


Figure A7.3- 5 Seasonal variation in typhoid fever cases in Santiago and the rest of Chile (average rates, 1977-1983)

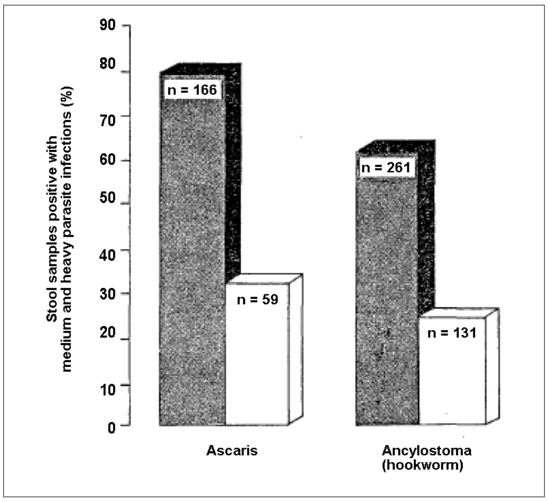


Figure A7.3- 6 Intensity of parasitic infection in sewage farm workers and controls in various regions of India

Their findings concluded that the WHO guidelines for crops eaten uncooked (and on pasture land where farm animals are grazed) that an effluent must contain one or fewer helminth eggs per litre, with a geometric mean of faecal coliforms not exceeding 1,000/100 ml can be readily achieved with low-cost, robust stabilization pond systems which are particularly suited to developing countries and high levels of pathogen removal can be achieved by such low-cost systems as shown hereunder.

Table A7.3- 5 WHO Recommended microbiological quality guidelines for wastewater use in agriculture

Category	Reuse conditions	Exposed group	Intrstinal Nematodes (b) (arithmetic Mean no. of Eggs per litre)	Faecal Coliforms (geometric Mean no. per 100ml) ©	Wastewater Treatment expected to Achieve the required microbiological quality
A	Irrigation of crops Likely to be eaten Uncooked, sports Fields, public Parks (d)	Workers, Consumers, Public	<1	< 1,000(d)	A series of tabilisation Ponds desigied to achieve The microbik equivalent indicated, or equivalent Treatment
B	Irrigation of cereal crops, industrial crops, fodder crops, pasture and trees (e)	Workers	<1	No standard recommended	Retention in stabilisation ponds for 8-10 days or equivalent helminth and faecal coliform removal
C	Localised Irrigation Of crops in category B If exposure of workers and the public Does not occur	None	Not applicable	Not applicable	Pre - treatment as required by the irrigation technology, but not less than primary sedimentation

A) In specific cases, local epidemiological, socio – cultural and environmental factors should be taken into account, and guidelines modified accordingly
b) Ascaris and Trichuris species and hookworms
c) During the irrigation Period
d) A more stringent guidation (<200 faecal coliforms /100ml) is appropriate for public lawns, such as hotel lawns, with the public may come into direct contact
e) In the case of fruit Lees , irrigation should cease two weeks before fruit is picked, and no fruit should be picked off the ground. Sprinkler

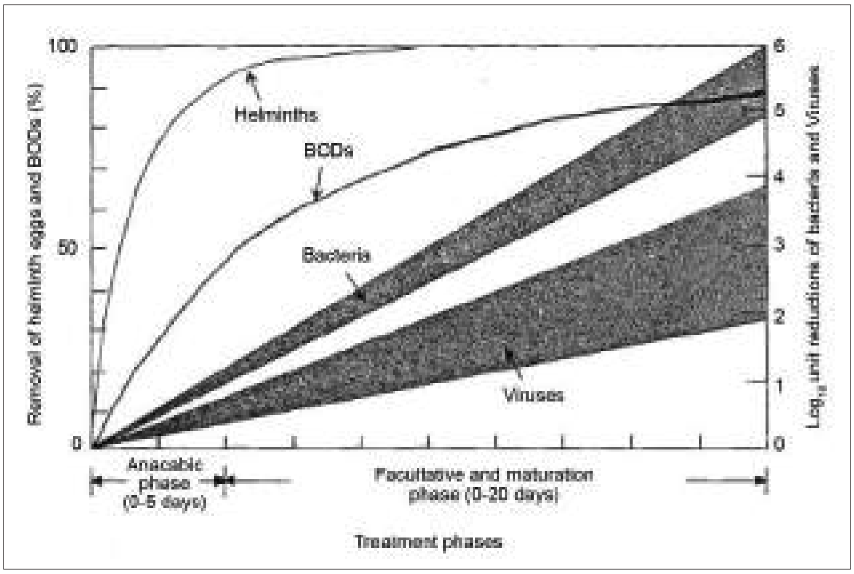


Figure A7.3- 7 Generalized removal curves for BOD, helminth eggs, excreted bacteria and viruses in waste stabilization ponds at temperatures above 20ºC

This study is of great relevance to India in appropriately treating the sewage of rural habitations by such natural pond systems without complicating sophisticated treatment technologies and ensuring the safe reuse of pond effluents for potential agriculture of non-edible crops. In 1970, an outbreak of cholera of some 200 cases occurred in Jerusalem and investigation provided a strong evidence that the main route of transmission was through the consumption of vegetables, including lettuce and cucumbers illegally irrigated with untreated sewage from Jerusalem and which villagers sold door-to-door throughout the city.

1.7 FINDINGS OF NEERI IN INDIA

A study of stool examinations of workers at five large sewage farms at Jaipur, Madras, Hyderabad, Trivandrum and Pune in India during 1973 revealed that out of 466 samples from sewage farm workers compared with 432 samples from a control population for the presence of Ancylostoma duodenale (hook worm), Ascaris lumbricoides (round worm), Trichuris trichiura (whip worm), Enterobius vermicularis (pin worm), Hymenolepis nana (dwarf tapeworm), Entamoeba histolytica and Giardia intestinalis, the incidence and multiplicity of infection was greater in sewage farm workers.

1.8 GUANAJUATO RIVER BASIN, MEXICO, SEWAGE RIGHTS ISSUES

The total sewage generated in Guanajuato’s 46 municipalities is 567 MLD and could be used for agricultural of 20,000 ha. There is little or no expected impact on the nutrient value resulting from treatment, given that the nutrient requirements of the principal crop, alfalfa, would continue to be met even after treatment. Additionally, other sources of untreated urban wastewater enter the river down-stream of the treatment plant, entailing sufficiently high nutrient loads that little effect of treatment was perceptible to the farmers. The benefits from the waste solid sludge are being lost because these go directly to a landfill instead of being spread on agricultural land. The areas of further study are (a) the conditions required for wastewater markets to function, specifically commercial feasibility for irrigation use of treated vs. untreated wastewater, pricing and supply mechanisms, etc. (b) water rights conflicts, (c) hydrological impact of selling the treated water outside the sub-basin, (d) water quality assessment of the final use, e.g. at the farm level for irrigation and (e) accounting for the nutrients lost in the treatment process. This is a classical case of inter basin rights of treated sewage.

1.9 TYPHOID FEVER OUTBREAK IN SANTIAGO, CHILE

In 1978 and 1983, the outbreak of typhoid fever in Santiago has been reported due to the use of untreated sewage for the irrigation of 13,500 ha of tomatoes, lettuce, cabbage, celery and cauliflower that were consumed raw.

1.10 THE SULAIBIA REUSE FACILITY, KUWAIT

By far the world’s largest of its kind, this facility extends the secondary sewage treatment beyond the conventional limits to nutrients removal, ultra filtration and reverse osmosis to a mammoth scale of 375 MLD with provision to expand to 600 MLD and when commissioned in 2004, it won ‘Wastewater Project of the Year’ in the 2005 Global Water Awards and described by the judges as a “Powerful statement of the future of water resources across the whole Middle East and North Africa region". The agriculture and industry demands were met till then by brackish ground water which slowly increased in its salt concentration and this project seeks to reverse it by substituting the agriculture and industry demands and aquifer recharge as well. The sale price for kilolitre was $ 0.60 for freshwater, 0.08 for secondary sewage and 0.15 for RO treated sewage it being colourless, clear and sparkling.

2 FARM FORESTRY

2.1 ABU DHABI

It is extensively practiced since 1976. The system, designed for 190 MLD is a dual distribution network for irrigation of 15,000 ha of urban forests, public gardens, trees, shrubs, and grassed areas along roadways. Secondary treated sewage is further provided tertiary treatment with rapid sand filtration, chlorination and ozonation. The infra-structure is painted purple and labelled to avoid any cross-connections.

2.2 EGYPT

Egypt exports some fruits and vegetables and hence use of treated sewage is impossible in this segment. However, considering the barren sandy terrain, the ministry promulgates a policy of "Treated sewage + sandy desert soil = Green Trees" and already 1,800 hectares are under cultivation with Eucalyptus, Mahogany, African Mahogany, Terminalia, Acacia and Neem trees with irrigation by modified flood or drip irrigation.

2.3 WOODBURN, OREGON, USA

In 1999, Woodburn was ready to discharge 6 MLD of treated sewage through micro-spray sprinklers into its newly developed 40 hectare poplar tree plantation. Sewage is treated using a biological nutrient removal aeration basin, a clarifier, sand filters, disinfection in an ultraviolet unit, chlorination and is pumped into a manifold system to the plantation through an underground piping system where micro-spray sprinklers are used and there is a 10 m offset to the property line where irrigation does not occur. In addition, the plantation is completely fenced and surrounded by signs warning intruders that treated effluent is being discharged. The irrigation system can be shut down in 2.5 hectare blocks, providing the capability for spot irrigation and operated from a microcomputer. The field was ploughed, levelled, and then grass and 10 cm cuttings were planted into the bare ground and when irrigated, they had a weed control problem and had to manually remove the grass from around the cuttings. This would condition the soil and there wouldn't be any weeds. Then they would plant the trees in the stream lines and rows without disturbing the soil. This way, grass was already growing and weeds were not a big problem. This natural system creates an attractive habitat for wildlife, provides 30 to 50% more evapotranspiration capacity than would a different crop of equal size, and provides a new source of revenue. Trees can be harvested every 7 to 12 years, and revenue from the sale of woodchips can be used to offset a portion of the capital and operation and maintenance costs of the system. The city plans to expand the facility every 5 years to match population growth. By 2020, the site will cover 140 hectares acres and will reuse 20 MLD of treated sewage.

2.4 WIDE BAY WATER CORPORATION, QUEENSLAND, AUSTRALIA

Wide Bay Water Corporation has come up with a winning solution for the dispersal of their treated sewage by recycling it onto a timber plantation of 300,000 trees on 220 hectares as of 2008 with expansion plans to a million trees by 2010 by a centrally controlled automated drip irrigation system from two sewage treatment plants. A 130 micron disc filtration technology was used to pre filter the treated sewage before putting into the drip irrigation. The monitoring consisted of a central base unit which is interfaced to a PC for ease of programming, viewing system status, water usage, graphing and SMS alarming, as well as graphical mapping and additional management programs. Currently, the base unit manages the RF communications to three, UHF radio linked field units which are installed at each of the main pumping sites. Additional sensors were installed to monitor water flow, pH and dam levels. The remote infield valve control and water meter monitoring was automated by Piccolo XR's. The Piccolo XR's utilize a low power licensed UHF radio, latching output technology and store & forward (repeater) communications. This combination allows low cost, yet simple and reliable communication to the remote fields. A wireless monitoring system was also installed at the site to record local environmental factors, such as a weather station to monitor temperature, humidity, solar radiation, wind speed, rainfall and ET. Soil moisture probes were installed at keys sites to monitor the effects of irrigation, localized rainfall, drainage and plant uptake in the soil profile. The plantation is stated to have won a Federal Government for Innovation in Irrigation, were both financial and environmental.

2.5 AL BIREH, PALESTINE

The sewage treatment facility is designed for 5.7 MLD with extended aeration followed by gravel media filtration for turbidity reduction, a chlorine-dosing unit calibrated to inject chlorine at a rate of 2 mg/l and 400 L vessel that retained the chlorinated water for 30 minutes. No fertilizer was applied in addition to the nutrients presents in the reclaimed wastewater. A nursery of 600 m^2 for annual cultivation of 80,000 seedlings of indigenous trees and cooked vegetables with irrigation system of micro-drippers was opted for. The eggplants were sterilized to ensure a safe distance of 50 cm from the drip lines. Two types of effluents are used being high quality effluent with drip irrigation and very high quality effluent with subsurface irrigation. The regulations, applications and achievements are shown hereunder.

Table A7.3- 6 Summary of Al-Bireh WWTP reclaimed water use achievements

Effluent Type	Regulation	Application	Achievements
High quality	BOD/TSS less than 20/30 mg/l, Faecal Coliform less than 1,000 MPN/100 ml	Orchard, olives, Ornamentals Grape stocks Processed vegetables Restricted area landscaping	High growth High yield
Very high quality	F.C non-detectable Effluent polishing	Cooked vegetables Nursery (eggplants)	High yield No contamination

Regular basis of the reclaimed water, soil and microbiological quality were tested. The test results show that the tertiary treatment generates reclaimed water suitable for unrestricted agriculture reuse application according to Israeli and US EPA guidelines. Crop quality tests showed that eggplants irrigated with reclaimed water were not contaminated with faecal coliform and intestinal viruses. In the nursery, seedling germination rates were high (>90%) and seedlings irrigated with the reclaimed water showed high vegetative growth. The nursery and Alfalfa plantation are shown hereunder.

Figure A7.3- 8 Nursery and Alfalfa plantation at Al-Bireh facility

2.6 SEWAGE SUSTAINED VERMICOMPOSTING

In the matter of producing vermin compost for tree plantations, a study by University of Agricultural Sciences, Dharwad, Karnataka found that sewage could be used in vermicomposting provided its details with respect to composition of toxic substances are known.

3 HORTICULTURE

3.1 EL PASO TEXAS, USA

Because of declining reserves of fresh groundwater and an uncertain supply of surface water, the El Paso Water Utilities has adopted a strategy to curtail irrigation use of potable water by substituting reclaimed municipal effluent. This strategy has been implemented in stages, starting with irrigation of a county-operated golf course using secondary effluent from the Haskell Plant, and a city-owned golf course with tertiary treated effluent from the Fred Hervey Plant. The reuse projects were expanded to use secondary effluent from the Northwest Plant to irrigate a private golf course, municipal parks, and school grounds. Reclaimed water use from the Haskell Plant is also expanded to include parks and school grounds. In these cases, the salinity of reclaimed water ranges from 680 to 1,200 ppm as total dissolved salts (TDS) depending on the plant. Reclaimed water from the Hervey Plant has the lowest salinity (680 ppm), and a large portion of it is now being injected into an aquifer for recovery as potable water.

3.2 DURBIN CREEK (WESTERN CAROLINA)

The Durbin Creek Wastewater Treatment Facility located near Fountain Inn, South Carolina, discharges to Durbin Creek, a relatively small tributary of the Enoree River. Average flow from the Durbin Creek Plant is 5,200 m^3/day with a peak flow of 22,700 m3/day during storm events. The plant is permitted for an average flow of 12,500 m^3/day. The Durbin Creek plant is located on an 81 ha site, half of which is wooded and the remaining half cleared for land application of bio solids. Hay is harvested in the application fields. Much of the land surrounding the plant site is used as a pasture and for hay production without the benefit of bio solids applications.

3.3 CHANDIGARH

Chandigarh is perhaps the first city to have developed infrastructure with pipe lines for a tertiary treatment plant to cover the horticulture needs of its vast 1,500 hectares by nearly 90 MLD out of which 45 MLD capacity is in place and another 45 MLD is under completion.

3.4 DELHI

With the sewage volume at about 4,400 MLD, planned reuse of treated sewage for designated institutional centres have been put in place as (a) Luytens at 90 MLD, (b) Japanese park at 35 MLD, (c) minor irrigation department at 350 MLD totalling 475 MLD.

4 TOILET FLUSHING

4.1 JAPAN

Because of the country's density and limited water resources, water reclamation and reuse programs are not new to Japan. By 1995, 89.6% of cities larger than 50,000 people were sewered, and 72% of the inhabitants of these cities were served with a sewage collection system. Therefore, buildings being retrofitted for flush toilets and the construction of new buildings offer excellent opportunities for reuse. Initially, the country's reuse program provided reclaimed water to multi-family, commercial, and school buildings, with a reclamation plant treating all of the wastewater for use in toilet flushing and other incidental non-potable purposes. Later, municipal treatment works and reclaimed water systems were used together, as part of a dual system, providing more effective and economical treatment than individual reclamation facilities.

In 1998, reclaimed water use in Japan was 130 Mm^3/year, with distribution as shown in Table A7.1- 7. At that time, about 40% of the reclaimed water was being distributed in dual systems. Of this more than 1/3 was being used for toilet flushing, and about 15% each for urban irrigation and cleansing. A wide variety of buildings were fitted for reclaimed water use, with schools and office buildings being most numerous. In Tokyo, the use of reclaimed water is mandated in all new buildings that have floor area larger than 30,000 m^2. Japan offers a very good reuse model for cities in developing countries because its historical usage is directly related to meeting urban water needs rather than only agricultural irrigation requirements. In addition, the reclaimed water quality requirements in Japan are different from the U.S., as they are more stringent for coliform counts for unrestricted use, while being less restrictive for other applications.

Examples of large area water reclamation systems in Japan can be found in Chiba Prefecture, Kobe City, and Fukuoka City. Outside the city limits of each of these urban areas, streams have been augmented, parks and agricultural areas have been irrigated, and greenbelts established with reclaimed water. The price of reclaimed water in these cities ranges from \$0.83/m^3 for residential use to \$2.99/m^3 for business and other uses. This compares with a potable water price range of \$1.08 to \$3.99/m^3 (USEPA, 2004).

Table A7.3- 7 Uses of reclaimed water in Japan

Use	Percent	Mm^3/year
Environmental Water	54%	63.9
Agricultural Irrigation	13%	15.9
Snow Melting	13%	15.3
Industrial Water	11%	12.6
Cleansing Water	9%	11.2

4.2 INDIAN SCENARIO

Reuse of treated sewage for toilet flush has been recognized as a means of water conservation especially in high rise apartments, office complexes, multiplexes, etc. Generally, this flushing consumes about 3 to 4 flushes per day per person and its volume will be about 20 to 30 litres daily. When a 135 lpcd is supplied, by reusing these 30 litres per day per person, we still have to deal with 110 litres per capita daily and this has to be disposed outside the house. But then, the entire 135 lpcd has to be treated before we can recover the 30 lpcd for toilet flush. Alternatively, there has to be twin sewerage within the house so that milder grey water can be separately collected and treated with less strenuous effort before reuse for flush. Such a system is least advisable especially in modern urban habitations where the potential dangers of cross connection in plumbing and nuisance value of dealing with a STP within the dwelling boundary are matters of great reluctance either to be enforced or to be embraced by the occupants. That brings matters to two options namely, condominiums to have such a facility as a centralized option for the entire condominium or a habitation to have a dual pipeline from the STP back all the way to the dwellings supplying such toilet flush grade water.

Obviously, such a new dual pipeline all over the habitation is almost next to impossible to be designed, implemented and attended to in O&M because, here is a system that will deliver as little as 30 lpcd and its design as a gravity pipeline discharging in short stints for rare timings in a 24 hours cycle calls for a pressurized system. The net result of all this is the fact that these are possible only in the case of condominiums and entirely new layouts. In India, the rule of culture far more outweighs the practicalities and culturally, no one can be forced to avail a water supply originating from the refuse of the neighbour and receive it into his habitation. The Karnataka State Pollution Control Board (KSPCB) has laid down that new layouts shall provide their own STPs by new urban standards juxtaposed with the erstwhile standards prescribed by MoEF/CPCB, and followed by KSPCB as well. An extract of the same along with remarks thereon is furnished hereunder.

Table A7.3- 8 Discharge standards for surface water Vs urban reuse standards by KSPCB

No	Parameters as per BIS	Values as per BIS	Proposed by KSPCB
1	pH	6.5-8.5	6.0 -9.0
2	BOD	< 20	< 10
3	TSS	< 30	Not specified
4	Oil & Grease	<10	Not specified
5	Turbidity	Not specified	< 2 NTU
6	E. Coli	Not specified	NIL
7	Res. Chlorine	Not specified	> 1 PPM
All Units except pH and NTU in mg/l			

Considering the E coli issue alone, total coliform is not addressed and thus, enteric viruses which are specific for water borne epidemics through unintentional portals of entry are left out. Thus total coliforms have to be not detected. The technology for this in the condominium scale can be the ultrafiltration membranes which can function under low pressure and trap the viruses as shown in Figure A7.3- 9 overleaf.

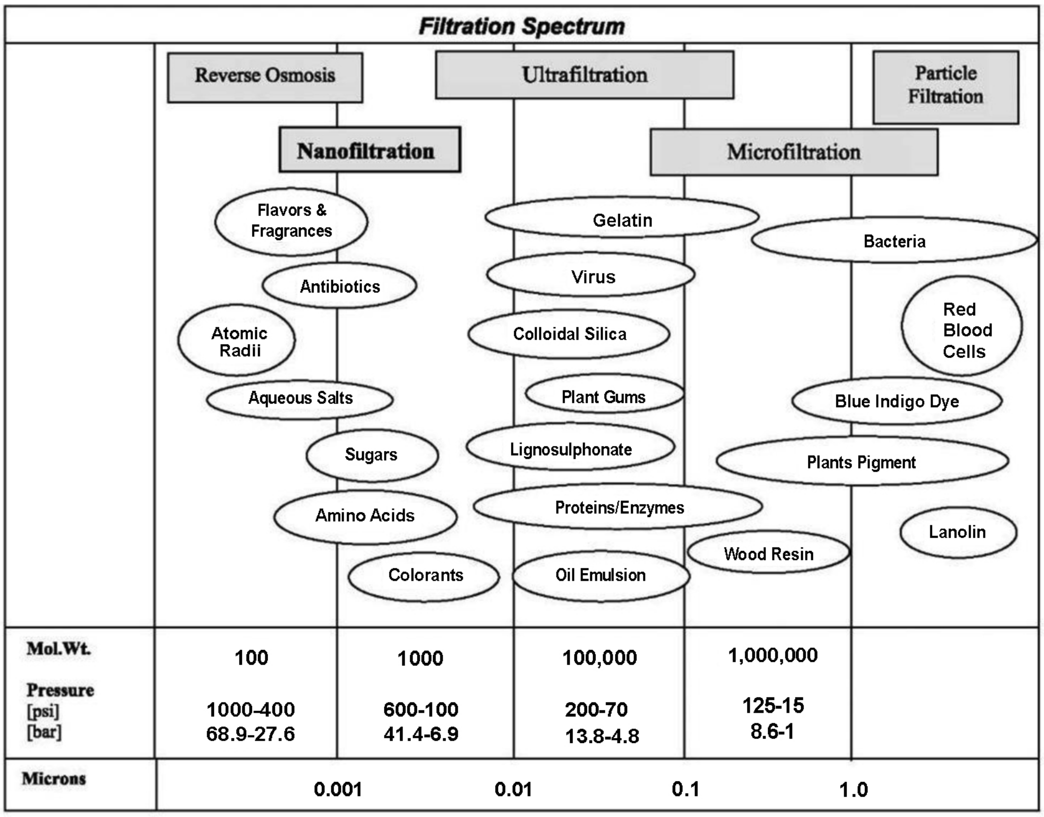


Figure A7.3- 9 Membrane filtration spectrum

5 INDUSTRIAL AND COMMERCIAL

5.1 SAKAIHAMA TREATED WASTEWATER SUPPLY PROJECT, JAPAN

To ensure sustainable water supply in Sakaihama district advanced treatment process was established at 3 STPs. Two types of advanced treatment processes are applied to the effluent obtained from step-feed multistage biological nitrogen removal facilities: 33,000 m^3/day of water is produced using fibre filtration to be used for purposes (cooling water in large industries) in which there is no contact with the human and 1,000 m3/day of water is produced using ozonation for uses in which human contact is expected. Reclaimed water is supplied to large-scale company groups, and small and medium-sized enterprises which include National Soccer Training Centre (NTC), disaster prevention base, Sakai solid waste disposal plant, and local industries. In these facilities, the reclaimed water is used for variety of purposes. In large-scale industries, it is mainly used for cooling purposes, and in small industries it is used for toilet flushing and watering plants through sprinkler. The use also includes sprinkling water over the football ground, and for fire extinguishing in emergency cases. Some hydrants are installed on transmission pipelines for this purpose.

Sakai city depends on water supplies from Osaka Prefecture, for which the main source is Yodogawa River. Considering increase in demand, it became important to secure additional water resources. To ensure sustainable water supply in Sakaihama district (covering area of about 300 ha) and to restore the environment in existing rivers, reuse of treated wastewater (advanced level) came into practice.

For this purpose, advanced treatment process was established at 3 STPs. To achieve the objectives, two types of advanced treatment processes are applied: 33,000 m^3/day of water is produced using fibre filtration to be used for purposes (cooling water in large industries) in which there is no contact with the human and 1,000 m^3/day of water is produced using ozonation for uses in which human contact is expected.

The facilities used for supplying treated water in this project are as follows:

- One water supply pumping station
- 2 satellite treatment plants (ozonation)
- About 12 km of transmission pipelines

Reclaimed water is supplied to large-scale company groups, and small and medium-sized enterprises which include National Soccer Training Centre (NTC), disaster prevention base, Sakai solid waste disposal plant, and local industries. In these facilities, the reclaimed water is used for variety of purposes. In large-scale industries, it is mainly used for cooling purposes, and in small industries it is used for toilet flushing and watering plants through sprinkler. The use also includes sprinkling water over the football ground, and for fire extinguishing in emergency cases. Some hydrants are installed on transmission pipelines for this purpose.

Table A7.3- 9 Treated wastewater supply

User	Quantity (m^3/day)	Proposed by KSPCB
Sakai National Training Centre (NTC)	400	Sprinkling on football ground
Disaster prevention base	500	Sprinkling
Solid waste disposal plant	20	Sprinkling
Local industry business cluster	80	Toilet flushing, Sprinkling
Large companies	33,000	Coolant
Total	34,000	

Figure A7.3- 10 Sakaihama treated wastewater reuse scheme

Table A7.3- 10 Quality of supplied treated wastewater

Parameter	Fibre filtration water	Ozonation water
E. Coli	-	Not detected
pH	5.8-8.6	5.8-8.6
Odour	Not unpleasant	Not unpleasant
Residual chlorine	-	Maintained
Appearance	Not unpleasant	Not unpleasant

Source: http://water.city.sakai.lg.jp/torikumi/file/konwakai4-03.pdf

5.2 BETHLEHEM STEEL MILLS, USA

Perhaps the first industrial use of treated sewage for industrial purpose was initiated here by Abel Wolman. One of the largest reclaimed water user is the Bethlehem Steel Sparrows Point steel mill in Baltimore. For many years, the plant used about 100 MGD of reclaimed water from the Back River Wastewater Treatment Plant. The reclaimed water was used for contact cooling of steel and for other process purposes in the mill.

5.3 HAWAII

The Sewage Treatment facility produces two grades of high-quality recycled water whereby the R-1 water is used for landscape, agriculture, and golf course irrigation, R-2 water which is the reverse osmosis permeate is used for industrial purposes such as boiler feed water and ultra-pure process water. The switching to RO water with only 1 ppm of silica turned into a savings for industrial users, as the ground water arising from filtering of rain water through the lava structure in the ground it picks up between 60 to 70 ppm of silica which scales up the boilers costing the power

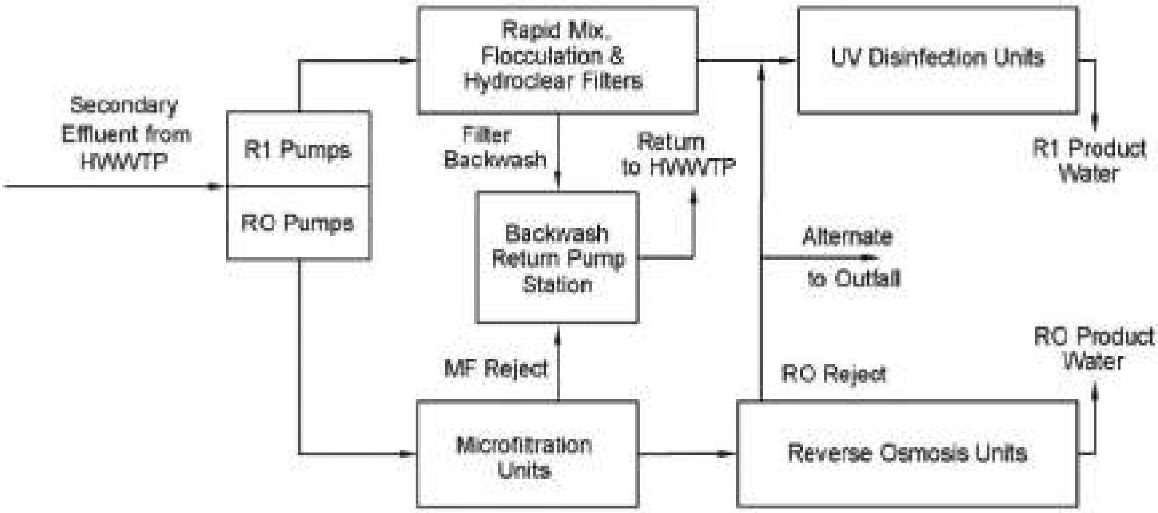


Figure A7.3- 11 Hawaii sewage reuse facility schematic

5.4 DURBAN, SOUTH AFRICA

Durban Metro experienced considerable pressure to provide basic services, including water, to its growing domestic and industrial customers. The natural water resources were not sufficient to meet the increasing demand. As a solution, the KwaZulu Natal pilot project was launched to install and operate a new affordable distribution network for the townships under successful tri-sector partnership (public-private-NGOs). Durban Metro decided to go further by implementing a public-private partnership water reuse project: the Durban Water Recycling project (DWR). The project included treating primary sewage and re-purifying the reclaimed water to the capacity of 47,500 m^3/d. As a result, about 7% of Durban's wastewater is reclaimed as high quality water supplied to the Mondi Paper Mill and SAPREF Refinery at a cost 25% lower than potable water instead of being discharged to the sea. The purification of the wastewater is handled by the newly refurbished wastewater treatment plant applying tertiary treatment using dual media filtration, ozonation, granulated carbon filters and chlorination.

The municipal authority, called "Durban Metro", experienced a dramatic population increase and customers increased from 1 million to nearly 3 million due to the incorporation of 30 local authorities and surrounding townships into the metropolitan area. As a result, Durban Metro experienced considerable pressure to provide basic services to its growing domestic customers, among whom 26% lived in the townships and relied on standpipes for clean drinking water. Moreover, several industries located in this area, such as Mondi Paper Mill and SAPREF refinery needed continuous supply of high quality water for process and cooling purposes. Average rainfall in the region is 200 mm/year, and suffers from periodic droughts. Hence, the natural water resources are not sufficient to meet the increasing demand.

In order to develop a workable solution to the water and sanitation problems, the KwaZulu Natal pilot project was launched as a part of the Worldwide “Business Partners for Development” (BPD) programme created by the World Bank in 1998. This project allowed Durban Metro to install and operate a new affordable distribution network for the townships through innovations in service delivery and tariff structures – first 200 l/day of water was free for domestic customers. This was the result of a successful tri-sector partnership (public-private-NGOs).

Considering the success of this first initiative, Durban Metro decided to go further by implementing a public-private partnership water reuse project: the Durban Water Recycling project (DWR). The project included treating primary sewage and re-purifying the reclaimed water to the capacity of 47,500 m^3/d. As a result, about 7% of Durban’s wastewater is reclaimed as high quality water supplied to the Mondi Paper Mill and SAPREF Refinery at a cost 25% lower than potable water instead of being discharged to the sea. The purification of the wastewater is handled by the newly refurbished wastewater treatment plant which includes tertiary treatment including dual media filtration, ozonation, granulated carbon filters and chlorination. (Mediterranean Wastewater Reuse Working Group, 2007).

6 FISH CULTURE

6.1 HANOI, VIETNAM

Hanoi, the capital of Vietnam has a major system of wastewater reuse involving vegetables, rice as well as fish in low lying Tranh Tri district which lies to the south of the city. Wastewater and storm water are discharged untreated, to four small rivers which play a dual role: drainage of wastewater from the city; and wastewater supply for reuse in agriculture and aquaculture. The system of waste-water reuse has largely been developed by the district farmers and local community over the past 40 years. After 1960s, land use stabilized into vegetable cultivation on higher land, rice/fish cultivation on medium level land, and year-round pond fish culture on deeper land adjacent to the main irrigation and drainage canals. Wastewater-fed aquaculture became the major occupation of cooperatives with easy access to wastewater.

The local aquaculture research institute provided seed of exotic fish species, and fish hatcheries and nurseries were developed by farmers. Farmers also learnt how to regulate the introduction of waste-water to produce fish. The major species are silver carp, rohu, and tilapia. Yields of fish of 3-8 tonnes/ha are harvested annually, lower yields from rice/fish and higher from pond culture.

Hanoi, the capital of Vietnam has a major system of wastewater reuse involving vegetables, rice as well as fish in low lying Tranh Tri district which lies to the south of the city. Produce from the reuse system provides a significant part of the diet of the city’s people.

Wastewater and storm water are discharged untreated, about 320,000 m^3/day, to four small rivers that play a dual role: drainage of wastewater from the city; and wastewater supply for reuse in agriculture and aquaculture. In 2009, only one wastewater treatment plant was reported to be operational and 3 treatment plants under construction. About one-third of the city is sewered. The wastewater is 75-80% domestic and 20-25% industrial.

Source: UNEP webpage: http://www.unep.or.jp/ietc/publications/techpublications/techpub 15/2-9/9-3-1.asp

Figure A7.3- 12 Raw wastewater being pumped into wastewater-fed fish ponds in Hanoi

The system of wastewater reuse has largely been developed by the district farmers and local community over the past 30 years. Before 1960 the area was a sparsely populated swamp where rice was grown but with low yields and frequent flooding. Following the formation of cooperatives in 1967, land use stabilized into vegetable cultivation on higher land, rice/fish cultivation on medium level land, and year-round pond fish culture on deeper land adjacent to the main irrigation and drainage canals. Wastewater-fed aquaculture became the major occupation of 6 cooperatives with easy access to wastewater and a minor occupation of 10 others out of the total of 25 district communes.

The local aquaculture research institute provided seed of exotic fish species, and fish hatcheries and nurseries were developed by farmers. Farmers also learned how to regulate the introduction of wastewater to produce fish. The major species are silver carp, rohu, and tilapia. Rohu and Tilapia has been the most popular species in recent years. Yields of fish of 3-8 tonnes/ha are harvested annually, lower yields from rice/fish and higher from pond culture.

An on-going project to improve the wastewater and drainage system of Hanoi has had only marginal impact on the wastewater-fed fish ponds through loss of a small area to construct a reservoir. A new industrial development area is being established outside the drainage area of the district so fish being cultured on city wastewater should be relatively free of contamination. However, the change in land use policy since the 1980s from cooperative to individual household management has adversely affected wastewater fed aquaculture. Over the decade since 1985 the area of wastewater fed aquaculture (essentially the rice/fish system) has declined in area by 36% from a total of 750 to 480 ha.

6.2 BANGLADESH

In Bangladesh, the development of the first duckweed, conventional wastewater treatment system began, in 1989 at the KHC in Mirzapur. The facility consists of one duckweed covered, 0.7 ha plug flow lagoon constructed as a 500 m long serpentine channel with seven bends. It is fed with a mixture of hospital, school and domestic wastewater from some 2,350 people. The plug flow wastewater-fed duckweed pond is preceded by a 0.2 ha anaerobic pond with a hydraulic retention time (HRT) of 2-4 days. HRT in the plug flow pond is estimated as 21-23 days. Duckweed harvested from the 0.7 ha wastewater treatment pond is fed daily to three adjacent fish ponds, each 0.2 ha.

The duckweed removes nutrients and the plant cover suppresses phytoplankton growth. Average removal efficiencies for BOD5, N and P, and faecal coliforms are 90-97%, 74-77%, and 99.9%, respectively. Effluent turbidity is always below 12 NTU. The effluent is used to top up the water level of the adjacent fish ponds. The wastewater treatment system produces from 220 to 400 tonnes fresh weight duckweed/ha/year (about 17 to 31 tonnes dry weight/ha/year). The fish ponds are stocked with Indian major carps (rohu, mrigal and catla), Chinese carps (grass carp and silver carp), and common carp. Fish production varies from 10 to 15 tonnes/ha/year, yield being relatively high because of frequent harvesting and addition of other feed besides duckweed such as oil cake and rice bran.
Duckweed based wastewater treatment and reuse, Mirzapur

PRISM (Project in Agriculture, Rural Industry Science and Medicine), a non-government organization (NGO) in Bangladesh, has carried out a research and development programme with duckweed based, wastewater treatment and reuse through fish culture. There are systems fed with conventional wastewater or sewage in three districts, the largest being at the Kumudini Hospital Complex (KHC), Mirzapur, Tangail district.

Source: UNEP webpage; http://www.unep.or.jp/ietc/publications/techpublications/techpub 15/2-9/9-3-3.asp

Figure A7.3- 13 A duckweed based wastewater treatment system at Mirzapur, Bangladesh

The development of the first duckweed, conventional wastewater treatment system began, in 1989 at the KHC in Mirzapur. The facility consists of one duckweed covered, 0.7 ha plug flow lagoon constructed as a 500 m long serpentine channel with seven bends. It is fed with a mixture of hospital, school and domestic wastewater from some 2,350 people (per capita estimated sewage generation 100 l/day). The plug flow wastewater-fed duckweed pond is preceded by a 0.2 ha anaerobic pond with a hydraulic retention time (HRT) of 2-4 days. HRT in the plug flow pond is estimated at 21-23 days. Duckweed harvested from the 0.7 ha wastewater treatment pond is fed daily to three adjacent fish ponds, each 0.2 ha.

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The wastewater treatment system produces from 220 to 400 tonnes fresh weight duckweed/ha/year (about 17 to 31 tonnes dry weight/ha/year). The fish ponds are stocked with a polyculture of Indian major carps (rohu, mrigal and catla), Chinese carps (grass carp and silver carp), and common carp. Tilapia is not stocked, but fingerlings enter the ponds incidentally. Fish production varies from 10 to 15 tonnes/ha/year, about 40% of which is tilapia. Fish yields are relatively high because of frequent harvesting and addition of other feed besides duckweed such as oil cake and rice bran.

Over the last two years the wastewater-fed duckweed-fish system has generated a net profit of almost US$3,000/ha/year. This is about three times that of the major agricultural crop of the area, rice.

7 GROUNDWATER RECHARGE

7.1 SOIL AQUIFER TREATMENT

7.1.1 DESCRIPTION

Soil aquifer treatment (SAT) makes use of the natural chemical and biological processes within soil (unsaturated zone) to “polish” treated sewage. Soil aquifer treatment is most commonly used to remove residual organic material, nitrogen and pathogenic microorganisms.

The most common SAT solution is infiltration ponds. After conventional sewage treatment, the water is discharged into infiltration ponds and then reused via recovery wells. Infiltration ponds are used in cycles: Sewage is treated during wet cycle; sludge is dried and solids are removed during dry cycle.

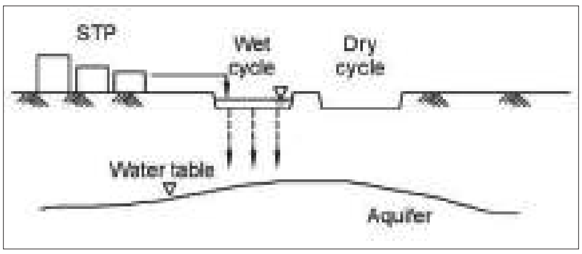

Figure A7.3- 14 Soil aquifer treatment

7.1.2 APPLICATION EXAMPLES

a. Sweetwater SAT Treatment Plant in Tucson, Arizona, USA

It was built in year 1989, and consists of 8 recharge basins and 2 wetland basins. The total size is 17 ha, the design percolation rate is 70 cm/day and the amount of water reclaimed per year is 1,233.5 m3 per year. The area is used also for wildlife protection.

Figure A7.3- 15 Sweetwater SAT treatment plant, Tucson, AZ

b. Rio Hondo Spreading Grounds, Montebello Forebay, California, USA

Table A7.3- 11 Site characteristics

Rio Hondo	
Year when initially started using	1937/38
Size (ha)	230
# of Sub-Basins	20
Percolation Rate (m^3/sec)	11
Water Storage (103 m^3)	4,557

Source: http://www.wrd.org/engineering/groundwater-replenishment-spreading-grounds.php [Accessed 20 Oct. 2011]

Figure A7.3- 16 Rio Hondo Spreading Grounds, Montebello Forebay, CA

c. Phoenix, Arizona, USA

d. Orange County, California, USA

e. Israel

7.1.3 ADVANTAGES AND DISADVANTAGES

The advantages and disadvantages of this process are as follows:

a. Advantages

i. No requirement of energy and mechanical equipment for aeration
ii. Natural process and virtually maintenance free
iii. System can provide renewed water sources
iv. It protects against sea water intrusion, when practiced in coastal areas.
v. It helps in the protection of wildlife

b. Disadvantages

i. Large areas of land are used
ii. Odours, mosquitoes and flies can be a problem if they are not properly controlled

7.1.4 TYPICAL DESIGN PARAMETERS

Table A7.3- 12 Typical design parameters

Parameter	Value
Sewage loading rate	<15cm/d
Total nitrogen loadings	56 to 67 kg/ha/d
BOD to Nitrogen ratio	>3:1
BOD removal	around 90%
SS in the percolate	1 to 2 mg/L

Source: Sherwood C. Reed, Ronald W. Crites, E. Joe Middlebrooks, 2006

Table A7.3- 13 Hydraulic loading cycles for soil aquifer treatment

Loading Objective	Wastewater Applied	Season	Application Period (d)	Drying/Resting Period (d)
Maximize infiltration rates	Primary	Summer	1-2	6-7
		Winter	1-2	7-12
	Secondary	Summer	2-3	5-7
		Winter	1-3	6-10
Maximize nitrification	Primary	Summer	1-2	6-9
		Winter	1-2	7-13
	Secondary	Summer	2-3	5-6
		Winter	1-3	6-10
Maximize nitrogen removal	Primary	Summer	1-2	10-13
		Winter	1-2	13-20
	Secondary.	Summer	6-7	9-12
		Winter	9-12	12-16

Source: Adapted from Crites, R.W. and Tchobanoglous, G., Small and Decentralized Wastewater Management Systems, McGraw-Hill. New York. 1998.

Important considerations related to design and maintenance of this process is as follows

A. The minimum pre-treatment is secondary or the equivalent. For small systems, a short detention-time pond is recommended. If facultative or stabilization ponds are to be used, it is recommended that a constructed wetland system be used between the pond and the infiltration point to reduce TSS levels.

B. High loadings of BOD and TSS from industrial activity will require careful management to avoid odour production and to avoid plugging of the soil.

7.1.5 APPLICABILITY

Soil aquifer treatment is an effective process for polishing BOD, TSS, and pathogen removal. Removal of phosphorus and metals depends on travel distance and soil texture. Nitrogen removal can be significant when systems are managed for that objective: ammonia is retained in the soil long enough for biological conversion through ammonia adsorption and the absence of available oxygen and the existence of carbon sources facilitates denitrification. Pathogens are filtered out by the soil and adsorbed onto clay particles and organic matter.

7.1.6 GUIDING PRINCIPLES

A. This technology is used as a method of conserving sewage without wasting it.

B. This technology can be considered where area is available.

C. In coastal cities where ground water is used, this can be a long term solution against sea water intrusion. This requires a separate study needs to be carried out at each location before its application in the field.

7.2 ORLANDO AND ORANGE COUNTY, FLORIDA, USA

The City of Orlando and Orange County, Florida, were mandated to cease discharge of effluent into Shingle Creek, which flows into Lake Tohopekaliga. To overcome this issue, a reuse project was constructed in West Orange and Southeast Lake counties along a high, dry, and sandy area known as the Lake Wales Ridge, called Water Conserv II. Water Conserv II is the largest reuse project of its type in the world, a combination of agricultural irrigation and rapid infiltration basins (RIBs). It is also the first reuse project in Florida to irrigate crops produced for human consumption with reclaimed water. Water Conserv II began operation on December 1, 1986. The project is designed for average flows of 50 mgd (2,190 l/s) and can handle peak flows of 75 mgd (3,285 l/s). Approximately 60% of the daily flows are used for irrigation, and the remaining ± 40% is discharged to the RIBs for recharge of the Floridan aquifer.

Citrus growers participating in Water Conserv II benefit from using reclaimed water. Citrus produced for fresh fruit or processing can be irrigated by using a direct contact method. Growers are provided reclaimed water 24 hours per day, 7 days per week at pressures suitable for micro-sprinkler or impact sprinkler irrigation. By providing reclaimed water at pressures suitable for irrigation, costs for the installation, operation, and maintenance of a pumping system can be eliminated.

Water Conserv II: City of Orlando and Orange County, Florida, US

As a result of a court decision in 1979, the City of Orlando and Orange County, Florida, were mandated to cease discharge of their effluent into Shingle Creek, which flows into Lake Tohopekaliga, by March 1988. To overcome this issue, the decision was made to construct a reuse project in West Orange and Southeast Lake counties along a high, dry, and sandy area known as the Lake Wales Ridge, and the project was named Water Conserv II. The primary use of the reclaimed water would be for agricultural irrigation. Daily flows not needed for irrigation would be distributed into rapid infiltration basins (RIBs) for recharge of the Floridan aquifer.

Water Conserv II is the largest reuse project of its type in the world, a combination of agricultural irrigation and RIBs. It is also the first reuse project in Florida permitted by the Florida Department of Environmental Protection to irrigate crops produced for human consumption with reclaimed water. The project is best described as “a cooperative reuse project by the City of Orlando, Orange County, and the agricultural community.”

The project is designed for average flows of 50 mgd (2,190 l/s) and can handle peak flows of 75 mgd (3,285 l/s). Approximately 60% of the daily flows are used for irrigation, and the remaining ±40% is discharged to the RIBs for recharge of the Floridan aquifer. Water Conserv II began operation on December 1, 1986.

At first, citrus growers were reluctant to sign up for reclaimed water being afraid of potential damage to their crops and land from the use of the reclaimed water. Study was carried out on the use of reclaimed water as an irrigation source for citrus. Later on, the Mid Florida Citrus Foundation (MFCF), a non-profit organization was created for conducting research on citrus and deciduous fruit and nut crops. Goals of the MFCF are to develop management practices that will allow growers in the northern citrus area to re-establish citrus and grow it profitably, provide a safe and clean environment, find solutions to challenges facing citrus growers, and promote urban and rural cooperation. All research conducted by the MFCF is located within the Water Conserv II service area. Reclaimed water is used on 163 of the 168 acres of research. MFCF research work began in 1987.

Research results to date have been positive. The benefits of irrigating with reclaimed water have been consistently demonstrated through research since 1987. Citrus on ridge (sandy, well drained) soils respond well to irrigation with reclaimed water. No significant problems have resulted from the use of reclaimed water. Tree condition and size, crop size, and soil and leaf mineral aspects of citrus trees irrigated with reclaimed water are typically as good as, if not better than, groves irrigated with well water. Fruit quality from groves irrigated with reclaimed water was similar to groves irrigated with well water. The levels of boron and phosphorous required in the soil for good citrus production are present in adequate amounts in reclaimed water. Thus, boron and phosphorous can be eliminated from the fertilizer program. Reclaimed water maintains soil pH within the recommended range; therefore, lime no longer needs to be applied.

Citrus growers participating in Water Conserv II benefit from using reclaimed water. Citrus produced for fresh fruit or processing can be irrigated by using a direct contact method. Growers are provided reclaimed water 24 hours per day, 7 days per week at pressures suitable for micro-sprinkler or impact sprinkler irrigation. At present, local water management districts have issued no restrictions for the use of reclaimed water for irrigation of citrus. By providing reclaimed water at pressures suitable for irrigation, costs for the installation, operation, and maintenance of a pumping system can be eliminated. This means a savings of $317 per hectare per year. Citrus growers have also realized increased crop yields of 10 to 30% and increased tree growth of up to 400%.

The increase is not due to the reclaimed water itself, but the availability of the water in the soil for the tree to absorb. Growers are maintaining higher soil moisture levels.

Citrus growers also benefit from enhanced freeze protection capabilities. The project is able to supply enough water to each grower to protect his or her entire production area. Freeze flows are more than 8 times higher than normal daily flows. It is very costly to the City and County to provide these flows (operating costs average $15,000 to $20,000 per night of operation), but they feel it is well worth the cost. If growers were to be frozen out, the project would lose its customer base. Sources of water to meet freeze flow demands include normal daily flows of 30 to 35 MGD (1,310 to 1,530 l/s), 38 million gallons of stored water (143,850 m^3), 80 MGD (3,500 l/s) from twenty-five 16-inch diameter wells, and, if needed, 20 MGD (880 l/s) of potable water from the Orlando Utilities Commission.

Water Conserv II is a success story. University of Florida researchers and extension personnel are delighted with research results to date. Citrus growers sing the praises of reclaimed water irrigation. The Floridan aquifer is being protected and recharged. Area residents view the project as a friendly neighbour and protector of the rural country atmosphere (USEPA, 2004).

7.3 PHOENIX (ARIZONA), USA

Water reclamation and reuse have become an important part of Phoenix Water Services Department's operational strategy. In 2001, Cave Creek Reclaimed Water Reclamation Plant (CCWRP), in northeast Phoenix, began operation which uses an activated sludge nitrification / denitrification process along with filtration and ultraviolet light disinfection to produce a tertiary-grade effluent that meets the Arizona standards. CCWRP is currently able to treat 8 MGD (350 l/s) and has an expansion capacity of 32 MGD (1,400 l/s). The Phoenix reclamation plant delivers reclaimed water through a nonpotable distribution system to golf courses, parks, schools, and cemeteries for irrigation purposes. The reclaimed water is sold to customers at 80% of the potable water rate.

CCWRP's sister facility, North Gateway Water Reclamation Plant (NGWRP), will serve the northwest portion of Phoenix. The design phase has been completed. The NGWRP will have an initial treatment capacity of 4 MGD (175 l/s) with an ultimate capacity of 32 MGD (1,400 l/s). The plant is modelled after the Cave Creek facility using the "don't see it, don't hear it, don't smell it" design mantra. Construction will be performed using the construction manager-at-risk delivery method.

Groundwater recharge and recovery is a key component of the water reuse program. Phoenix is currently exploring the use of vadose zone wells because they do not require much space and are relatively inexpensive to construct. This method also provides additional treatment to the water as it percolates into the aquifer. A vadose zone recharge facility along with a recovery well is being designed for the CCWRP site.

Water Reclamation and Reuse: Integrated Approach to Wastewater Treatment and Water Resources Issues in Phoenix, Arizona

The rapidly developing area of North Phoenix is placing ever-increasing demands on the city's existing wastewater collection system, wastewater treatment plants, and potable water resources.

As an integrated solution to these issues, water reclamation and reuse have become an important part of Phoenix Water Services Department's operational strategy.

Cave Creek Reclaimed Water Reclamation Plant (CCWRP), in northeast Phoenix, began operation in September 2001. The facility uses an activated sludge nitrification/denitrification process along with filtration and ultraviolet light disinfection to produce a tertiary-grade effluent that meets the Arizona standards. CCWRP is currently able to treat 8 MGD (350 l/s) and has an expansion capacity of 32 MGD (1,400 l/s). The Phoenix reclamation plant delivers reclaimed water through a non potable distribution system to golf courses, parks, schools, and cemeteries for irrigation purposes. The reclaimed water is sold to customers at 80% of the potable water rate.

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Phoenix is using geographic information system (GIS) technology to develop master plans for the build out of the reclaimed water distribution system for both the Cave Creek and North Gateway reclamation plants. Through GIS, potential reclaimed water customers are easily identified. GIS also provides information useful for determining pipe routing, reservoir, and pump station locations. The goal is to interconnect the 2 facilities, thus building more reliability and flexibility into the system. The GIS model is dynamically linked to the water system, planning, and other important databases so that geospatial information is constantly kept up to date. A hydraulic model is being used in conjunction with the GIS model to optimize system operation.

Irrigation demand in Phoenix varies dramatically with the seasons, so groundwater recharge and recovery is a key component of the water reuse program. Phoenix is currently exploring the use of vadose zone wells because they do not require much space and are relatively inexpensive to construct. This method also provides additional treatment to the water as it percolates into the aquifer.

A pilot vadose zone well facility has been constructed at the NGWRP site to determine the efficacy of this technology. A vadose zone recharge facility along with a recovery well is being designed for the CCWRP site.

Nonpotable reuse and groundwater recharge with high quality effluent play an important role in the City's water resources and operating strategies. The North Phoenix Reclaimed Water System integrates multiple objectives, such as minimizing the impact of development in the existing wastewater infrastructure by treating wastewater locally and providing a new water resource in a desert environment. By using GIS, Phoenix will be able to plan the build out of the reclaimed water system to maximize its efficiency and minimize costs (USEPA, 2004).

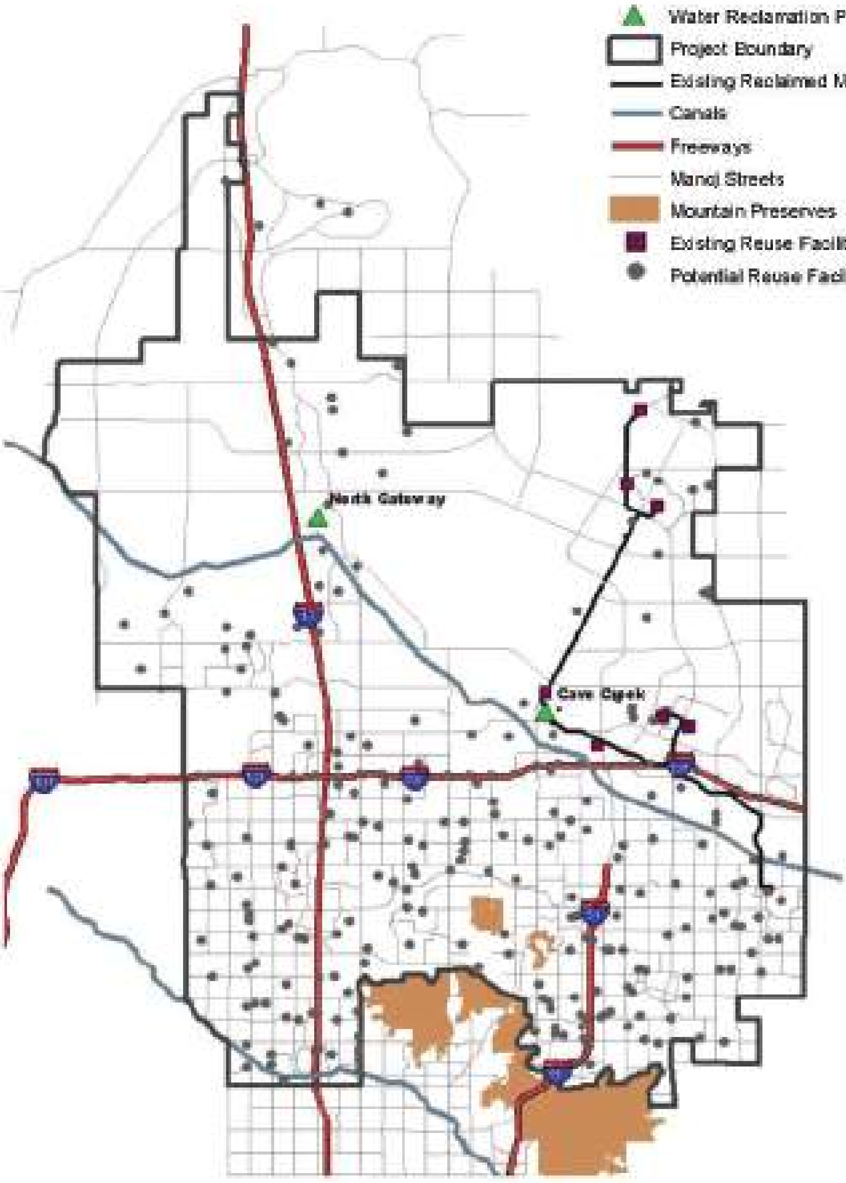

Figure A7.3- 17 North Phoenix reclaimed water system

7.4 SANTA ROSA (CALIFORNIA) RECHARGE PROJECT, USA

Since early 1960s, the Santa Rosa Subregional Water Reclamation System has provided reclaimed water for agricultural irrigation in the Santa Rosa Plain, and later on for irrigating golf courses, and local parks. The remaining reclaimed water not used for irrigation was discharged to the Laguna de Santa Rosa. Growing concerns over water quality impacts in the Laguna de Santa Rosa pressured the system to search for a new and reliable means of reuse.

In the northwest quadrant of Sonoma County lies the Geysers Geothermal Steamfield, a super-heated steam resource used to generate electricity. Due to overexploitation, the electricity generation has declined from 2,000 MW (1987) to about 1,200 MW. As a result, the operators are seeking a source of water to recharge the deep aquifers that yield steam.

In 1998, the Santa Rosa Sub-regional Reclamation System decided to build a conveyance system to send 11 MGD (480 l/s) of tertiary-treated water from the Laguna treatment plant to the northwest Geysers steam field for recharge and it began operations in in November 2003.

About 100 megawatts of additional power is generated each day by the Geysers Recharge Project. In January 2008, the delivery went up to 12.62 MGD and helped generate enough electricity for 100,000 households in Sonoma and other North Bay counties.

The conveyance system to deliver water to the steamfield includes 40 miles (64 km) of pipeline, 4 large pump stations, and a storage tank located high in the Mayacmas Mountains. The system requires a lift of 1,005 meters. Distribution facilities within the steamfield include another 29km of pipeline, a pump station, and tank, plus conversion of geothermal wells from production wells to injection wells.

Geysers Recharge Project: Santa Rosa, California, USA

The cities of central Sonoma County, California, have been growing rapidly, at the same time regulations governing water reuse and discharge have become more stringent. Since the early 1960s, the Santa Rosa Subregional Water Reclamation System has provided reclaimed water for agricultural irrigation in the Santa Rosa Plain, primarily to forage crops for dairy farms. In the early 1990s, urban irrigation uses were added at Sonoma State University, golf courses, and local parks. The remaining reclaimed water not used for irrigation was discharged to the Laguna de Santa Rosa. But limited storage capacity, conversion of dairy farms to vineyards (decreasing reclaimed water use by over two-thirds), and growing concerns over water quality impacts in the Laguna de Santa Rosa, pressured the system to search for a new and reliable means of reuse.

In the northwest quadrant of Sonoma County lies the Geysers Geothermal Steamfield, a super-heated steam resource used to generate electricity since the mid-1960s. At its peak in 1987, the field produced almost 2,000 megawatts (MW). Geysers operators have mined the underground steam to such a degree over the years that electricity production has declined to about 1,200 MW. As a result, the operators are seeking a source of water to recharge the deep aquifers that yield steam.

Geothermal energy is priced competitively with fossil fuel and hydroelectric sources, and is an important “green” source of electricity. In 1998, the Santa Rosa Sub-regional Reclamation System decided to build a conveyance system to send 11 MGD (480 l/s) of tertiary-treated water from the Laguna treatment plant to the northwest Geysers steamfield for recharge and in November 2003 it began operating. About 100 megawatts of additional power is generated each day by the Geysers Recharge Project. In January 2008, the delivery went up to 12.62 MGD and helps generate enough electricity for 100,000 households in Sonoma and other North Bay counties.

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One of the major benefits of the Geysers Recharge Project is the flexibility afforded by year-round reuse of water. The system has been severely limited because of seasonal discharge constraints and the fact that agricultural reuse is not feasible during the wet winter months. The Geysers steam-field will use reclaimed water in the winter, when no other reuse options are available. However, during summer months, demand for reuse water for irrigation is high. The system will continue to serve agricultural and urban users while maintaining a steady but reduced flow of reclaimed water to the Geysers.

In addition to the benefits of power generation, the Geysers Recharge Project will bring an opportunity for agricultural reuse along the Geysers pipeline alignment, which traverses much of Sonoma County's grape-growing regions (USEPA, 2004).

7.5 UNDP STUDY AT CHENNAI

A study by UNDP in the 1970's identified a sand basin on the coast of Bay of Bengal where secondary treated sewage of the Chennai city can be infiltrated through percolation ponds and extracted for specific industrial use in the nearby petro-chemical complex. However, the public acceptance of this project has not been forthcoming.

8 INDIRECT RECHARGE OF IMPOUNDMENTS

8.1 WINDHOEK (NAMIBIA)

The Windhoek reclamation plant has been in operation since 1968 with an initial production rate of 4800 m^3/d. This operation is the only existing example of direct potable water production. The plant has since been upgraded in stages to its present capacity of 21,000 m^3/d.

The wastewater from residential and commercial settings is treated in the Gammans treatment plants by trickling filters (6,000 m^3/d) and activated sludge (12,000 m3/d), with enhanced phosphorus removal. The effluents from these processes go to 2 separate maturation ponds (4 to 12 days retention). Only the polished effluent from the activated sludge system is directed to the Windhoek reclamation facility added with water from the Goreangab Dam (blending ratio 1:3.5), where it is treated to drinking water standards. After tertiary treatment, reclaimed water is blended again with bulk water from different sources.

Advanced treatment processes (including ozonation and activated carbon) have been added to the initial separation processes of dissolved air flotation, sedimentation, and rapid sand filtration. A chlorine residual of 2 mg/l is provided in distribution systems. Membrane treatment has been considered, as well as additional 140 days storage of the secondary effluent from the maturation ponds in the Goreangab Dam.

Windhoek, the capital of Namibia, has a population of 200,000 and is located in the desert. In 1960, low rainfall (below 300 mm/year) caused the necessary water supply to fall short of the water demand. To meet this need, the country's water supply master plan included the long distance transport of 80% of its water supply from the Eastern National Water Carrier, extensive aquifer withdrawals from around the city, the development of a local surface reservoir, and the construction of a reclamation plant. The Windhoek reclamation plant has been in operation since 1968 with an initial production rate of 4,800 m3/d. This operation is the only existing example of direct potable water production. The plant has since been upgraded in stages to its present capacity of 21,000 m^3/d.

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Risk studies and evaluations of toxicity and carcinogenicity have demonstrated that reclaimed water produced at the Windhoek facility is a safe and acceptable alternative water resource for potable purposes. Treatment capacity at the Windhoek treatment plant is currently being increased to 40,000 m^3/d (USEPA, 2004).

Source: USEPA, 2004

Figure A7.3- 18 Goreangab Dam, adjacent to the Windhoek reclamation plant in Namibia.

8.2 GERMANY

The Berlin water works are located near the surface water system. The water works' wells are drilled mostly at short distance from the rivers and lakes near the bank, from where water is abstracted. One of the largest water works of Berlin (Tegel) abstracts lake water (80%) via bank filtration and artificial groundwater recharge. In Tegel drinking water, advanced treated wastewater portions have been calculated to be 14-28%.

In order to study the properties and consequences of artificial recharge, a research and development project, namely Project Natural and Artificial Systems for Recharge and Infiltration (NASRI), was initiated in 2002. The NASRI project indicates that aquifer recharge offers great potentials in the future of integrated water management, provided that the necessary precautions are taken.

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The climatic water balance is approaching a negative value with transpiration equalling precipitation. Even in the near future an equalised water balance is not expected. Since 1997, part of the advanced treated wastewater is discharged into former drainage ditches to redress the disturbed water balance in the lowland which had resulted in a 0.15 to 0.50 m reduction in the low moors thickness over 50 years. This beneficial reuse has increased the water table, restored the ecology and increased the grass yields from the meadows to 4-5 harvest per year instead of 2.

In order to study the properties and consequences of artificial recharge, a research and development project, namely Project Natural and Artificial Systems for Recharge and Infiltration (NASRI), was initiated in 2002. Their purpose was to study: physical, chemical and biological processes; transport processes; the long-term sustainability of bank filtration; models of transport processes; water managing scenarios; the development of guidelines for the optimised operation of existing bank filtration and groundwater recharge scheme.

The NASRI project provided valuable data concerning artificial aquifer recharge. This project indicates that aquifer recharge offers great potentials in the future of integrated water management, provided that the necessary precautions are taken. It is a process of high value for the future of Berlin's drinking water supply. Mechanisms governing the removal of impurities and chemical reactions were better understood (Mediterranean Wastewater Reuse Working Group, 2007).

APPENDIX A 8.1
DESIGN EXAMPLE OF INTERCEPTOR TANK

The following design criteria are extracted from the publication "The Design of Small Bore Sewer Systems" by the World Bank indexed as TAG 14 available from website *http://www-wds.worldbank.org/* and authorised for public use.

Design of an interceptor tank to pre-treat the sewage from a household of 8 persons at 70 lpcd of sewage and desludging every 3 years

(i) Minimum hydraulic retention time is given by the following equation:

$t_h = 1.5 – 0.3 \log (Pq)$

Where,

t_h = minimum mean hydraulic retention time, days

P = contributing population

q = sewage flow, lpcd

Hence, minimum hydraulic retention time = $1.5 − 0.3 \log (8 \times 70) = 0.68$ day

(ii) Volume required for sedimentation is given by the following equation:

$V_h = 10^{-3} (Pq)$ th

Where,

V_h = volume required for sedimentation, m^3

Hence, volume required for sedimentation = $8 \times 70 \times 0.68 /1000 = 0.39\ m^3$

(iii) Combined solids digestion and storage volume is given by equation:

$V_s = 70 \times PN / 1000$

Where,

V_s = combined solids digestion and storage volume, m^3

N = desired interval between successive desludging operations, years

Hence, solids digestion and storage volume = $70 \times 8 \times 3/1000 = 1.68\ m^3$

Assume a tank of 3 m × 1 m in plan area (A)

(iv) Maximum depth of sludge = Vs/A = 1.68/3 = 0.56 m

(v) Maximum submerged scum depth (d_{ss}) = 0.7/A = 0.7/3 = 0.23 m

(vi) Minimum sludge clear space = 0.82 − 0.26 A

= 0.82 − (0.26 × 3) = 0.04 m

Minimum to be provided is 0.3 m

Minimum height between bottom of scum and invert of the outlet pipe must be 75 mm

Clear height above invert of outlet pipe	= 0.3 + (75/1000) = 0.375 m
Diameter of outlet pipe	= 75 mm = 75 / 1000 = 0.075 m
Effective depth	= 0.56 + 0.375 + 0.23 = 1.165 m
Freeboard	= 0.3 m
Overall inner dimensions	= 3 m × 1 m × 1.5 m height

APPENDIX A 9.1
DESIGN EXAMPLE OF LEACH PIT
(Retained as in 2nd edition)

Design example : Twin Leach Pits (Dry conditions) for 5 users :

1 ASSUMPTIONS

a) litres of wastewater is generated per capita per day

b) litres of water is used per day for floor washing and pan cleaning

c) The water table remains 2 meters or more below ground level through out the year for dry pit and 50 cm below for wet conditions

d) The local soil is porous silty loams and

e) The pits are designed for 2 year sludge accumulation capacity.

2 SOLUTION 1

a) Calculate the total waste water flow (Q) in litres per day

Q = 9.5 l/d × 5 users + 5 litres for floor wash etc.

= 52.5 liters per day

b) Assuming a pit of 800 mm internal diameter (inside lining 75 mm thick with brick on edge and effective depth 800 mm, check for infiltrative surface area (At); this is given by :

At = π d h

Where d is the external diameter and h is the effective depth of the pit.

At = π × 0.95 × 0.8 = 2.39 m^2

c) If the soil is porous silty loams, the infiltrative area required is 52.5/20 = 2.6 m^2 ; hence the infiltrative area provided is insufficient, Therefore by choosing a depth of 0.9 m ; the infiltrative area At will be

= π × 0.95 × 0.9 = 2.69 m^2, which is sufficient.

d) Check for the required solid storage volume (V) for a solids accumulation rate of 0.04 m^3 per capita per year, (Table 9.2) for a dry pit with water being used for anal cleansing and for a dislodging interval of 2 years and a household size of 5 persons

V = 0.04 × 2 × 5 = 0.40 m^3

Whereas, the volume of proposed pit is :

$$\frac{\pi \times 0.8 \times 0.8 \times 0.9}{4} = 0.45\ m^3$$

Hence pit proposed has the sufficient storage capacity.

e) Allowing a free space of say 0.225 m, the dimensions of the pit are as follows:
   Internal diameter 800 mm
   Total depth 1,125 mm (900 mm + 225 mm free board)

Since the pit bottom is more than 2 m above the maximum ground water table, the pit will function in dry condition.

3 SOLUTION 2

The ground water table is 50 cm below the ground surface, but all other assumptions are the same as in the above example.

The pit size is determined by taking the sludge accumulation rate from Table 9.2, Assuming the pit desludging period as 2 years.

Volume of the pit = $0.095 \times 2 \times 5$

= 0.95 m^3

Allowing a free board of 0.225 m. Pit dimensions come as follows :

Internal diameter 1,100 mm

Total depth 1,225 mm (1,000 mm + 225 mm free board)

APPENDIX A 9.2
SOIL PERCOLATION TEST
(Retained as in 2nd edition)

To design a suitable soil absorption system for disposal of effluent from septic tanks, percolation test shall be carried out, on the proposed site for location of the absorption system, in the following manner.

Six or more test holes spaced uniformly over the proposed absorption field shall be made.

A square or circular hole with side width of diameter of 10 cm to 30 cm and vertical sides shall be dug or bored to the depth of the proposed absorption trench. The bottom and sides of the holes shall be carefully scratched with a sharp-pointed instrument to remove any smeared soil surfaces and to provide a natural soil interface into which water may percolate, the holes shall be filled for a depth of 5 cm with loose material to protect the bottom from scouring and settling.

Before the actual readings for percolation tests are taken, it is necessary to ensure that the soil is given ample opportunity to swell and approach the condition it will be in during the wettest season of the year, This is done by pouring water in the hole up to a minimum depth of 30 cm over the grave! and allowed to soak for 24 hours. If the water remains in the test hole after the overnight swelling period, the depth of water shall be adjusted to 15 cm over the gravel. Then from a fixed reference point, the drop in water level shall be noted over a 30 minute period. This drop shall be used to calculate the percolation rate.

If no water remains in the hole, at the end of 30 minute period, water shall be added to bring the depth of the water in hole 15 cm over the gravel. From a fixed reference point, the drop in water level shall be measured at 30 minute intervals for 4 hours, refilling to 15 cm level over the grave! as necessary. The drop that occurs during the final 30 minute period shall be used to calculate the percolation rate. The drop during the earlier periods provide information for the possible modification of the procedure to suit local circumstances.

In sandy soils or other porous soils in which the first 15 cm of water seeps away in less than 30 minutes after overnight swelling period, the time interval between measurements shall be taken as 10 minutes and the test run for one hour. The drop that occurs in the final 10 minutes shall be used to calculate the percolation rate.

Based on the final drop, the percolation rate, which is the time in minutes required for water to fall 1 cm, shall be calculated.

APPENDIX A 9.3
EXAMPLE OF DESIGN OF MINI-PACKAGED TREATMENT PLANT (ON-SITE CONSTRUCTION-TYPE)

The following shows an example of designing an on-site construction-type sewage treatment system - for 500 persons in Japan.

Basic settings

Number of users: 500

Sewage generation rate: 200 L/day

BOD concentration of sewage: 200 mg/L

- Inflow rate of sewage [Q]

Q [m^3/day] = 500 [persons] × 200 [L/person/day]/1,000 = 100 [m^3/day]

First flow equalization tank

Sewage inflow time [T]: 16 hours (per day)

Peak time [Tm]: 2 hours (per day)

Peak factor [Km] (maximum flow rate/mean flow rate): 3.0

Flow adjustment ratio (margin) [Kc]: 1.3

The necessary capacity [V] is given by the following formula:

V [m^3] = [(Km/T) - (Kc/24)] × Tm × Q

= [(3.0/16) - (1.3/24)] × 2 × 100 = 28 m^3

- Contact aeration tank

This tank shall meet all the following requirements:

Requirement 1: The retention time shall exceed 8 hours.

Requirement 2: BOD volume load shall be less than 0.3 [kg/m^3/day].

Requirement 1: V1 [m^3]

V1 [m^3] = Q × (8/24) = 34 [m^3]

Requirement 2: V2 [m^3]

Assuming that the inflow BOD concentration is 200 [mg/L],

V2 [m^3] = (200 × Q/1,000)/0.3 = 67 [m^3]

Accordingly, the capacity meeting both requirements is V [m3] = 67 [m^3].

- Sedimentation tank

Assuming that the retention time [T] is 3 hours,

V [m^3] = Q × (3/24) = 12.5 [m^3]

- Second flow equalization tank

Assuming that the retention time [T] is 2 hours,

V [m^3] = Q × (2/24) = 8.4 [m^3]

- Flocculation tank

Assuming that the retention time [T] is 30 minutes,

V [m^3] = Q × (1/24) × (30/60) = 2.1 [m^3]

- Flocculation sedimentation tank

Assuming that the retention time [T] is 3 hours,

V [m^3] = Q × (3/24) = 12.5 [m^3]

- Disinfection tank

Assuming that the retention time [T] is 15 minutes,

V [m^3] = Q × (1/24) × (15/60) = 1.1 [m^3]

- Sludge storage tank

Assuming that the BOD reduction ratio is 90% (raw sewage: 200 mg/L and treated sewage: 20 mg/L) and 100% of removed contaminants are converted to sludge, the sludge generation (dry substance) rate [Sq(DS)] is given by the following formula:

Sq(DS) [kg-DS/day] = (200 × Q/1,000) × (90/100) × (100/100)
= 18 [kg-DS/day]

Assuming that the sludge concentration is 15,000 mg/L (1.5%),

Sq [m^3/day] = 18 [kg-DS/day]/(1.5/100)/1,000 = 1.2 [m^3/day]

Assuming that the retention period is one week, the capacity of the sludge storage tank [V] is given by the following formula:

V [m^3] = 1.2 [m^3/day] × 7 [day/week] = 8.4 [m^3]

Table A9.3-1 Example of designing a on-site construction-type treatment system

Capacity	500 Persons (100 m^3/day)		
Tank Volume	Tank Name	Effective Volume	Setting Value
	Flow equalization tank (1st)	28 m^3	Peak coefficient:3.0 Inflow time:16hr/24 hr Flow adjustment ratio:1.3 Peak time:2hr/24 hr
	Contact aeration tank	67 m^3	Retention time:8 hr BOD volumetric loading :0.3 kg/m^3/day
	Sedimentation tank	12.5 m^3	Retention time:3 hr
	Flow equalization tank (2nd)	8.4 m^3	Retention time:2 hr
	Flocculation tank	2.1 m^3	Retention time:30 min
	Flocculation sedimentation tank	12.5 m^3	Retention time:3 hr
	Disinfection tank	1.1 m^3	Retention time:15 min
	Sludge storage tank	8.4 m^3	Gross yield coefficient:100% Sludge concentration:15,000 mg/L Storage days:1 week

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