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.

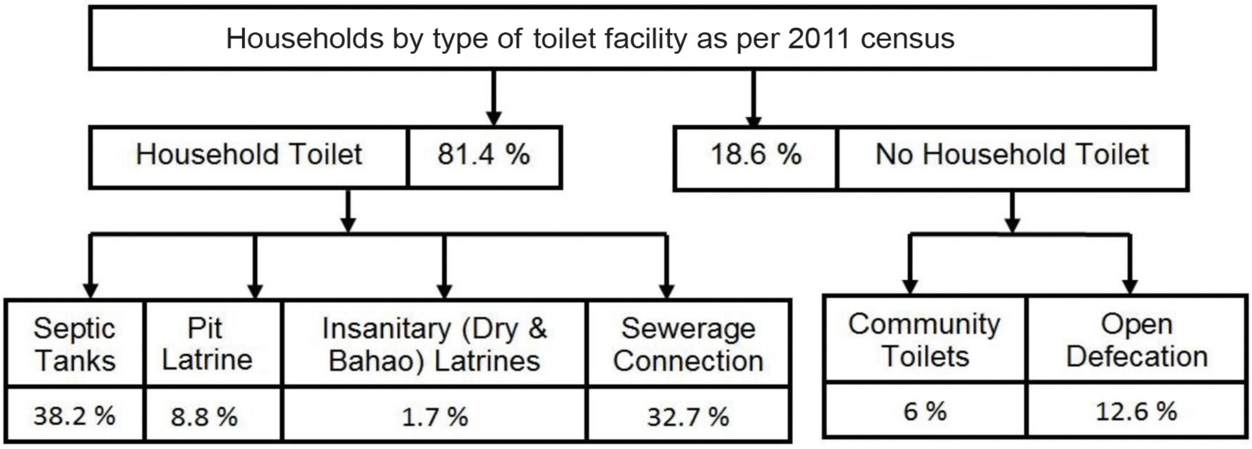


Figure 1.1 Status of Toilets in Urban Households in India

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 .

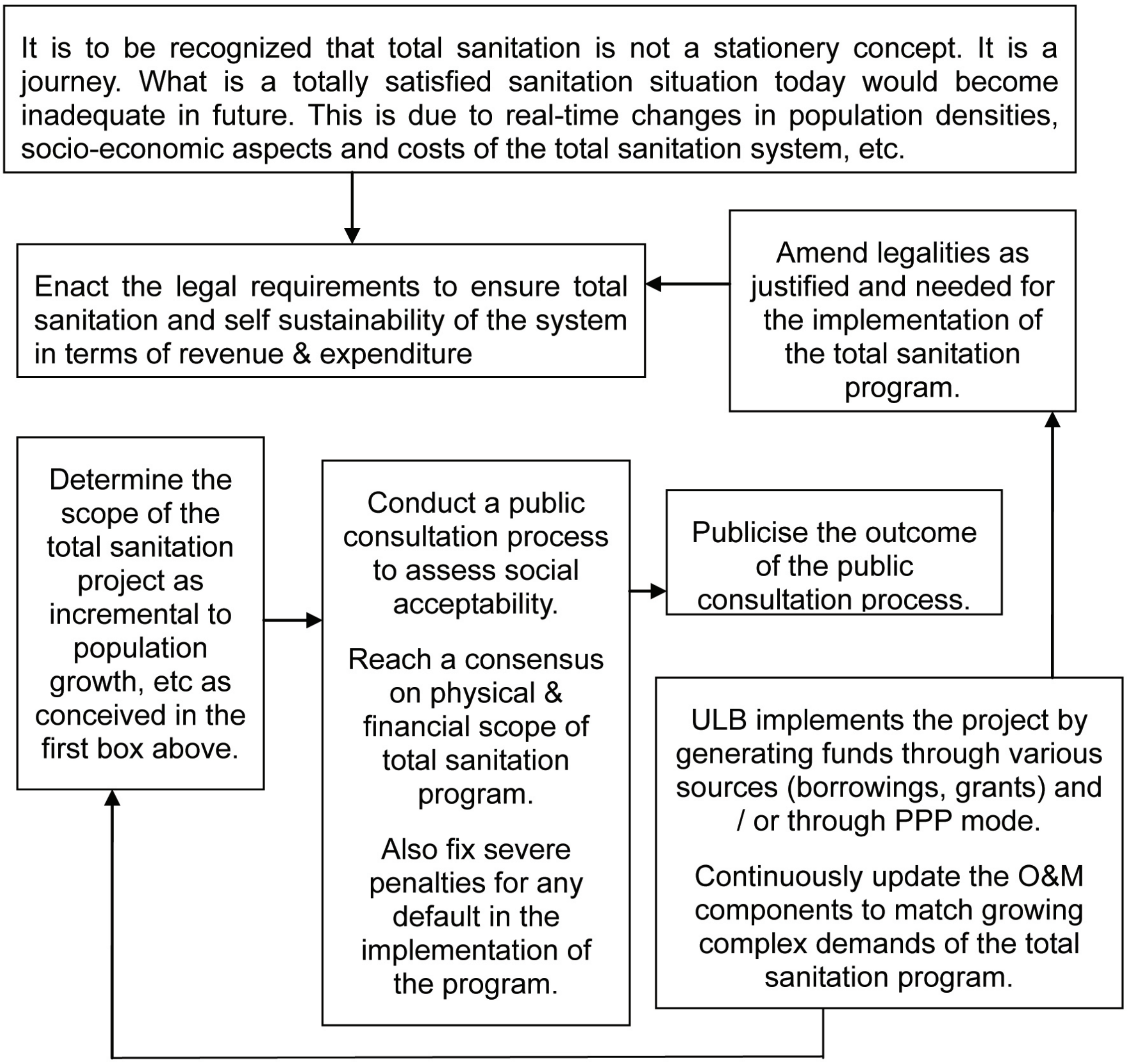


Figure 1.2 Suggested Real Time Total Sanitation Model

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:

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

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) $$i = \frac{a}{\left(t^n\right)} \tag{3.3}$$

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

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

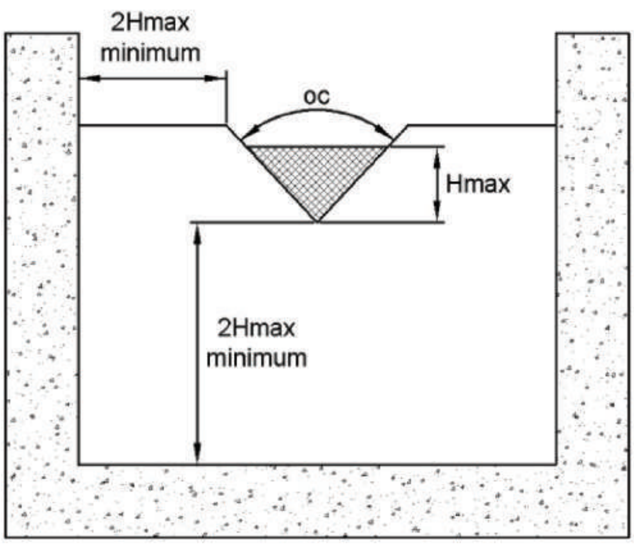


Figure 3.1 Typical mounting of a V Notch in a drain

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× H power 2.5 (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.

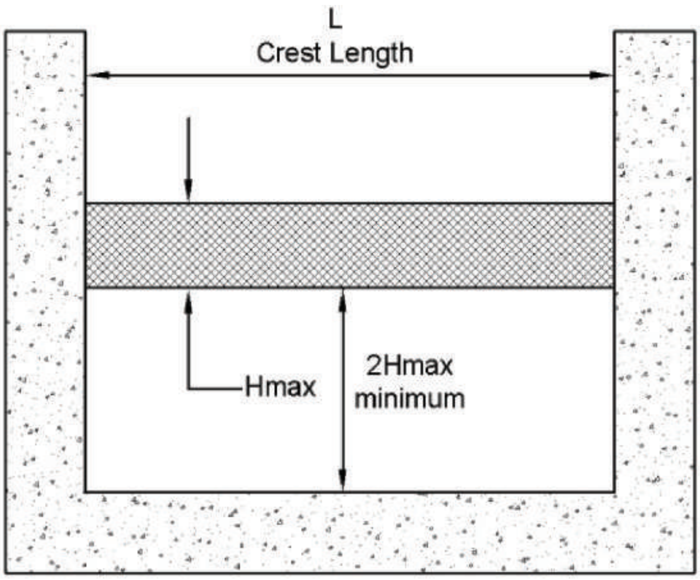


Figure 3.2 Typical mounting of a rectangular weir in a drain

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

Q = 1.85 × L × H power 1.5 (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 × (L − 0.2H) × H power 1.5 (3.8)

Where

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

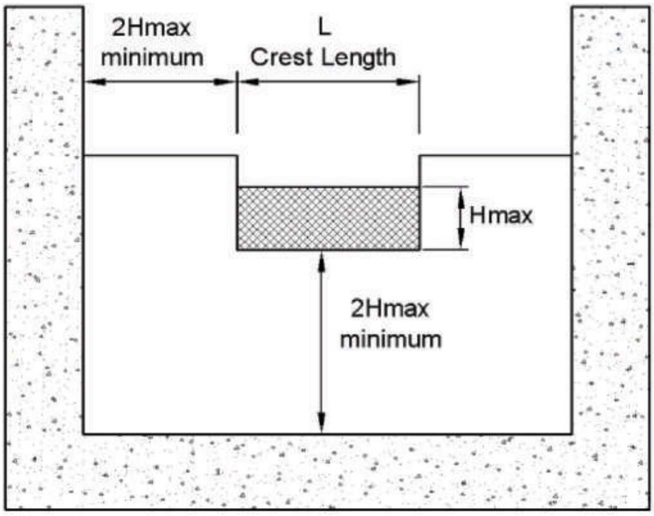


Figure 3.3 Typical mounting of a rectangular weir with end constrictions in a drain

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.

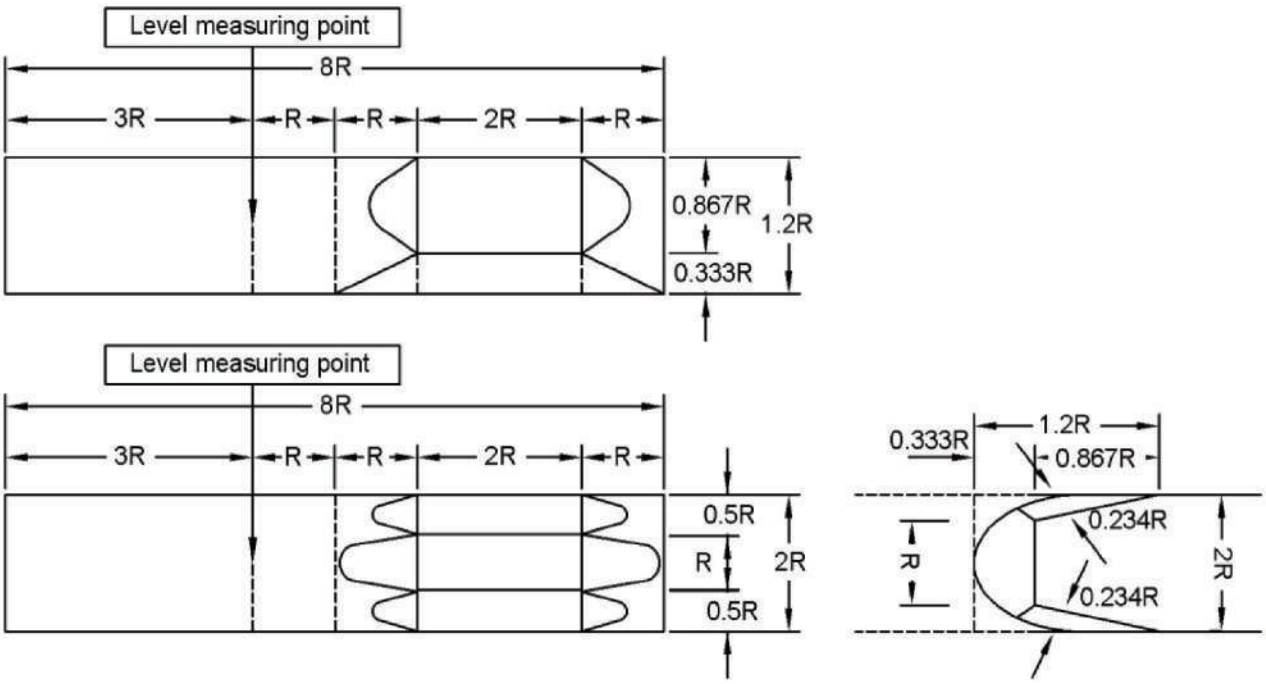


Figure 3.4 Palmer-Bowlus flume installation in drains

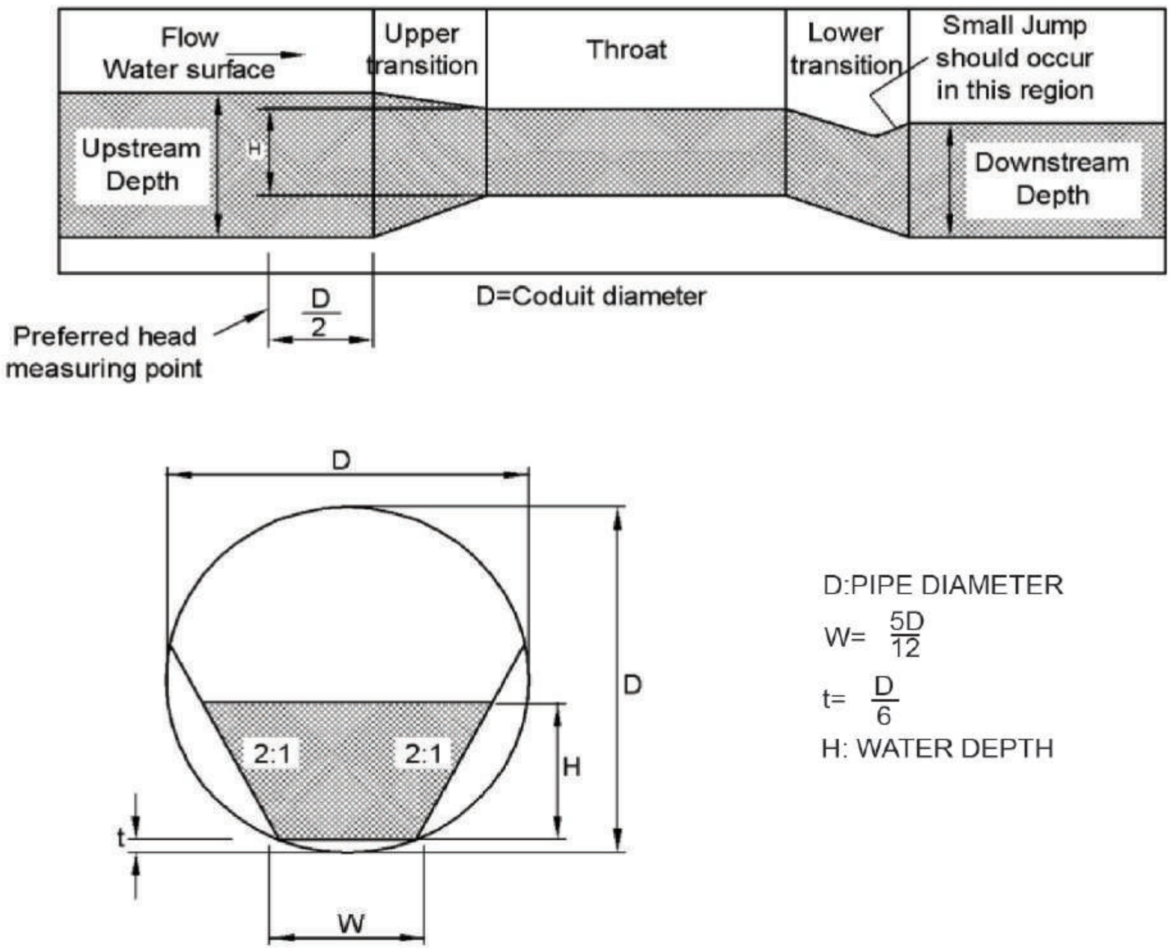


Figure 3.5 Palmer-Bowlus flume installation in circular sewer

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.

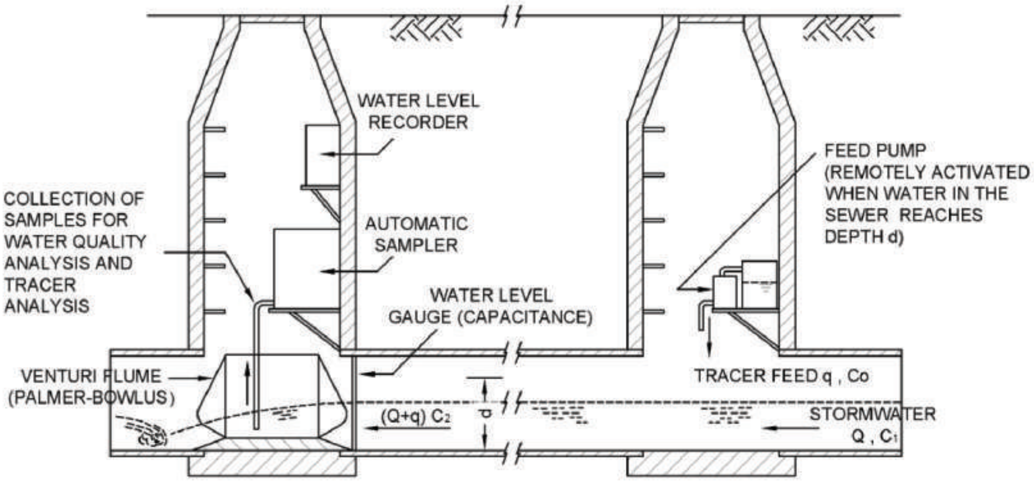
The combination of the Palmer-Bowlus and Tracer dye techniques have been reported as early as 1974 as illustrated in Figure 3.7. It is a system worth inducting in large trunk sewers near the outfalls to have an integrated measurement of the flows and key quality parameters or at least for the flow details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

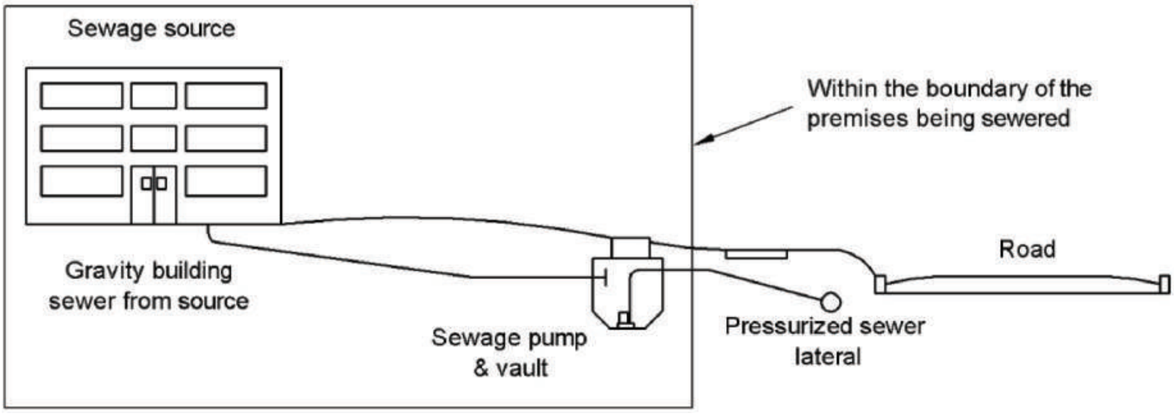
Pressurized sewers are for collecting sewage from multiple sources to deliver to an existing collection sewer, and/or to the STP and are not dependent on gravity and thus topography is not a challenge. Typically, sewage from establishments in the vicinity is collected in a basin fitted with submersible 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.

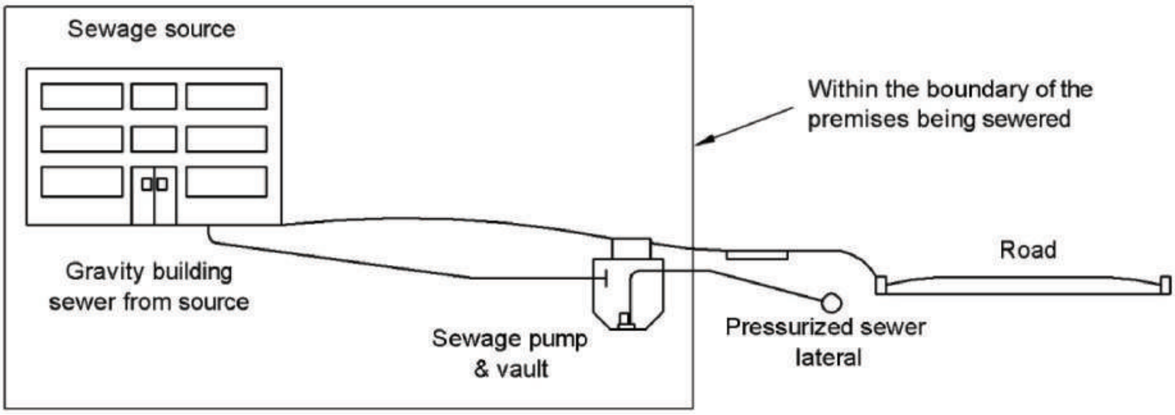


Source: WERF, Fact sheet 2

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.

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

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:

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

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.

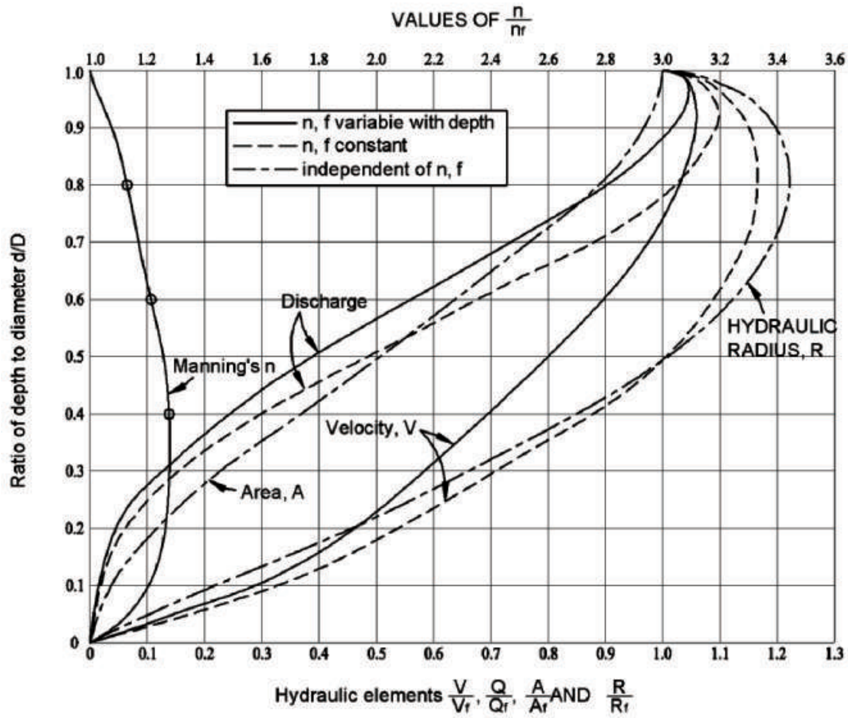
Accordingly, the maximum depth of flow in design shall be limited to 0.8 of the diameter at ultimate peak flow. In order to facilitate the calculations easily, the hydraulic properties at various depths of flow are compiled in Figure 3.12 and Figure 3.13 and Table 3.12.

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

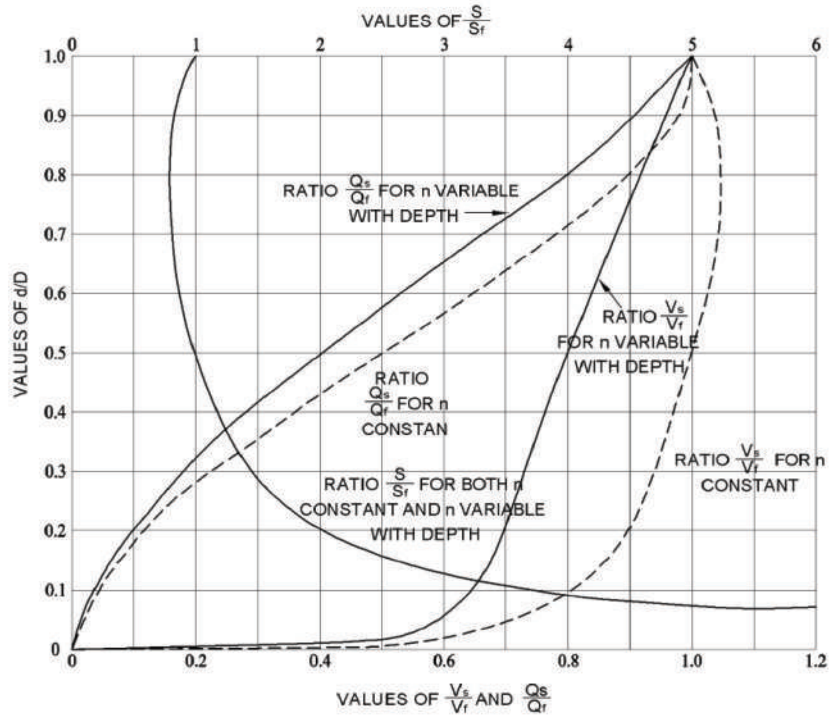
Note: Values of n may be taken as 0.015 for unlined metallic pipes and 0.011 for plastic and other smooth pipes.

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)
V = Velocity at full depth
n = Manning's coefficient at full depth
Q = Discharge at full depth

d = Actual depth of flow
v = Velocity at depth 'd'
nd = Manning's coefficient at depth 'd'
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

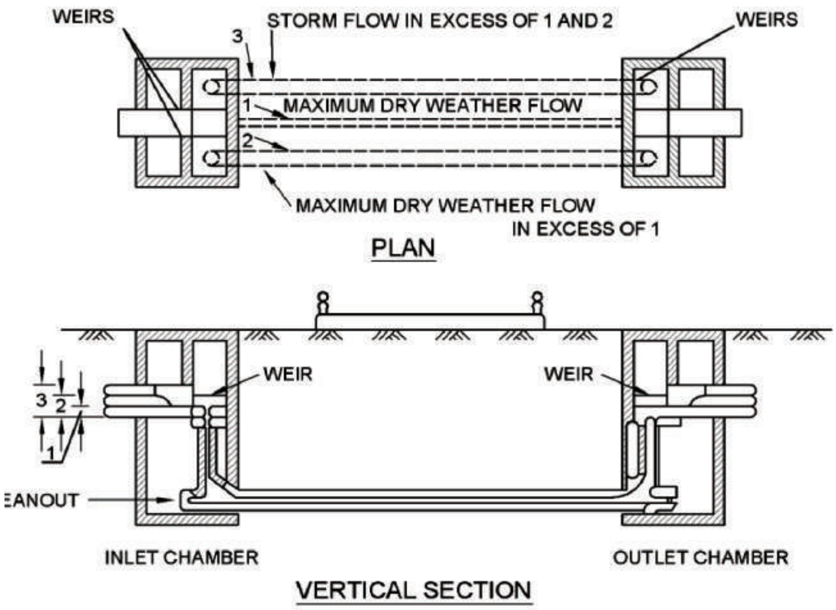
As the inverted siphon is a pipe under pressure, a difference in the water levels at the inlet and outlet is the head under which the siphon operates. This head shall be sufficient to cover the entry, exit and friction losses in pipes. The friction loss through the barrel will be determined by the design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

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

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.

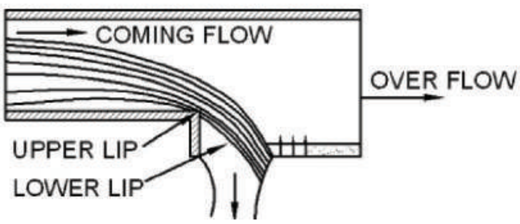
Incoming flows in excess of the desired dry weather flow will "leap" over the weirs on the sides to be diverted to storm drans. This can also be used for a flow equalisation to the STP. When the sewage level drops below the weir lip, submersible pumps can pump back the sewage from the bottom well so 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.

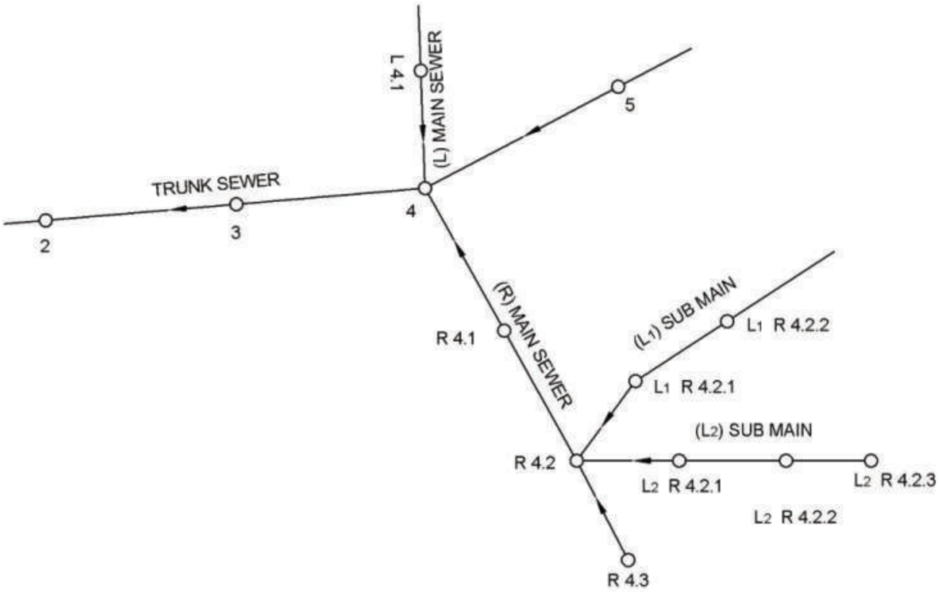
Thus, L_2R4.2.3 (Figure 3-19 overleaf) will pinpoint a particular manhole on the sub main from which the flow reaches manhole number 4 on the trunk sewer through a sub main and a main. The first numeral (from the left) is the number of the manhole on the trunk sewer. The numerals on the right of this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

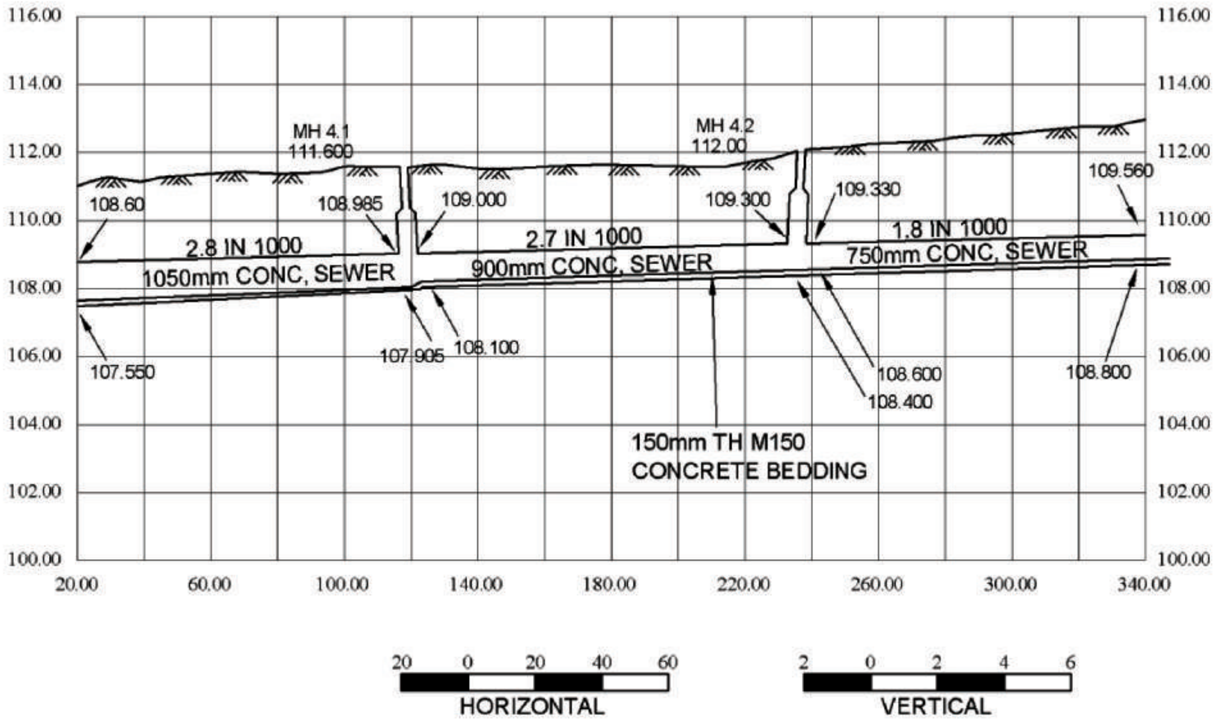
Once the rough sections have been prepared, the designer should review the work for improving the spacing of manholes, the sizes and gradients of the sewers and so forth, economising on materials and excavation to the extent possible. At the same time the designer must ensure that the sewer will serve 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.



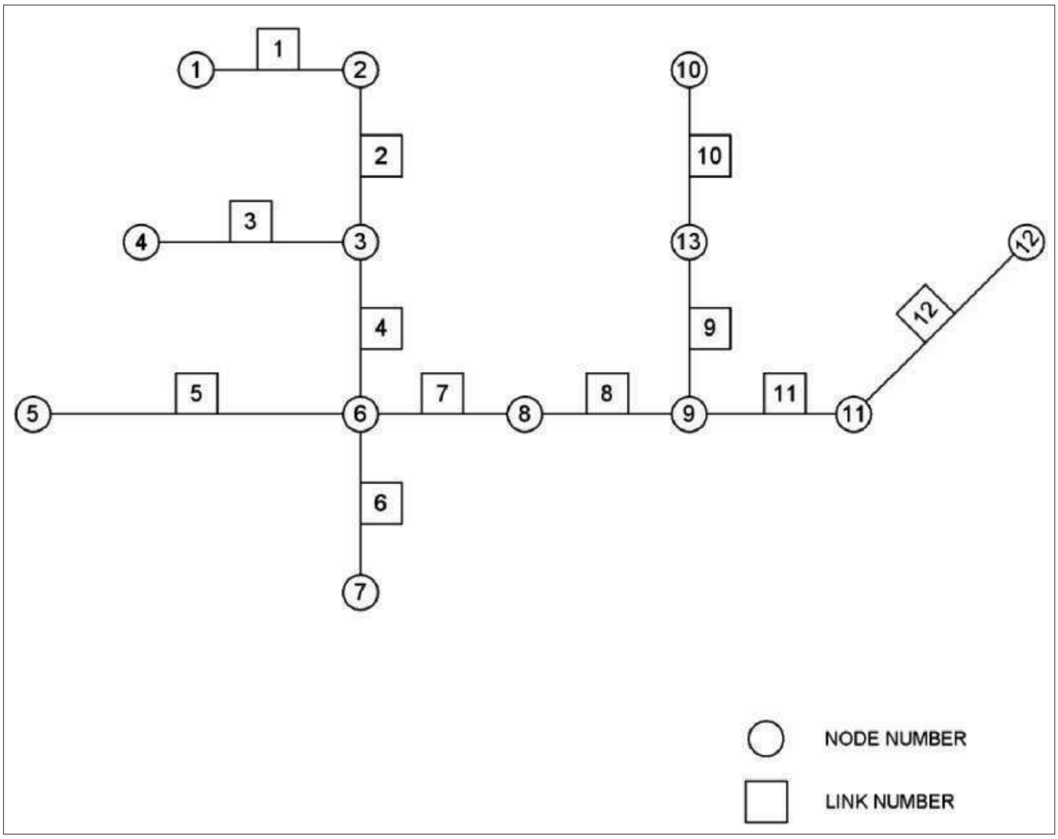
Source: CPHEEO, 1993

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

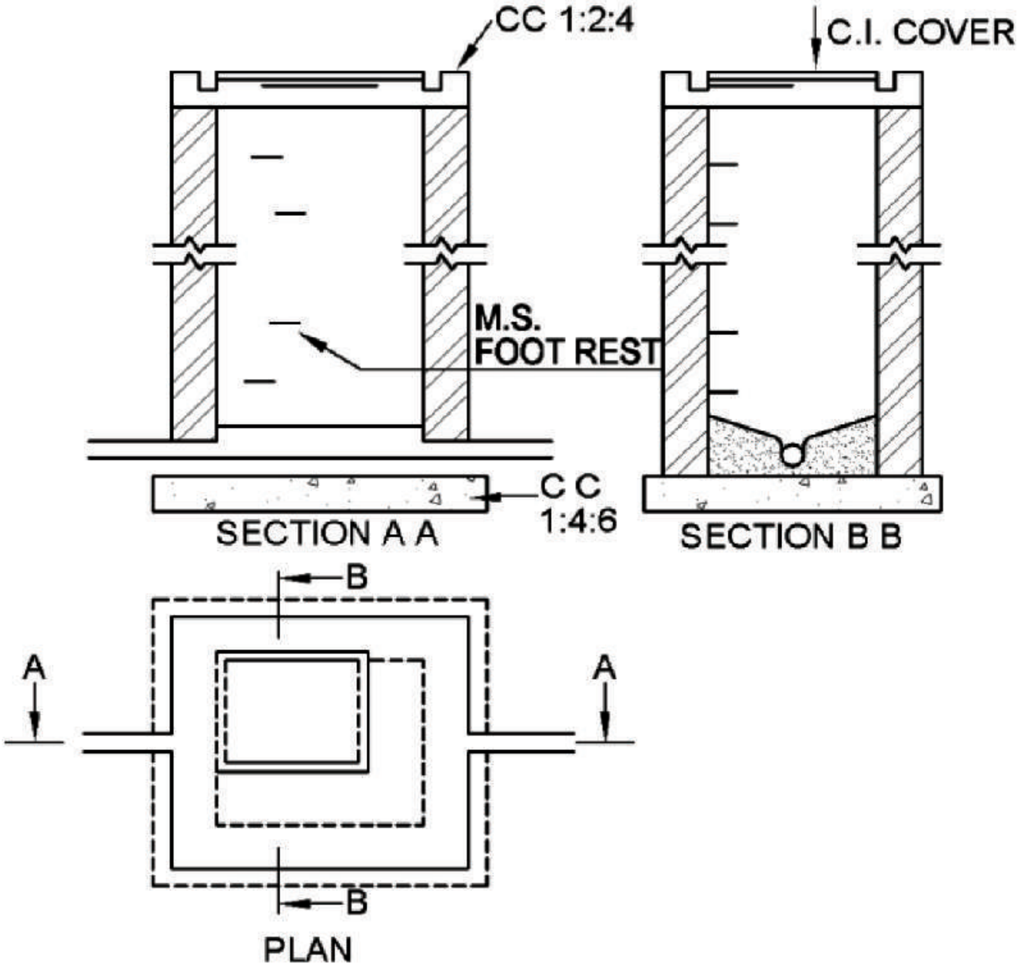
Note:
Where depths exceed 6 m below GL, lift stations as in Section 4.18 shall be inserted and sewage lifted to initial cover depth of 0.9 m.
In specific situations deeper depth can be justified in the DPR like outfall, etc

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

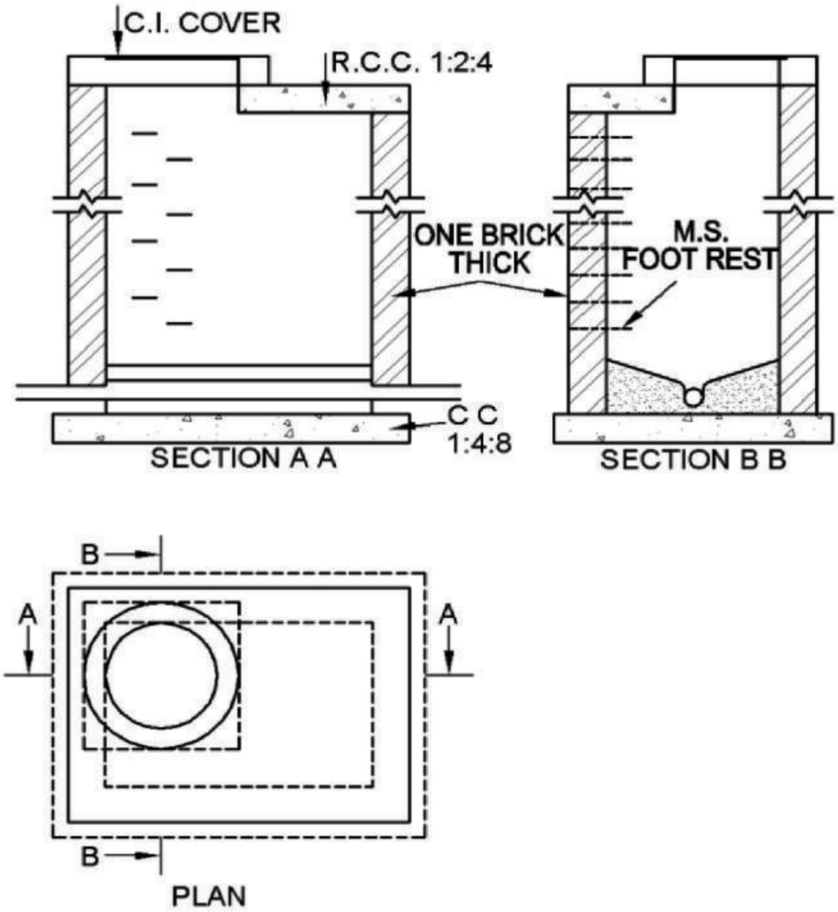
Figure 3.22 Rectangular manhole for 0.9 m × 0.8 m clear in plan and depth less than 0.9 m

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

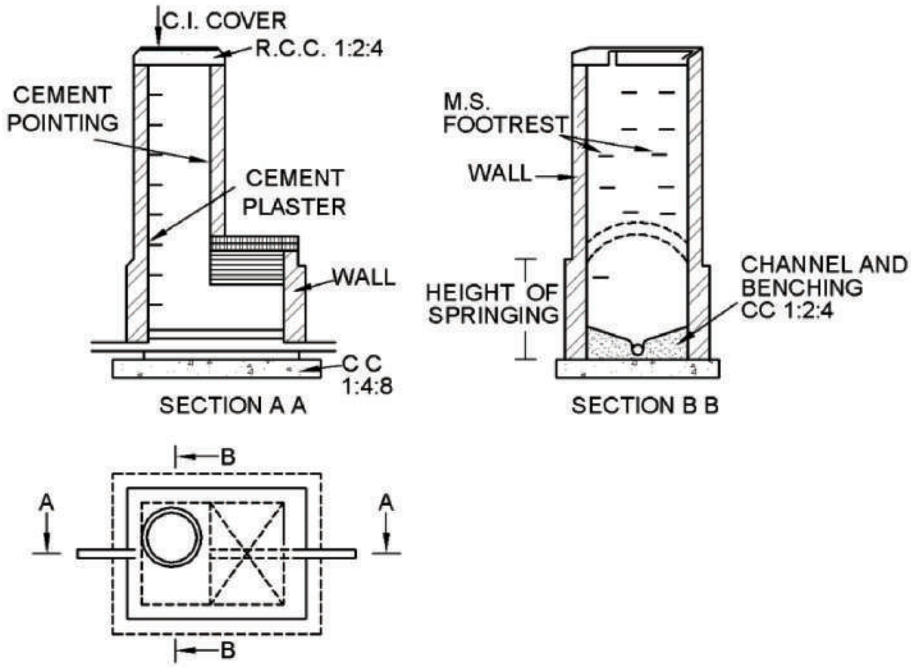
The entries and exits of main sewers as well as house service sewers requires careful detailing because the issue of puncturing the walls for insertions of especially house service sewers later on is impossible. These shall be managed as detailed below.

- 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.



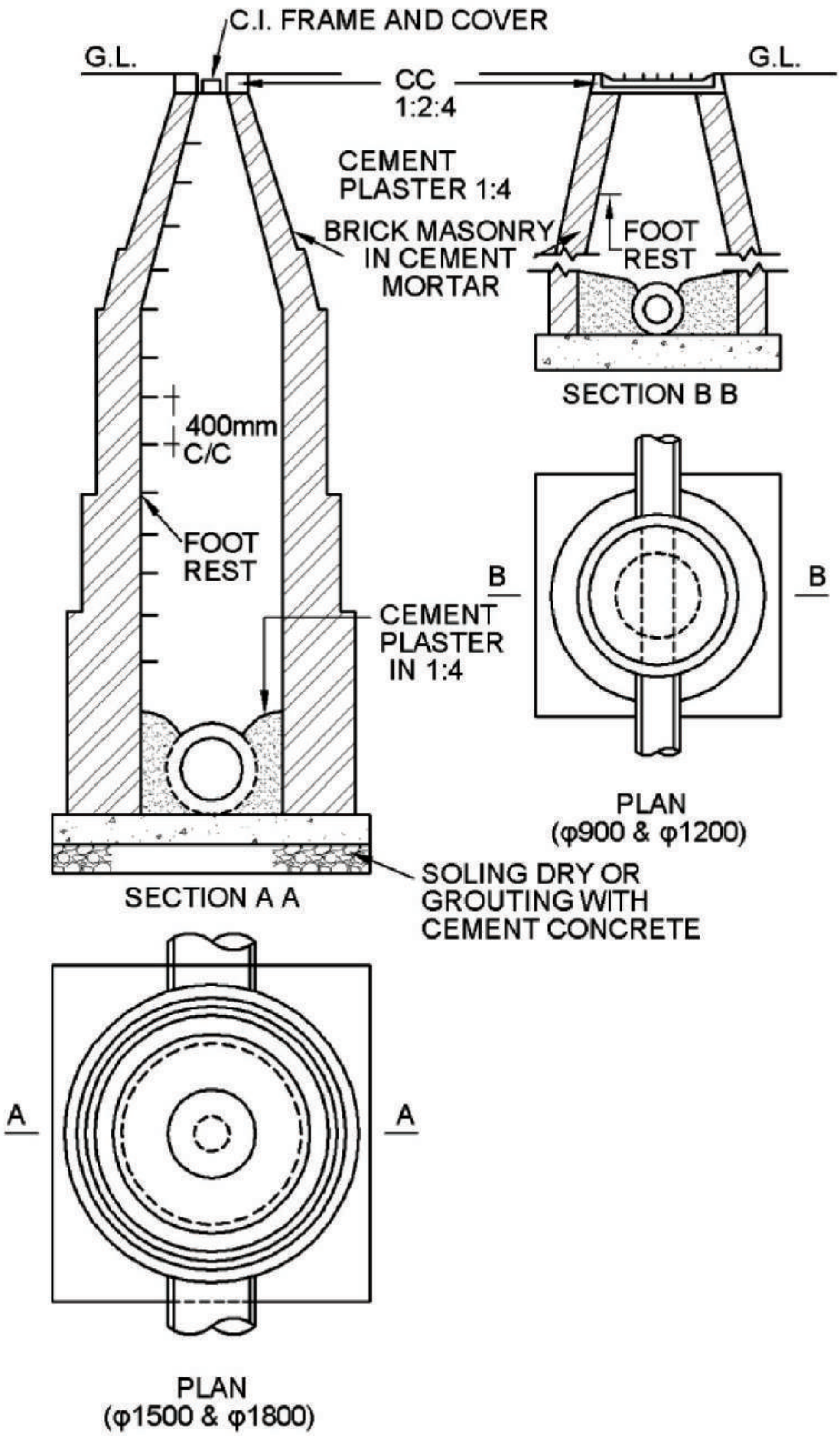
Source: CPHEEO, 1993

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

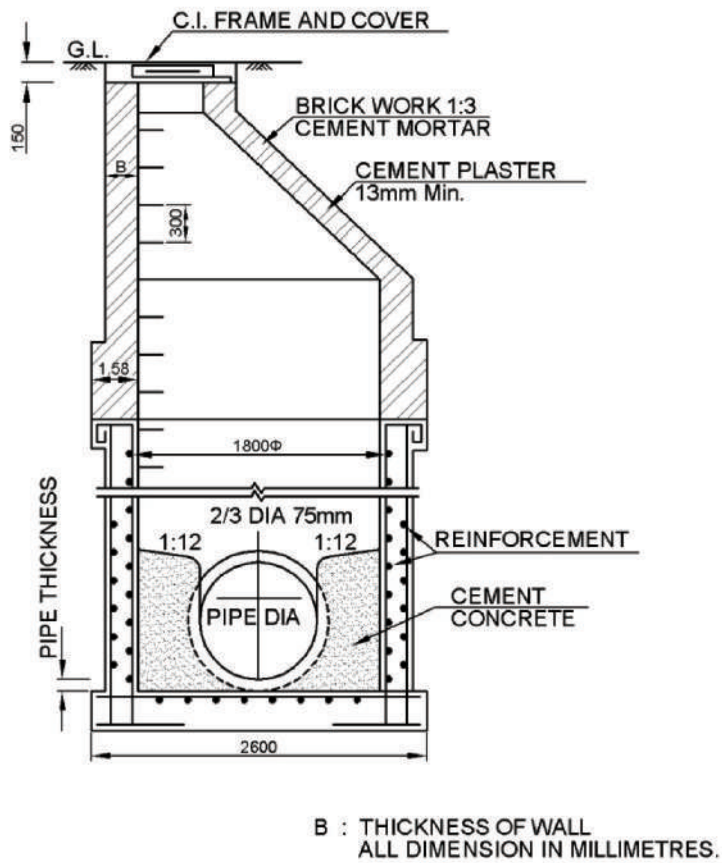
- The varying heights of the manhole are obtained by choosing the bottom ring deeper than the fractional height needed there and filling up the bottom floor after placing the ring such that the invert level of the sewer is obtained thereby.

- 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

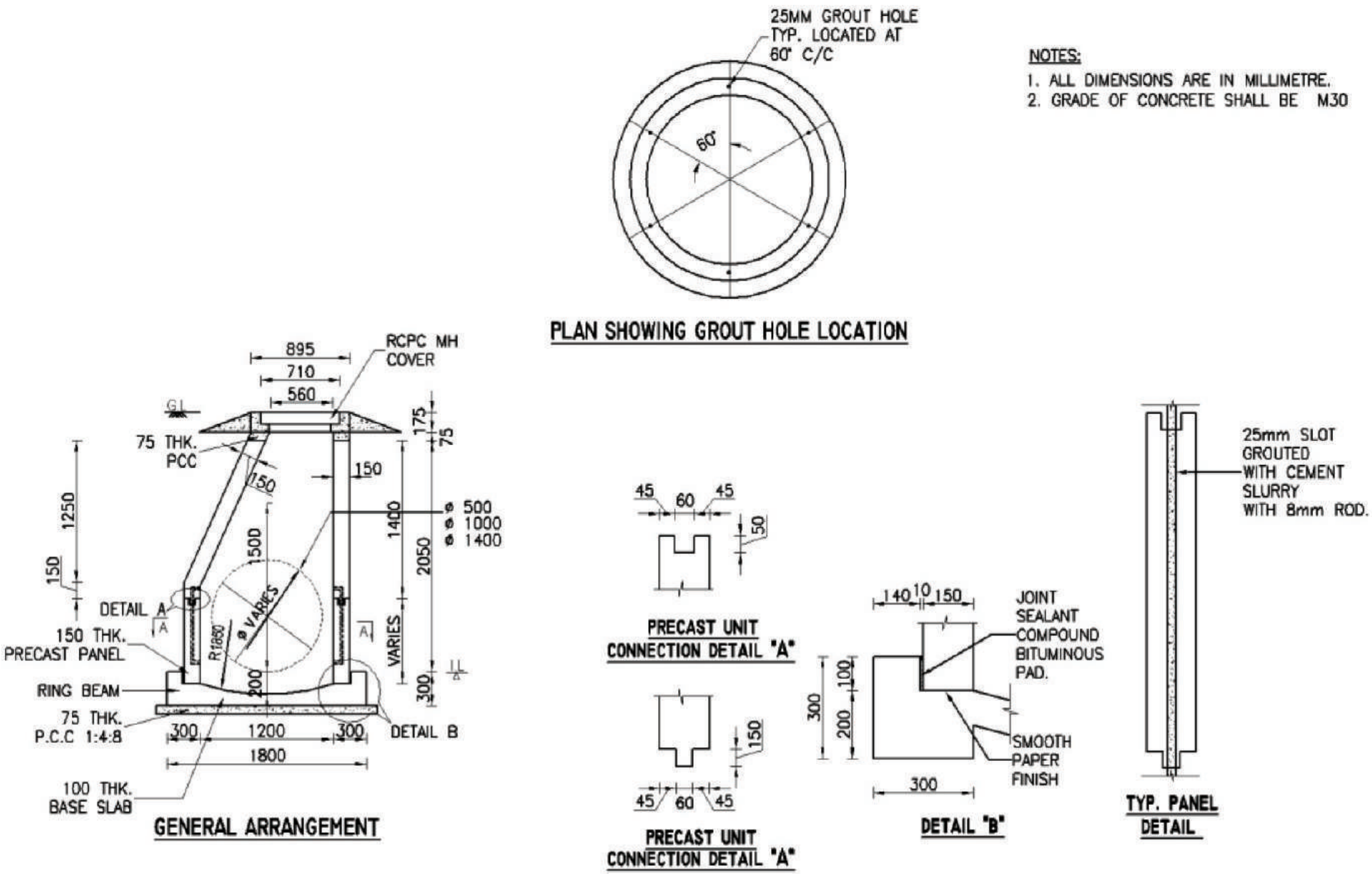


Figure 3.27 RCC Manhole for a depth between 0.6 m to 2.3 m

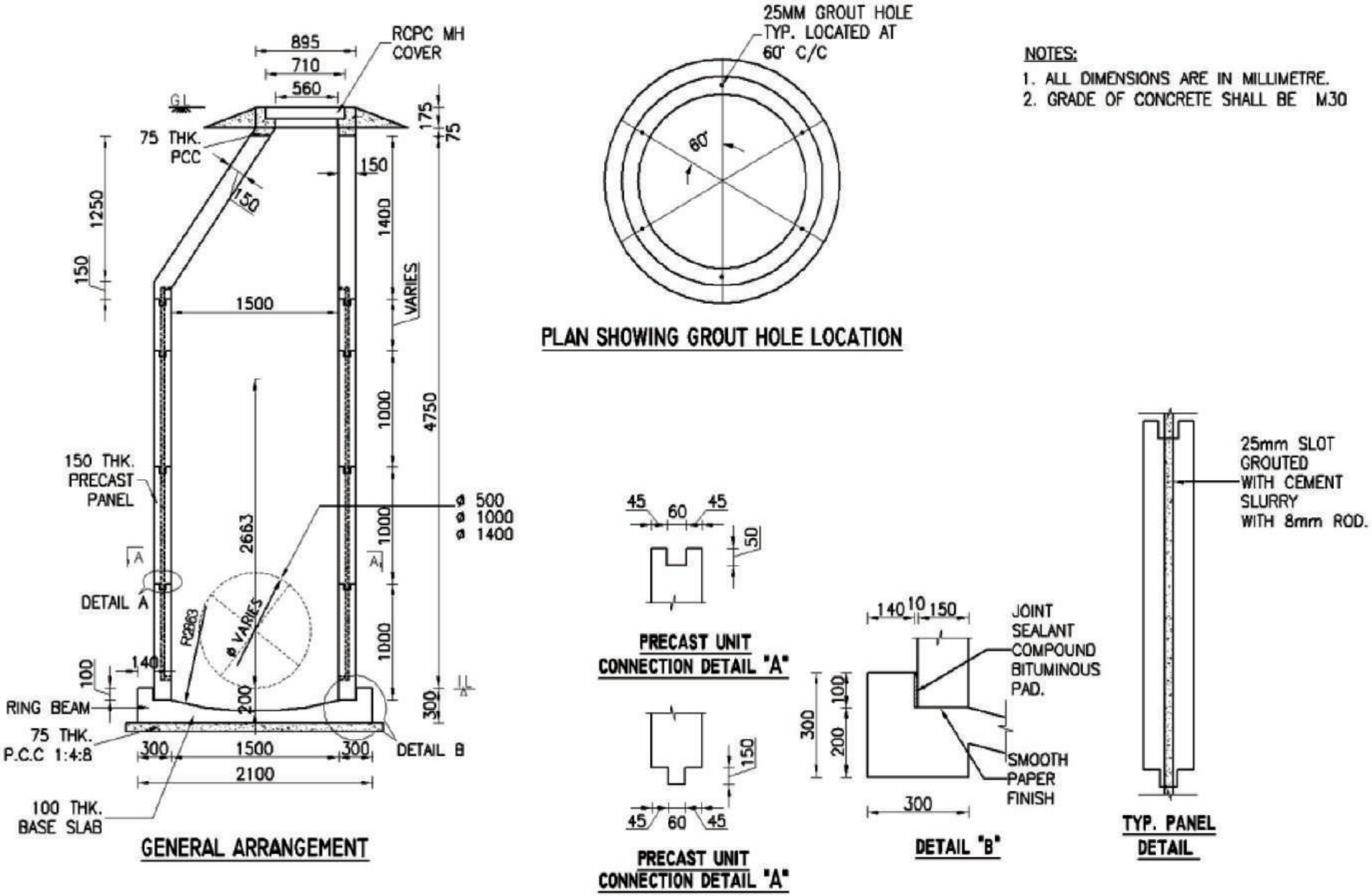


Figure 3.28 RCC Manhole for a depth between 2.3 m to 5 m

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.

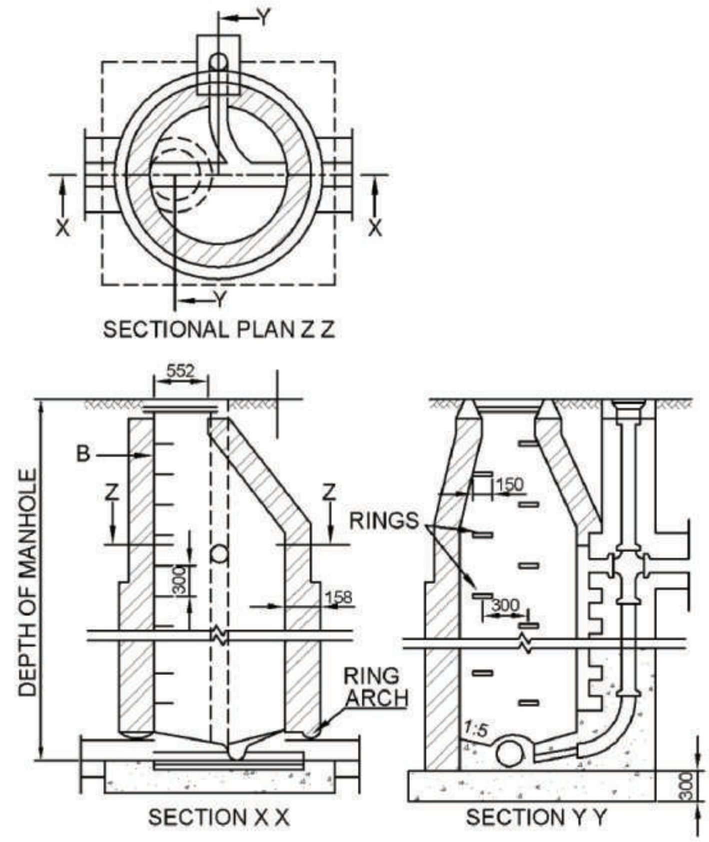
The diameter of the back-drop should be at least as large as that of the incoming pipe. A typical illustration of a drop manhole is shown in Figure 3-29 overleaf.

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 l.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.

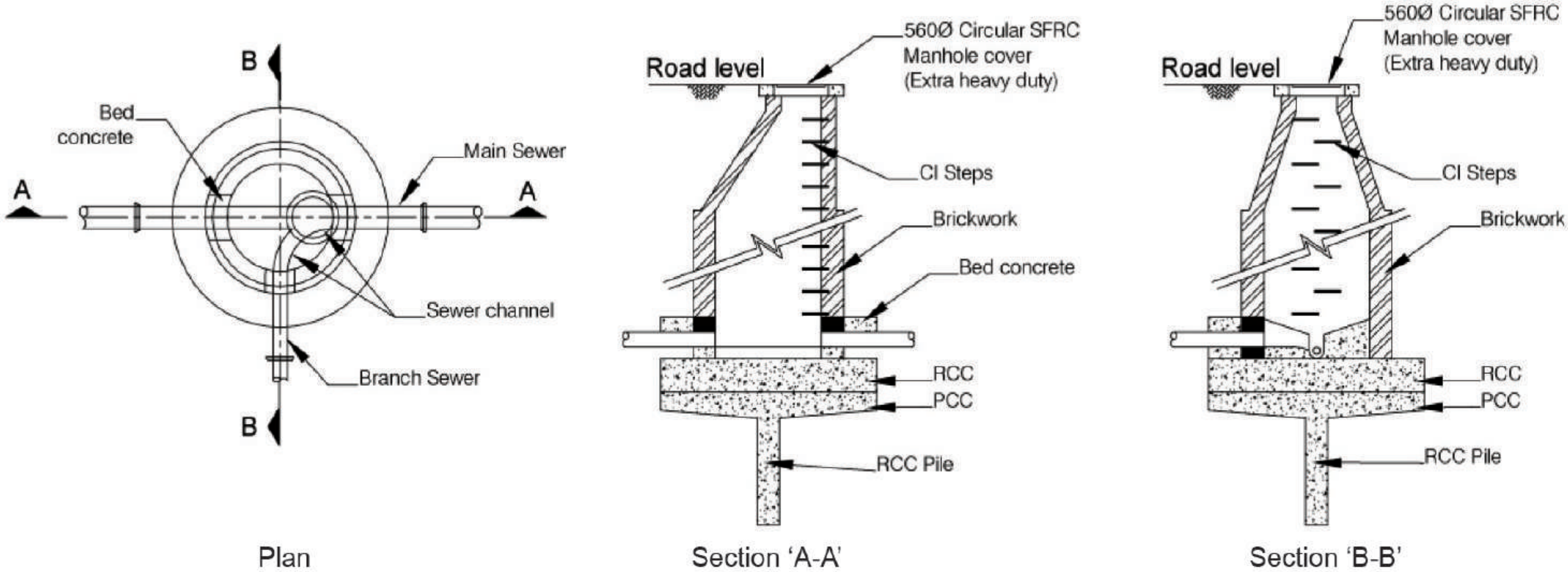
Where the depth of the manhole exceeds 90 cm below the surface of the ground, rungs shall be built into the brickwork. The vertical distance between the two consecutive rungs shall not be more than 30 cm and the centre-to-centre horizontal distance between alternate rungs shall not exceed 38 cm. The 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.

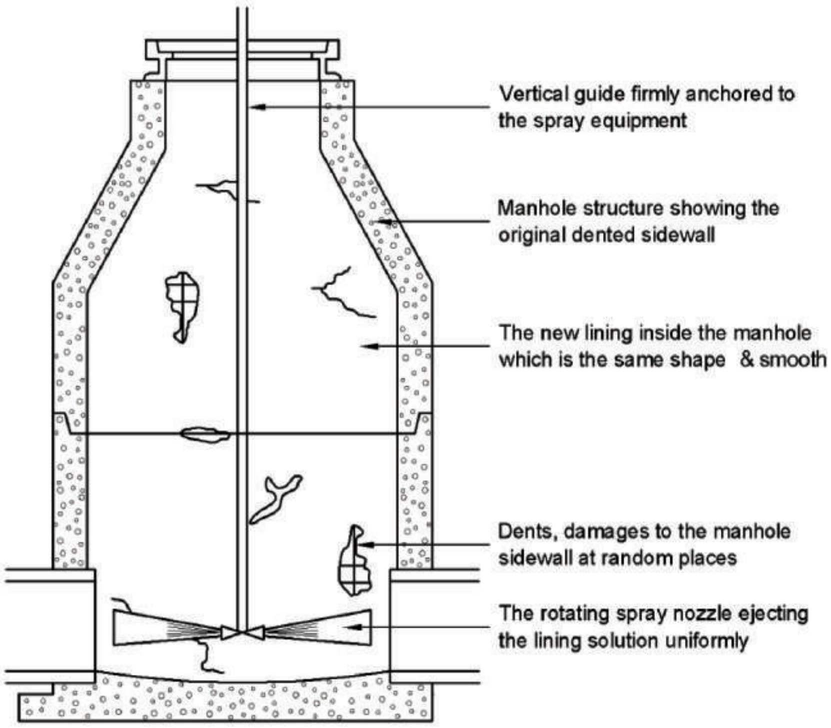


Figure 3.31 Depiction on in-situ sewer manhole lining (Spray equipment not shown)

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

$$W = C\,wB^2 \tag{3.23}$$

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

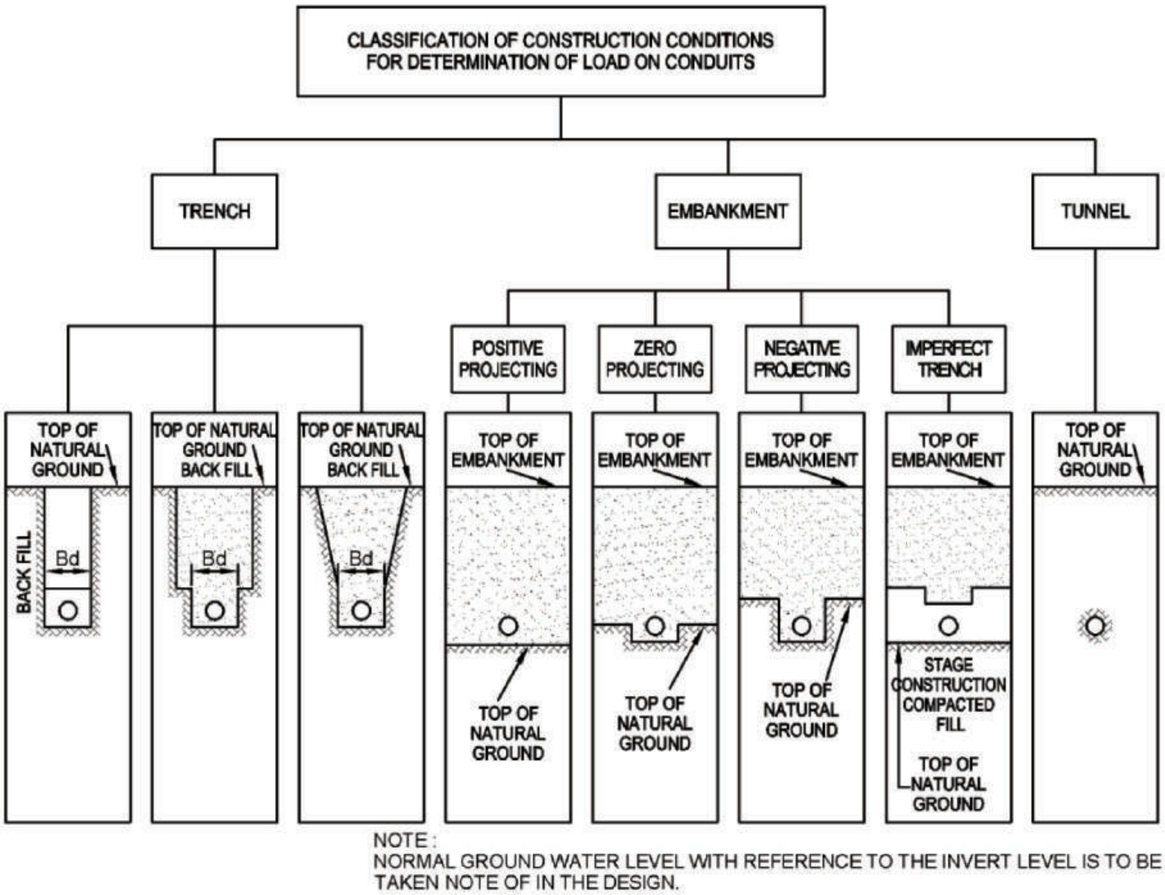
a) Ratio of height of fill to width of trench or conduit

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

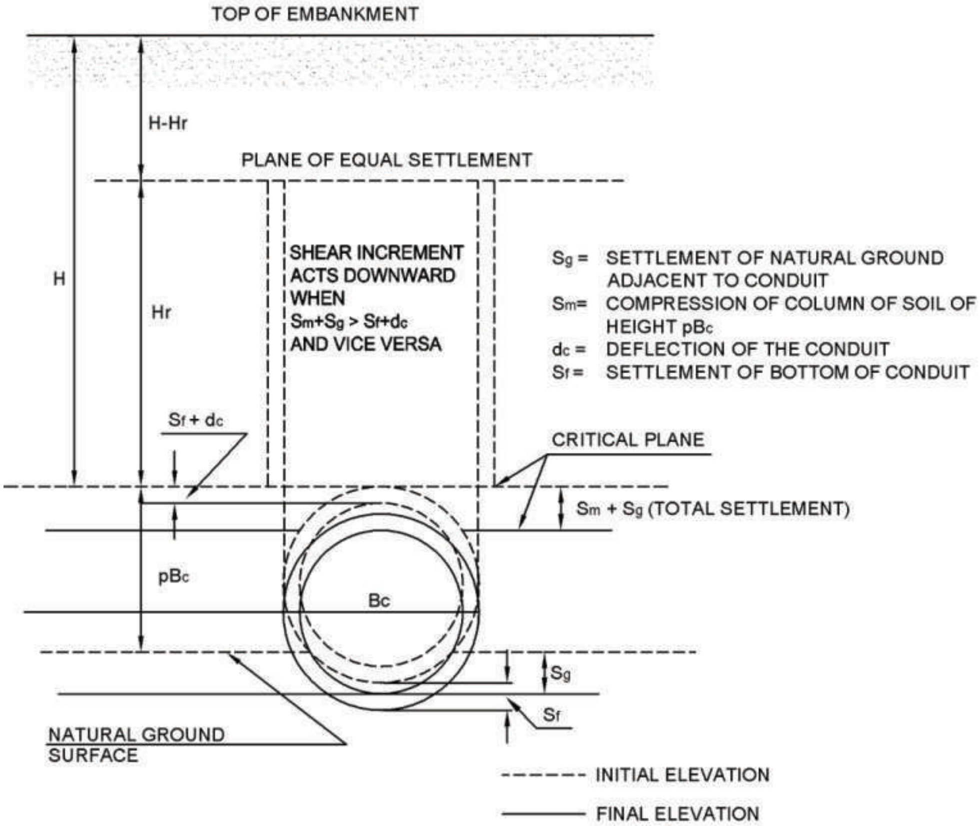
Trench condition exists when the pipe or conduit is installed in a relatively narrow trench (not wider than twice the external diameter of the pipe) cut in undisturbed soil and then covered with earth backfill up to the original ground surface. Tunnel condition exists when the sewer is placed by means 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.

$$Settlement\ ratio\ r_{SD} = \frac{Settlement\ of\ critical\ plane - settlement\ of\ top\ of\ conduit}{Compression\ of\ height\ of\ column\ H\ of\ embankment} \quad (3.24)$$

$$= \frac{(S_m + S_g) - (S_f + d_c)}{S_m}$$

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.

$$W_c = C_c \; w \, B_c^{\;2} \tag{3.25}$$

where,

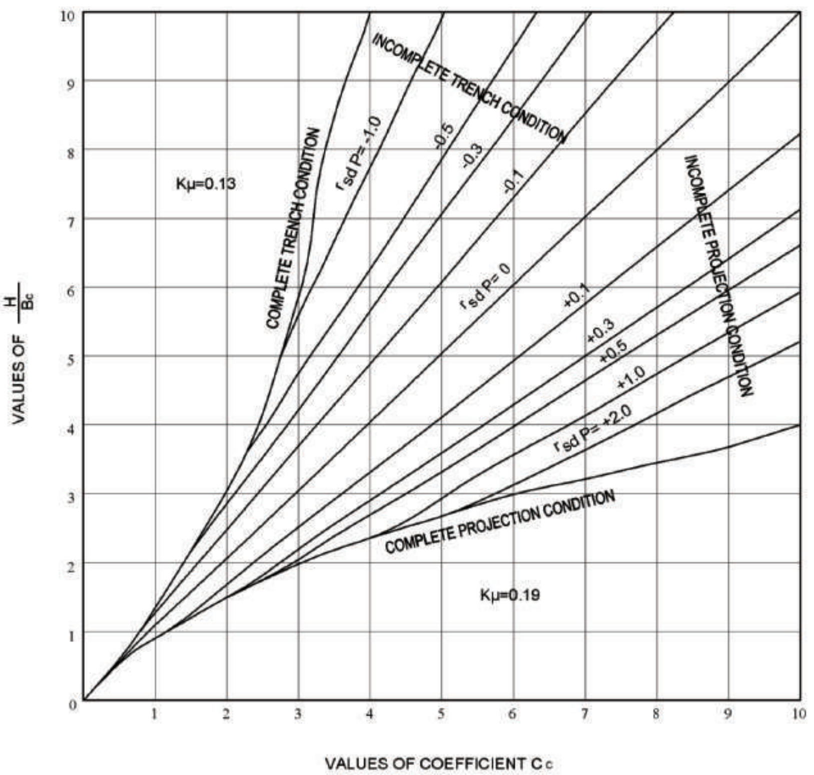
W_c : Load on conduit in kg/m

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.

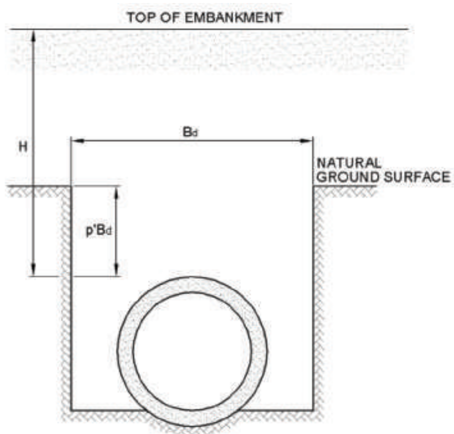


Source: CPHEEO, 1993

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

$$W_c = C_n \, w B_d^{\,2} \qquad (3.26)$$

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

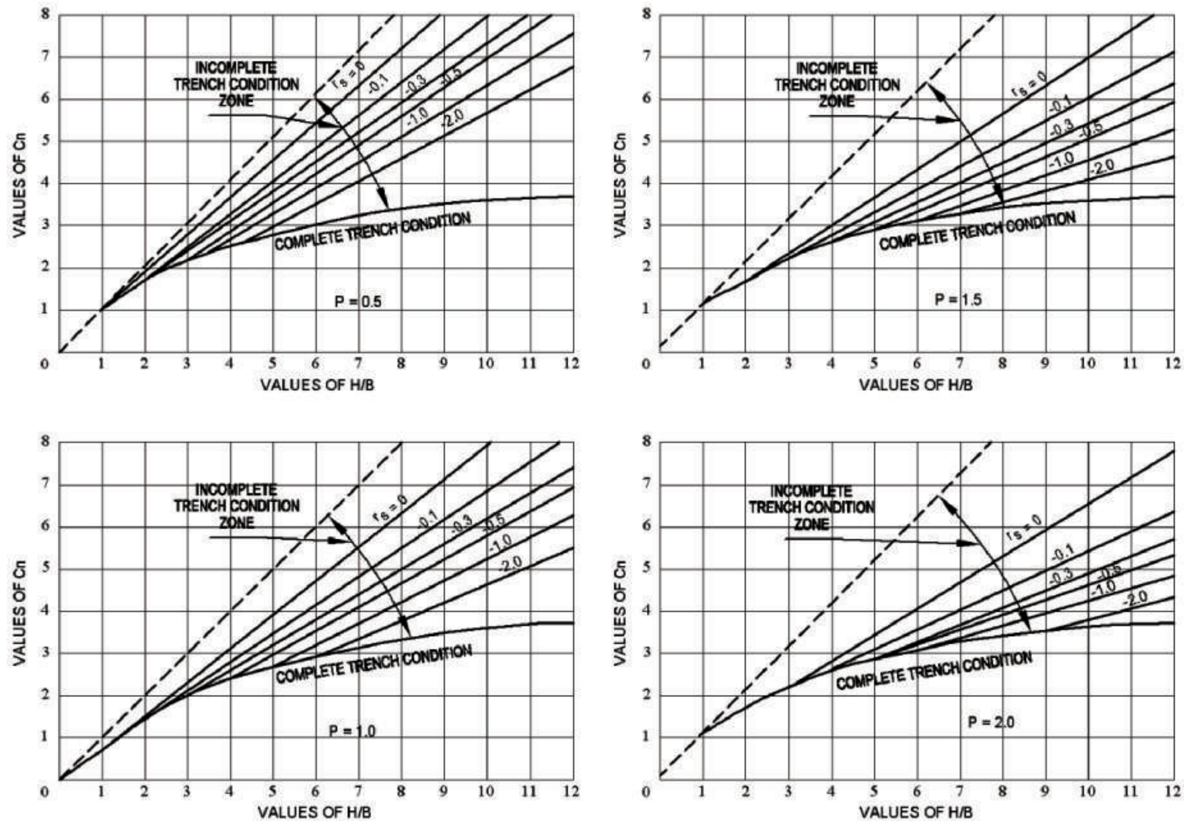
C_n : Load coefficient, which is a function of the ratio (H/Bd) of the height of fill and the width of trench equal to the projection ratio p (Vertical distance from the firm ground surface down to the top of the conduit/width of the trench) and the settlement ratio rsd given by the expression,

$$r_{sd} = \frac{\text{settlement of natural ground} - \text{settlement of critical plane}}{\text{compression of the backfill within the height } pB_d} \qquad (3.27)$$

$$= \frac{S_g - (S_d + S_f + d_c)}{S_d}$$

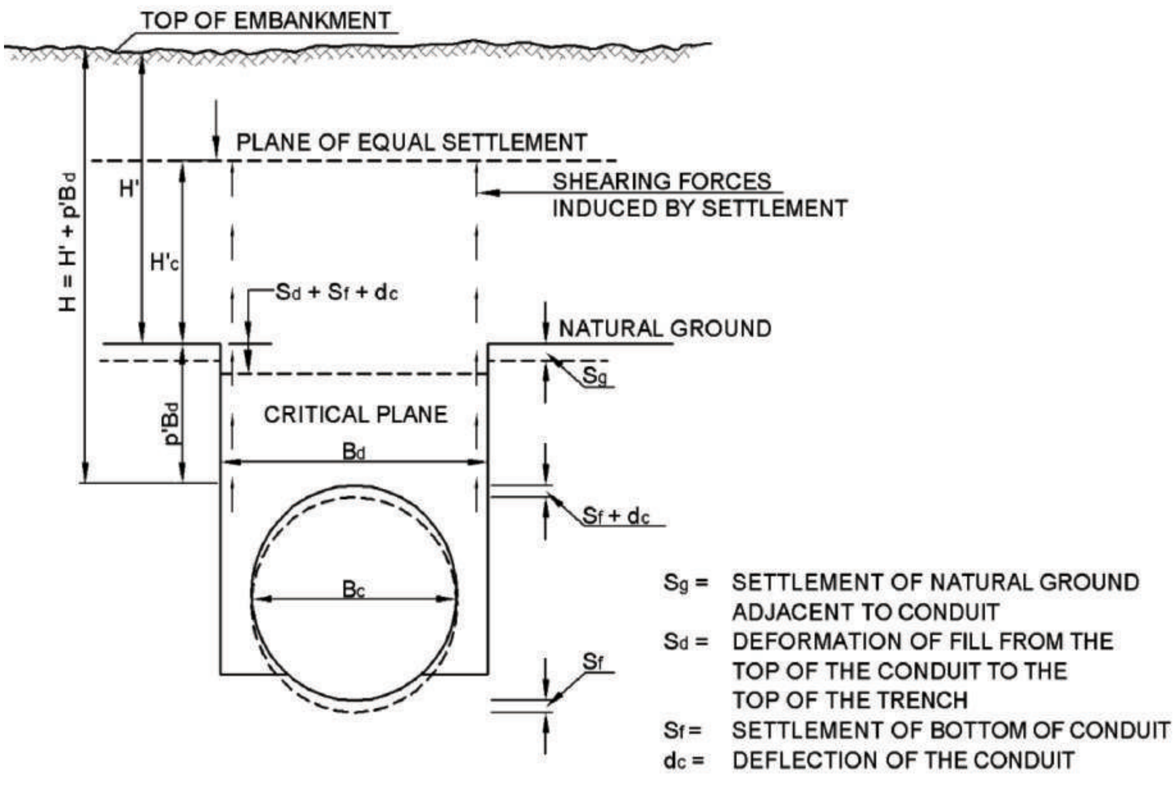
Values of C_n, for various values of H/B_d, r_{sd}, and p are given in Figure 3.36 overleaf.

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

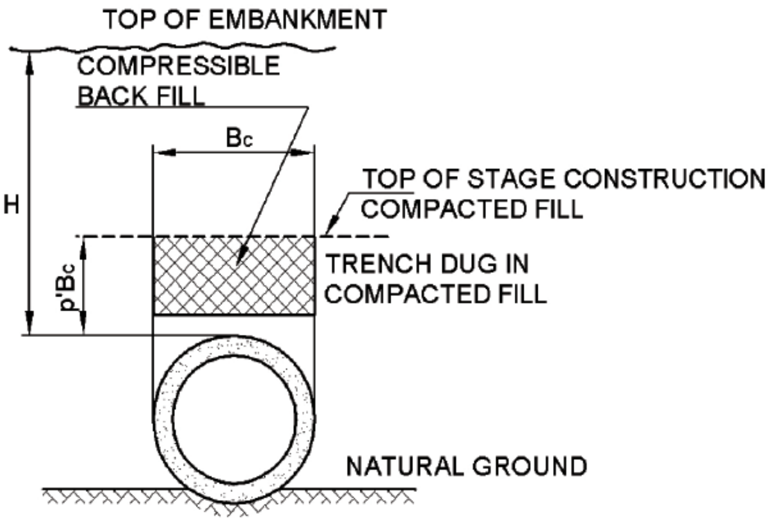


Source: CPHEEO, 1993

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

$$W_c = C_n \; w \, B_c^{\;2} \tag{3.28}$$

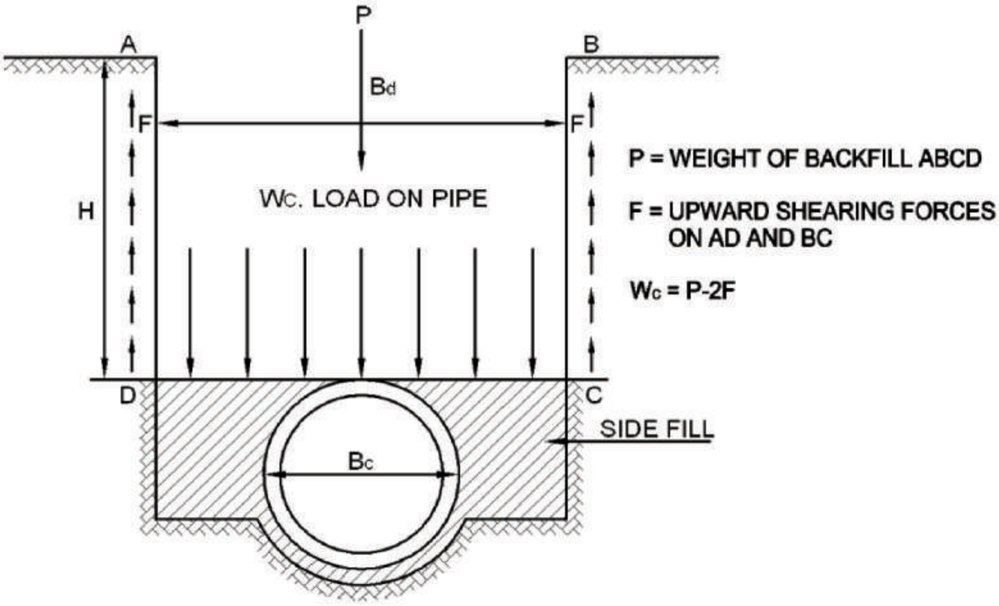
The values of Cn in this case also may be obtained from Figure 3.36 for negative projecting conduits taking Bc= Bd on the assumption that the trench fill is no wider than the pipe.

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

$$W_c = C_d \; w \, B_c \; B_d \tag{3.30}$$

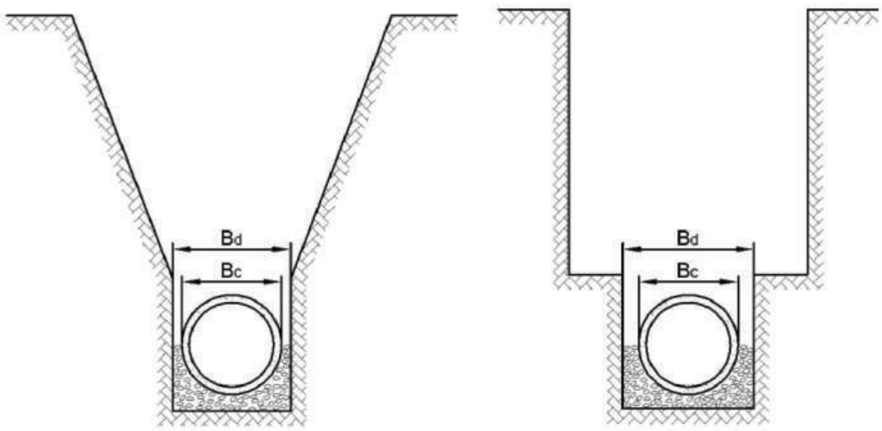
where,

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.



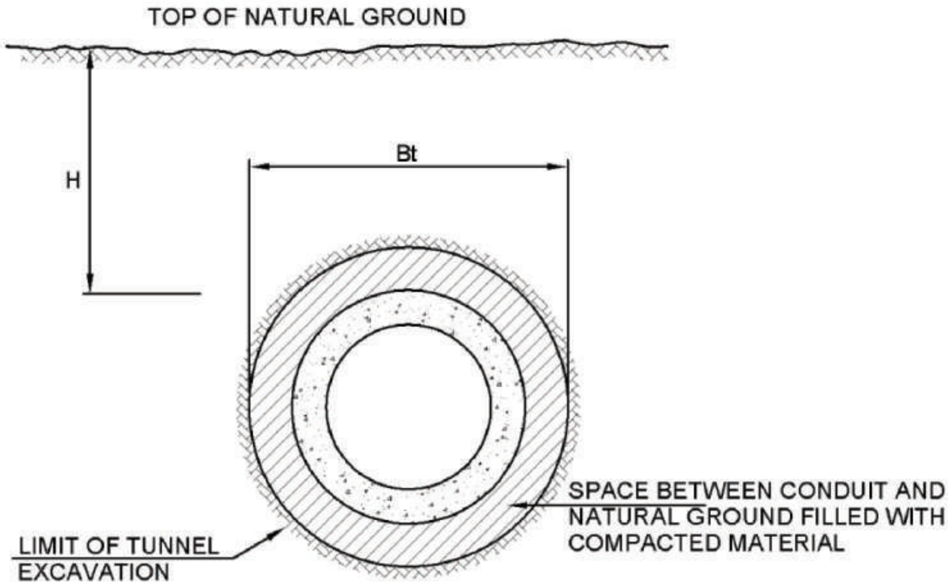
Source: CPHEEO, 1993

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:

$$W_t = C_t\, B_t \left(wB_t - 2C\right) \qquad (3.31)$$

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

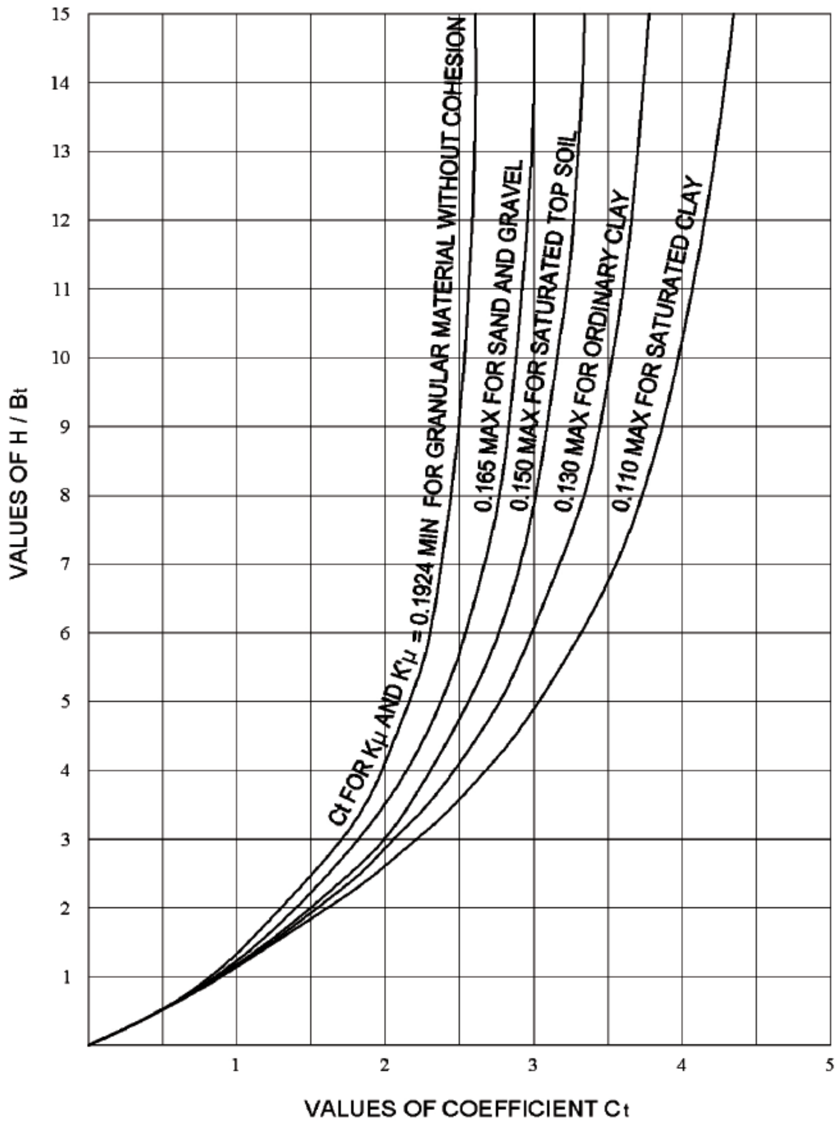
B_t : Maximum width of the funnel excavation in m

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:

$$H_{\min} B_c (w_s - w_o) + W_c = (\pi / 4) B_c^2 w_o \quad (3.32)$$

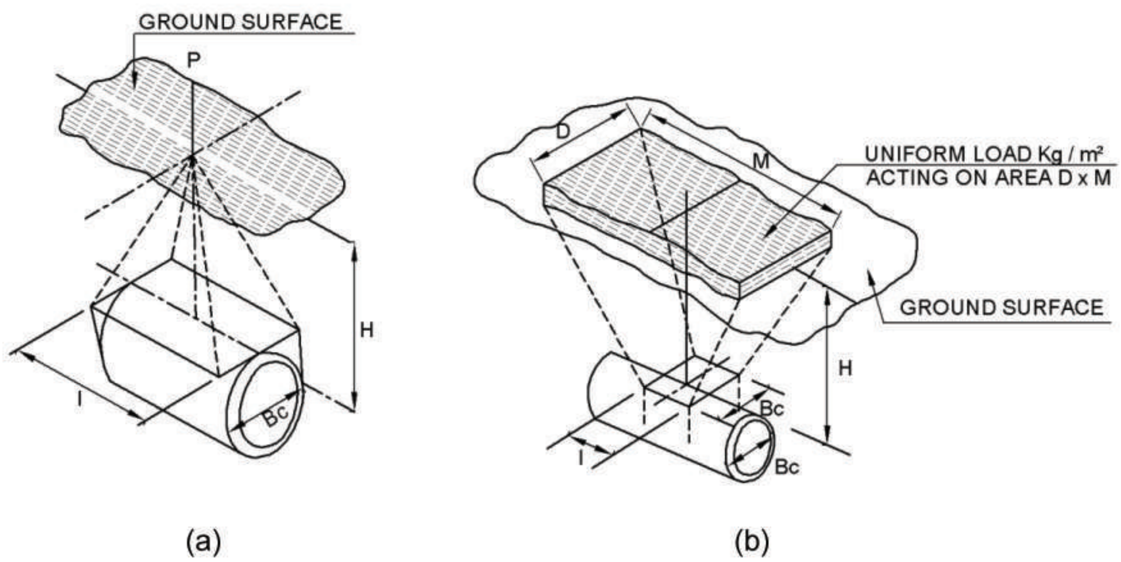
where,

- 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),

$$\begin{aligned} W &= \text{Field supporting strength} / \text{Factor of safety} \\ &= \text{Load factor} \times \text{three edge bearing strength} / \text{Factor of safety} \end{aligned} \tag{3.39}$$

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.

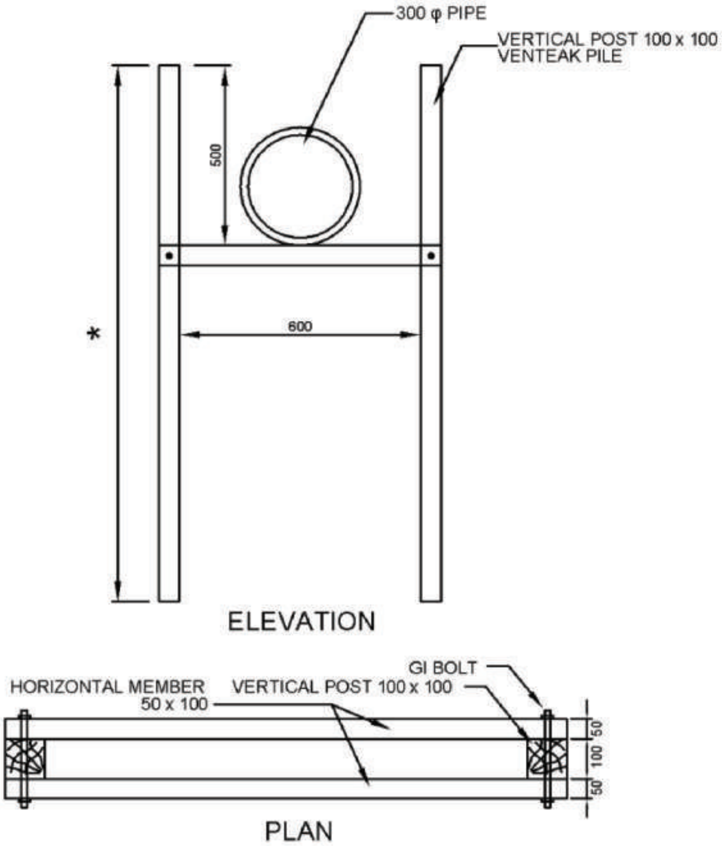
4. All pipes used on the work should be tested as per the IS specifications and test certificates of the manufacturers should be furnished for every consignment brought to the site.

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.

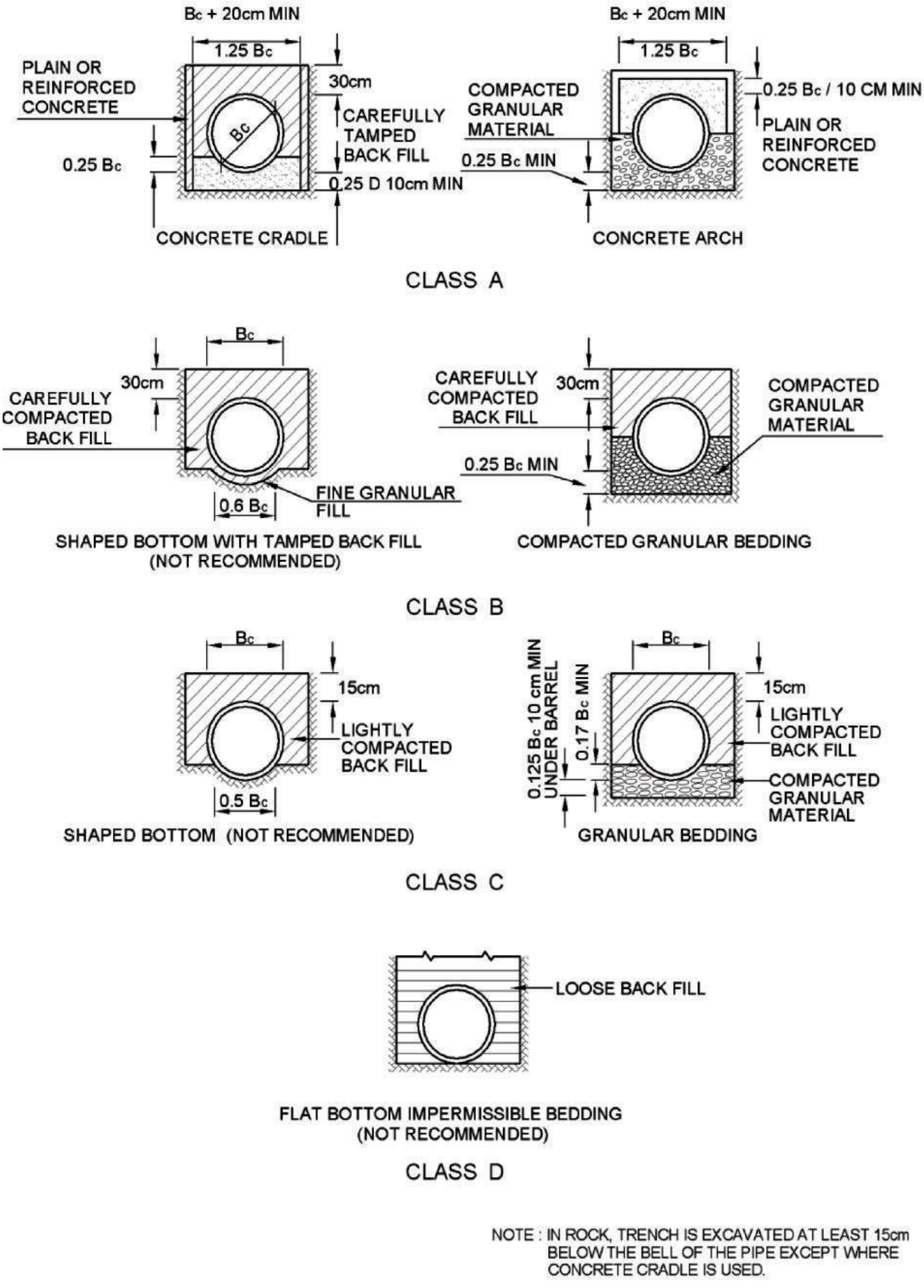
The pipe bedding materials must remain firm and not permit displacement of pipes.

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.

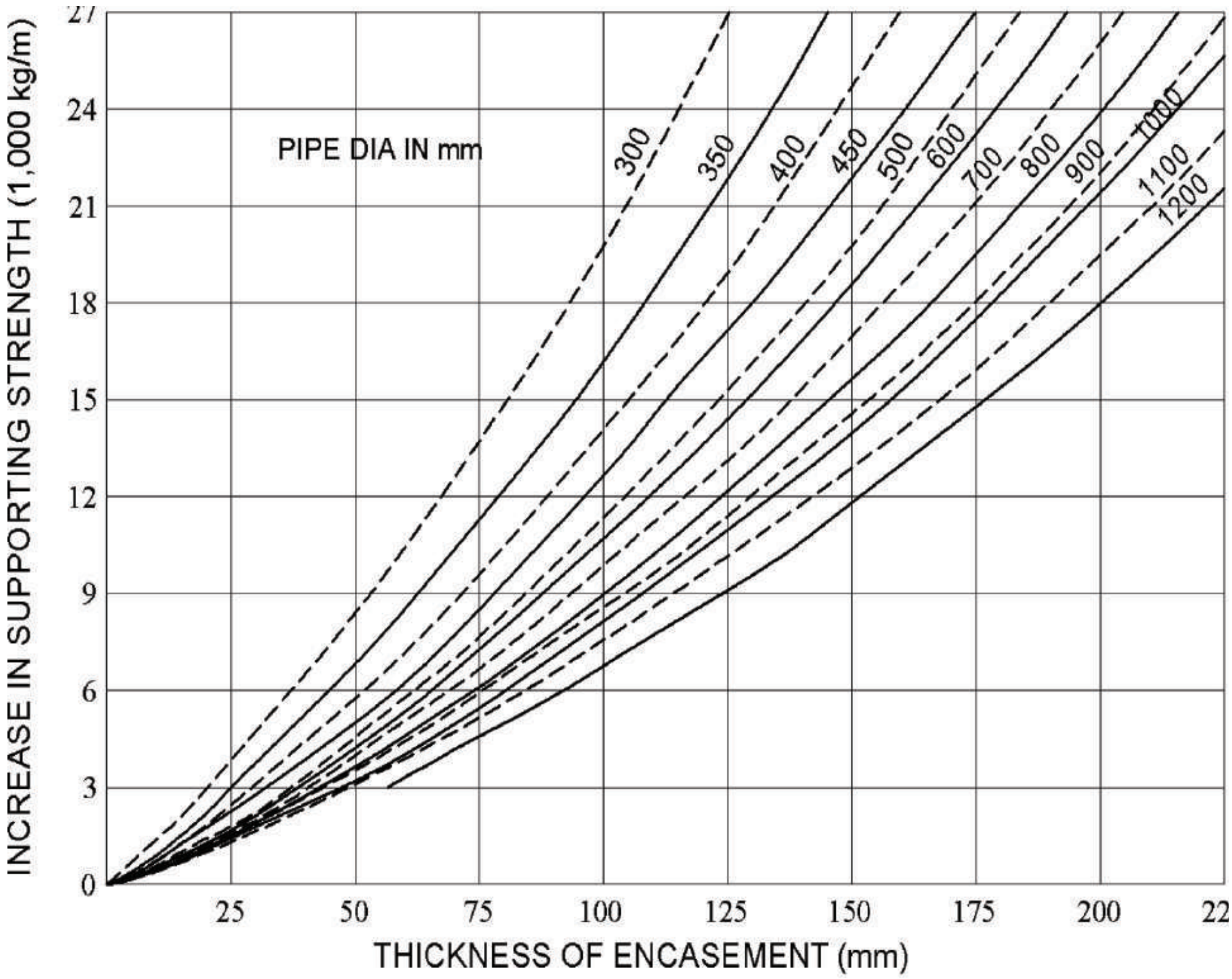
Class A: The hard unyielding material should be excavated down to the bottom of the concrete cradle.

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.

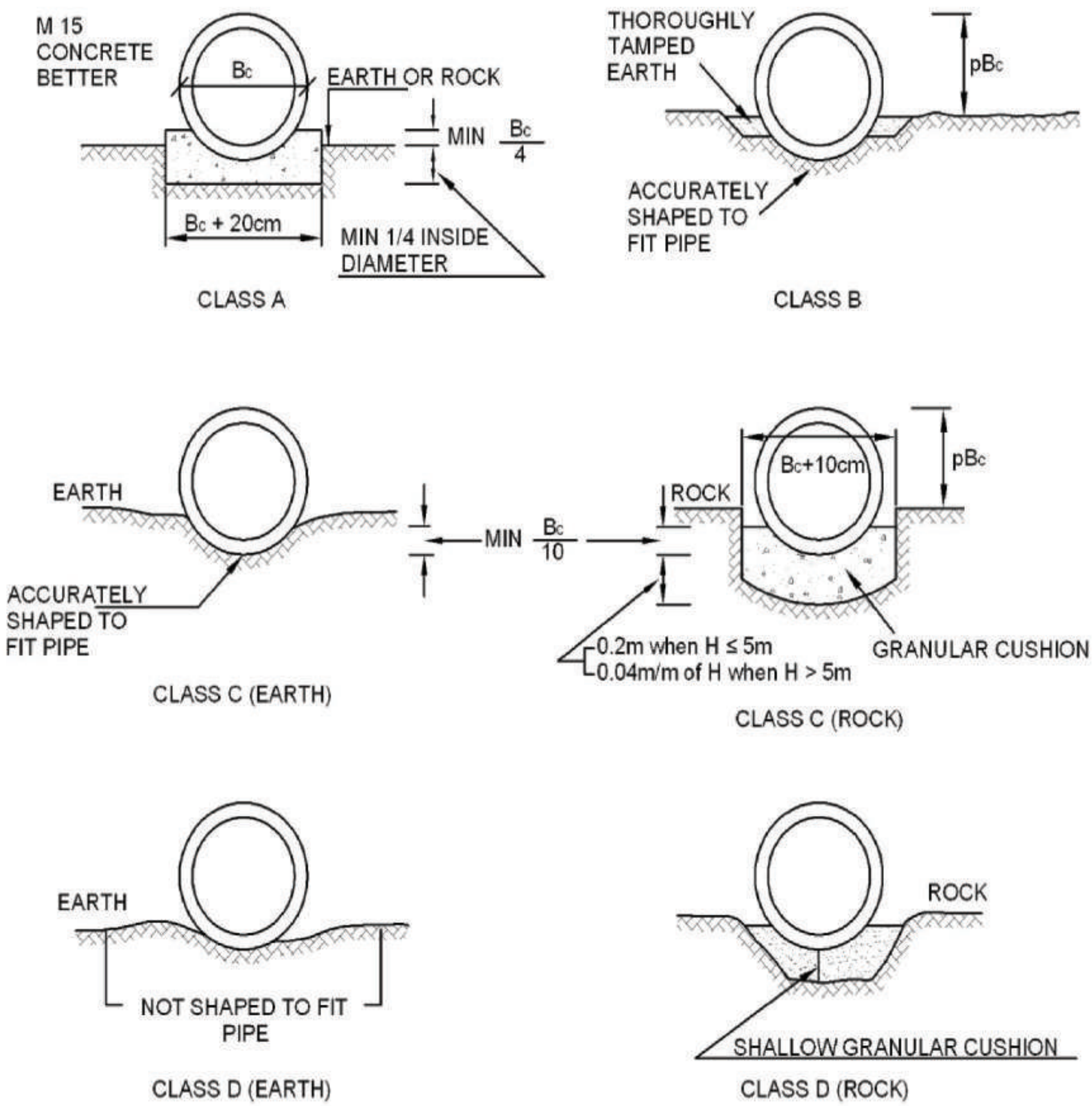
3.50.3.7 Classes of Bedding

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

$$\text{Safe working strength} = \frac{\text{Ultimate three edge bearing strength}}{\text{Factor of safety}} \qquad (3.44)$$

Safe field supporting strength = Safe working strength × load factor (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.

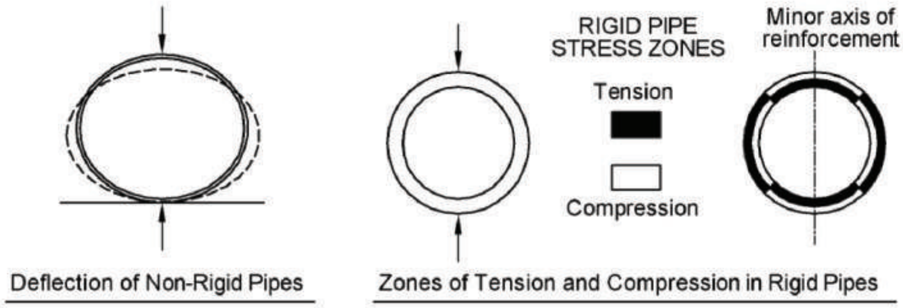


Figure 3.50 Mechanism of resisting superimposed loads by rigid and non-rigid piping

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.

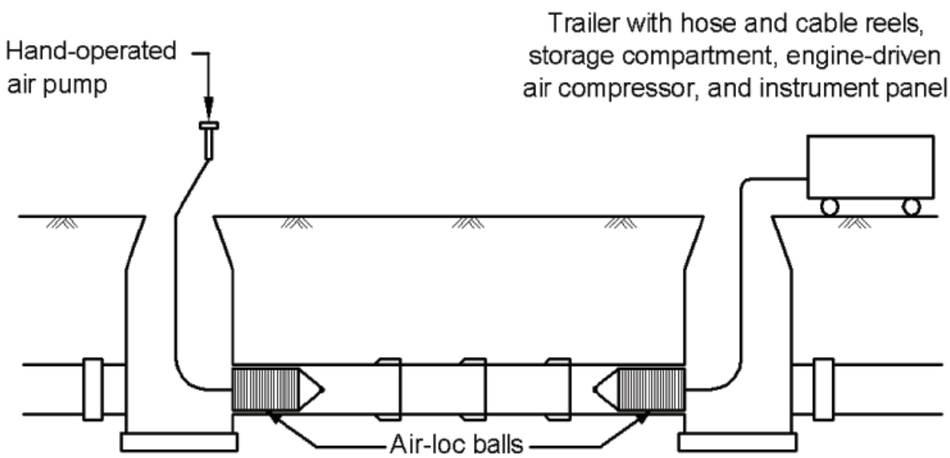


Figure 3.52 Typical arrangement for low pressure air testing of sewer pipeline

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.

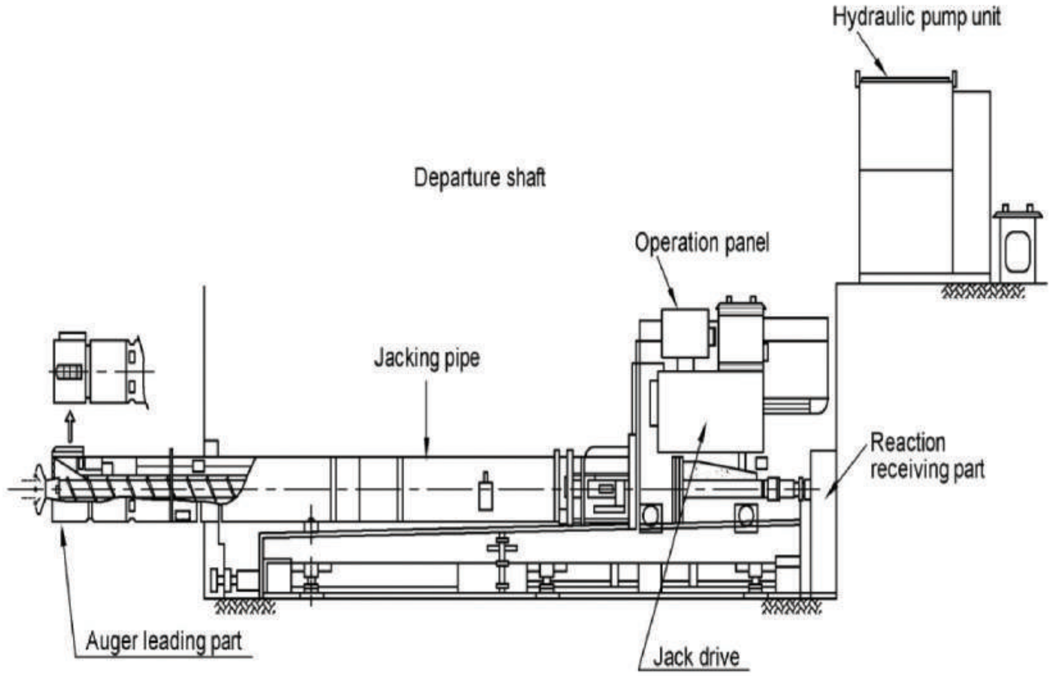
g) Guided Auger Boring

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.

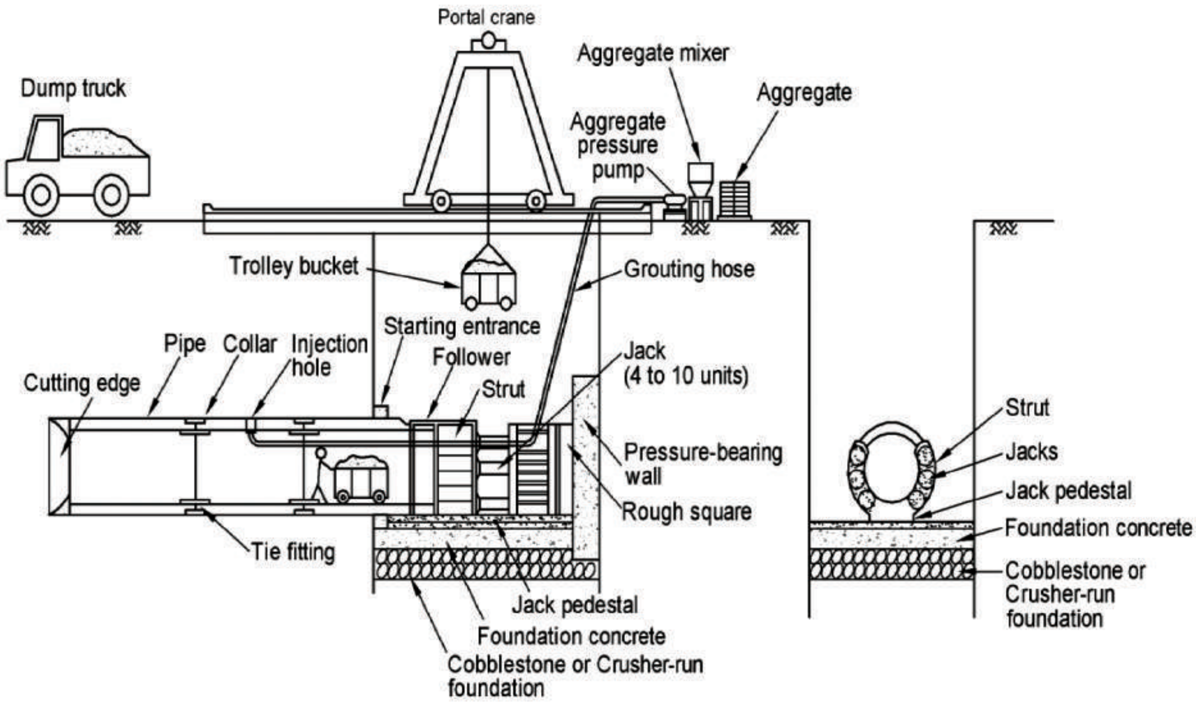
The jacking pit size is a function of pipe diameter, length of pipe segment, shield dimensions, jack size, thrust wall design, pressure rings and guide rail system. The space available at the site governs the selection of all the above components. The jacking pit should be shored and braced unless it 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

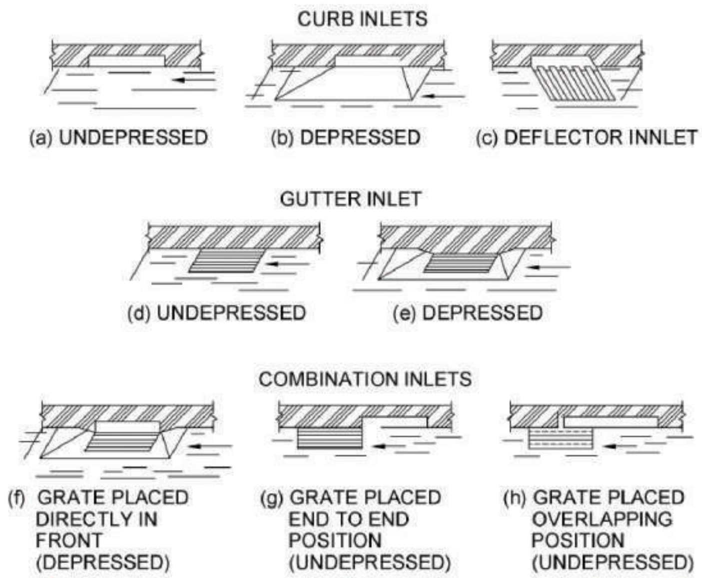
These consist of horizontal openings in the gutter, which is covered by one or more gratings through which the flow passes.

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

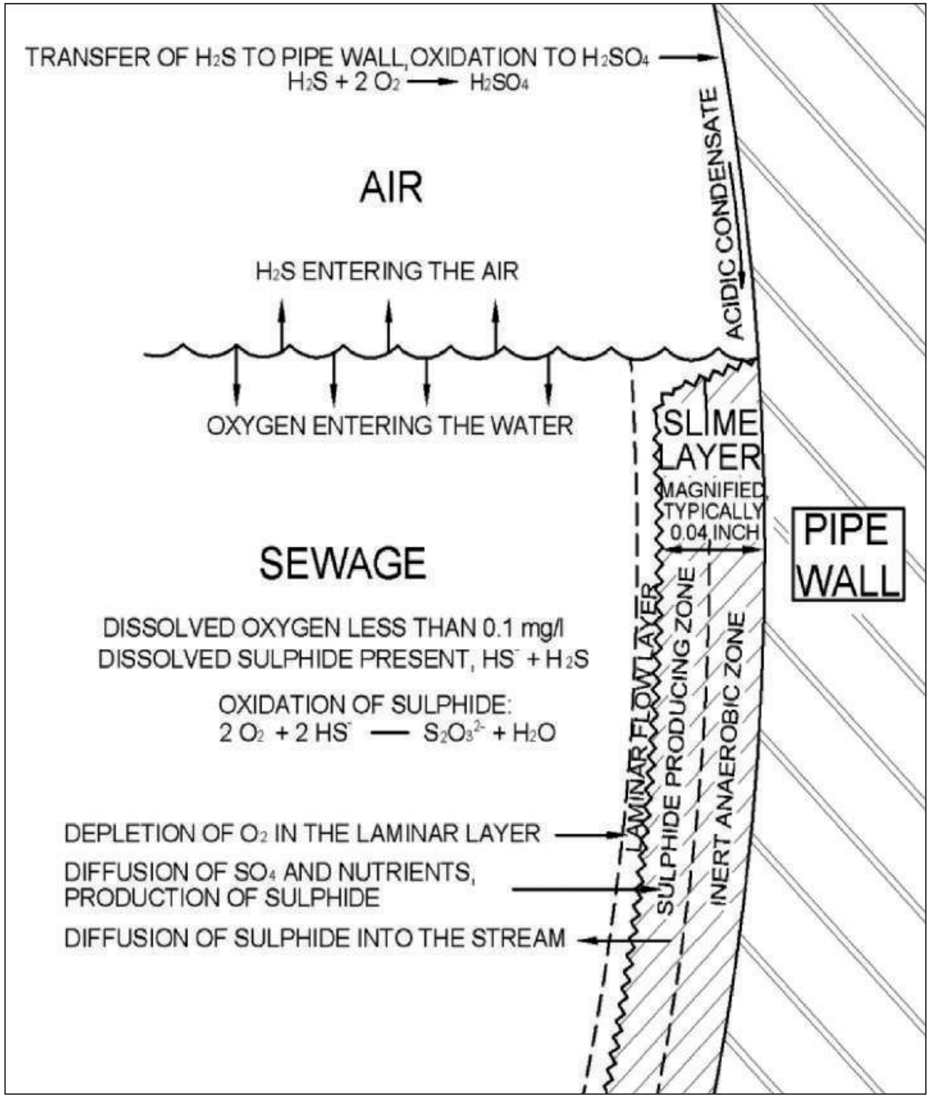
Since sulphide generation is a biological phenomenon, it is obvious that sewage temperature influences the rate of sulphide generation. Temperature below 20°C generally will not cause any appreciable sulphide build up. From 20°C to 30°C, the rate of sulphide generation increases at about 7% per °C rise 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.

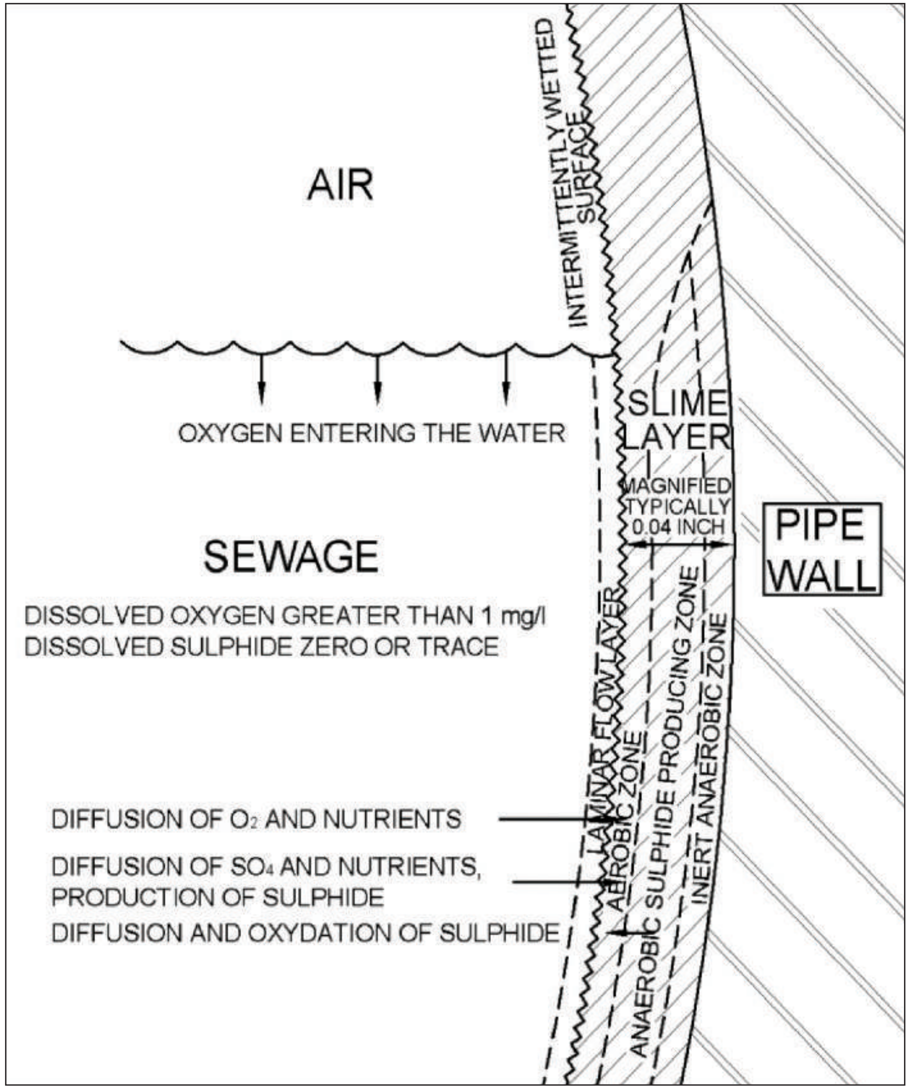
For any specified sewage temperature and flow condition in a sewer, there is limiting sewage strength, usually less than 80 mg/l of BOD, below which a build-up of hydrogen sulphide will practically cease.

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

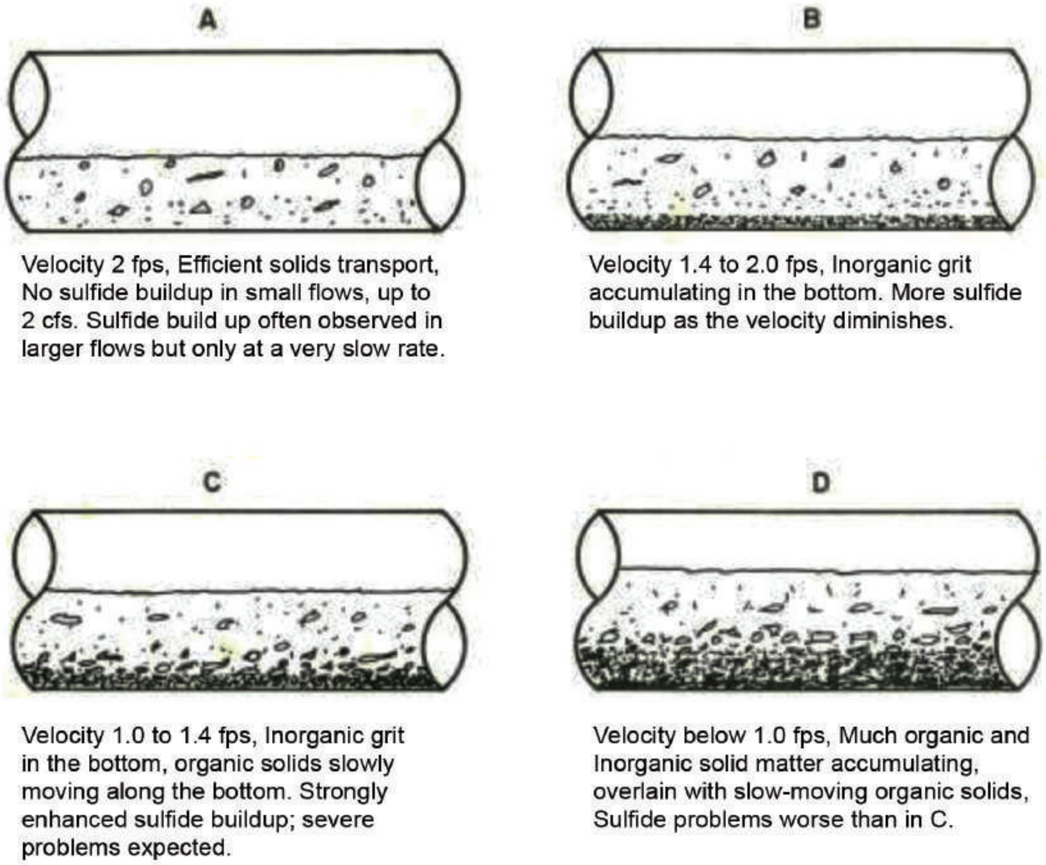


Source: USEPA, 1974

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.

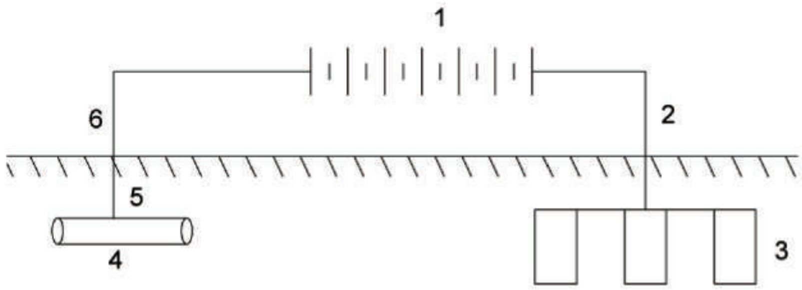
3.69.7.3 Cathodic Protections

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.

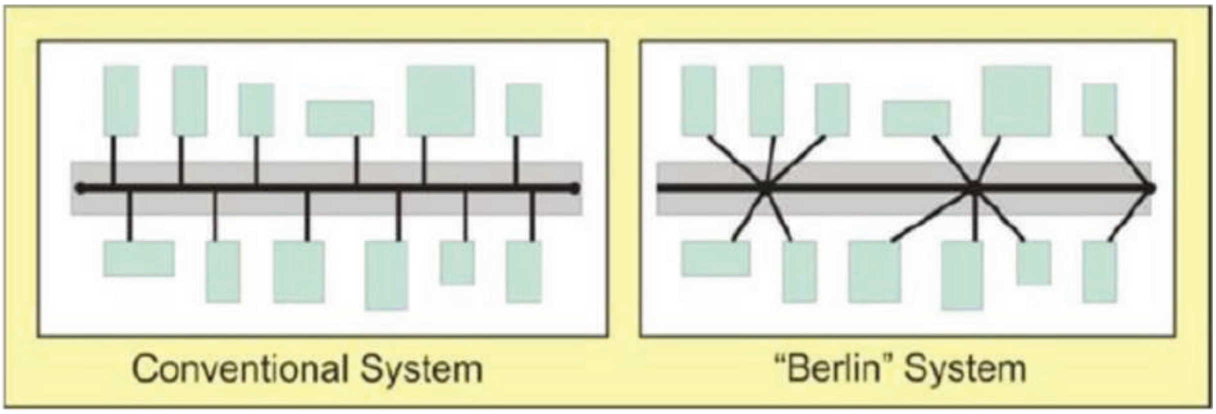
From this point of view, it becomes necessary to limit the length of such house service sewers. Considering that typically properties are developed on plots of about 10 m width, a spacing of 30 m between manholes permits both the objectives of easy cleaning of public sewer stretches and eliminating the 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.

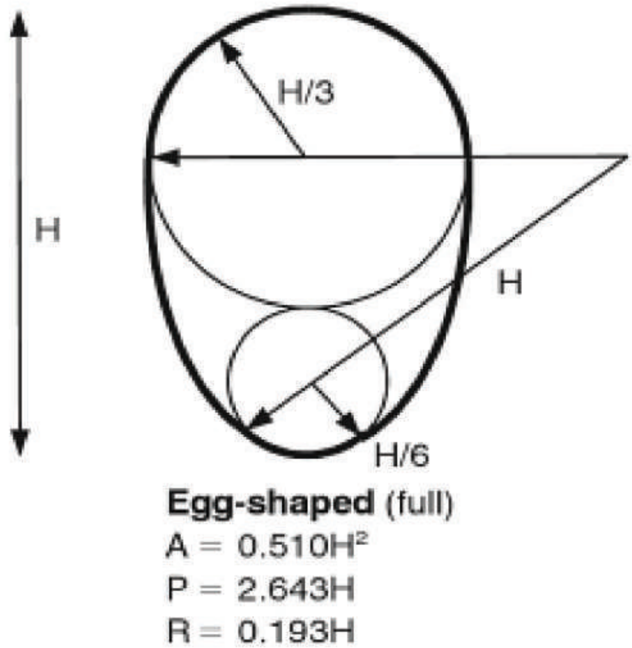


Figure 3.62 Geometry and hydraulics of egg shaped sewer at full flow

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

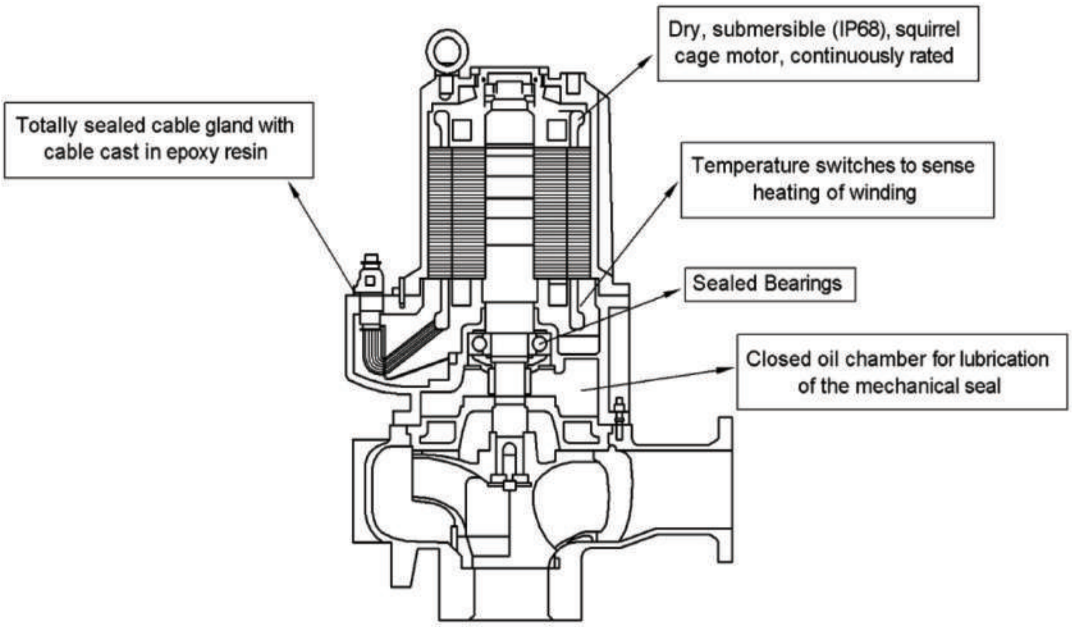
4.5.1 Types of Pumps

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

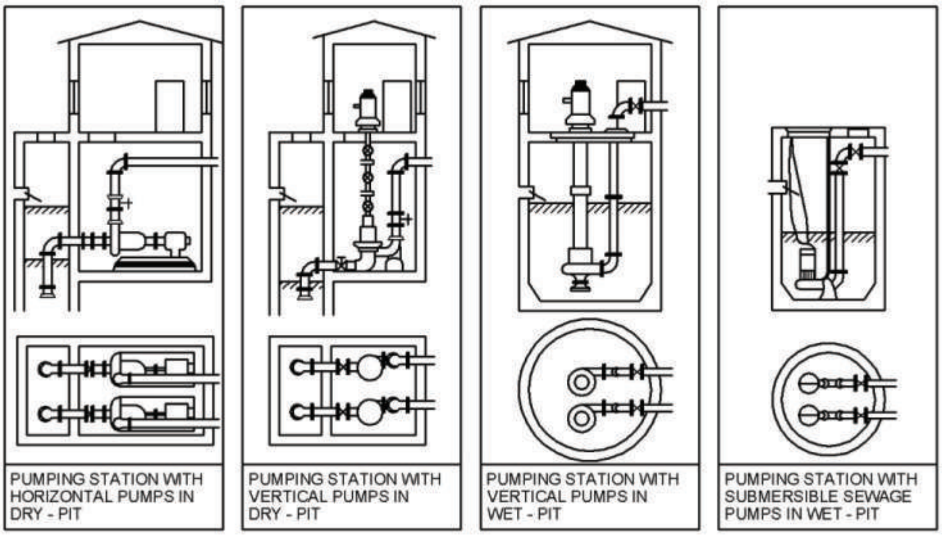
SELF-LOCKING CLAMP TO SEAL THE PUMP DELIVERY TO THE PIPE BRANCH

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

$$V = T \times Q/4 \tag{4.1}$$

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.

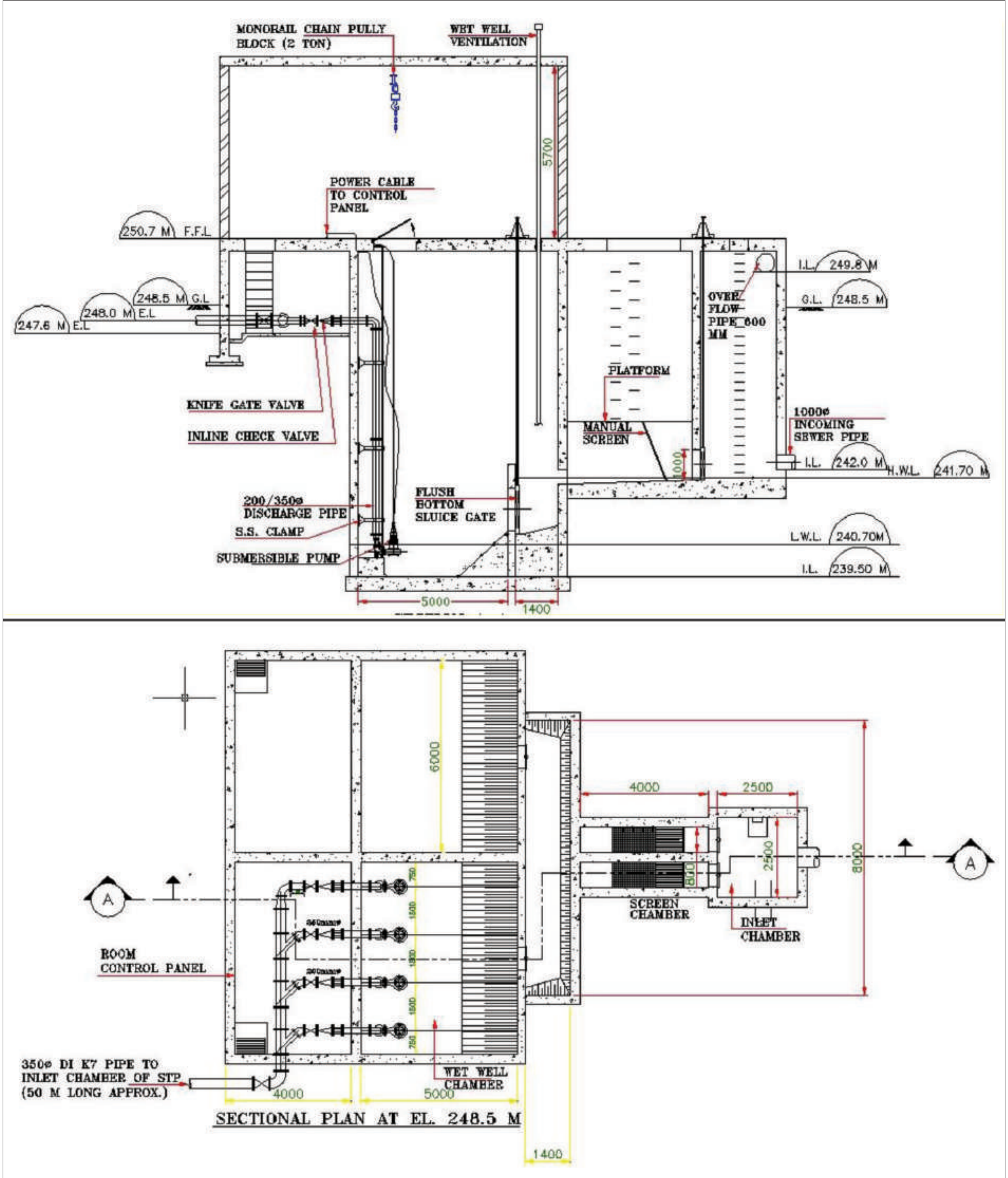
Following points should be considered while designing the wet well pumping system:

- 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:

$$n_s = \frac{3.65n\sqrt{Q}}{H^{0.75}} \tag{4.2}$$

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

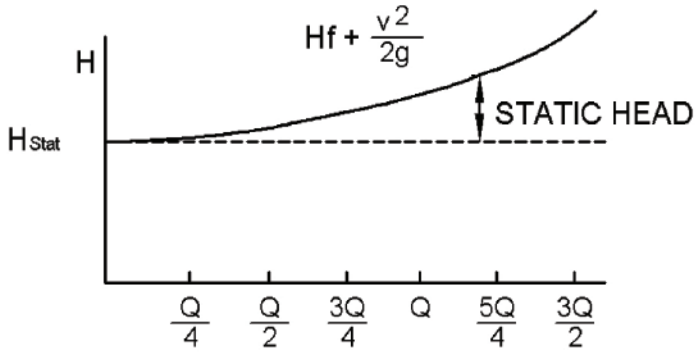
At the stage of planning, the method of computing the total head of pumping should be to estimate it over a range of flow-rates, for different variations in the static levels and for different options of piping sizes and layouts. This obtains the system head curve, as illustrated in Figure 4-6 overleaf.

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.

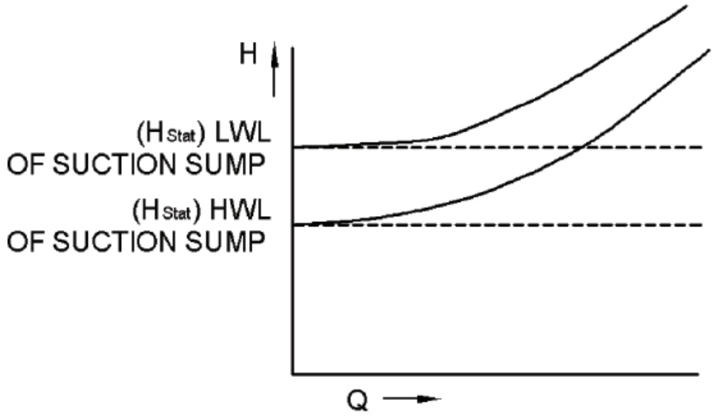
Table 4.2 Friction factor for fittings in pumping mains

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



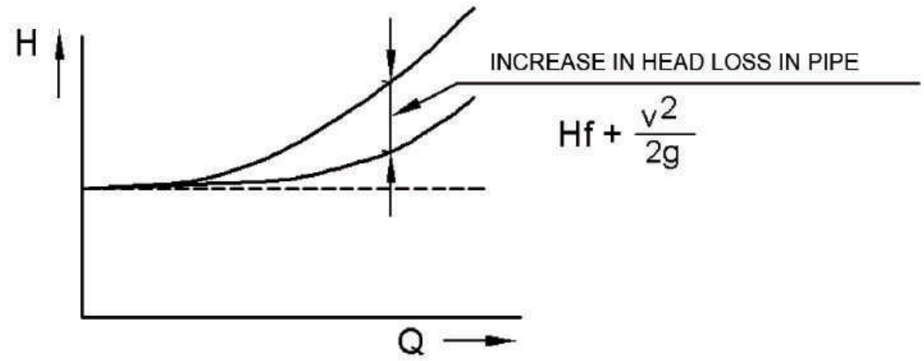
Source:CPHEEO, 1993

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



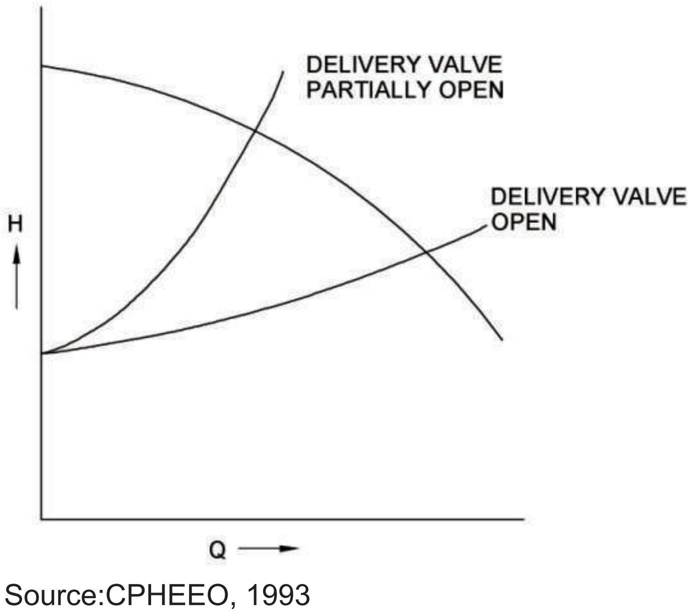
Source:CPHEEO, 1993

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.

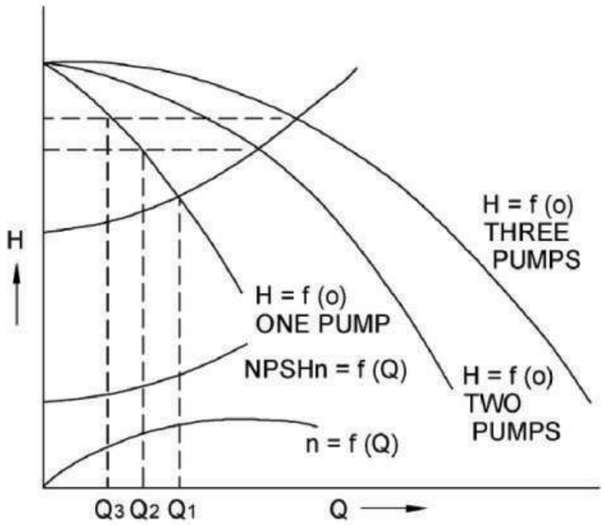


Source:CPHEEO, 1993

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.



No. OF PUMPS IN OPERN	FLOW OF EACH PUMP
1	Q_1
2	Q_2
3	Q_3

Source: CPHEEO, 1993

Figure 4.10 Operation of pumps in parallel

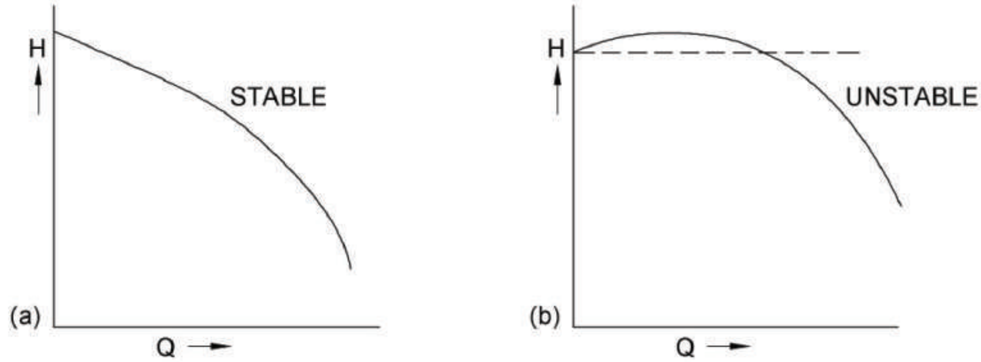
The operating point of parallel operation is the point of intersection of the combined H-Q curve with the system head curve. Because the point of intersection on the combined characteristics is at a head higher than that at the point of intersection on the H-Q curve of a single pump, the discharge at 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.

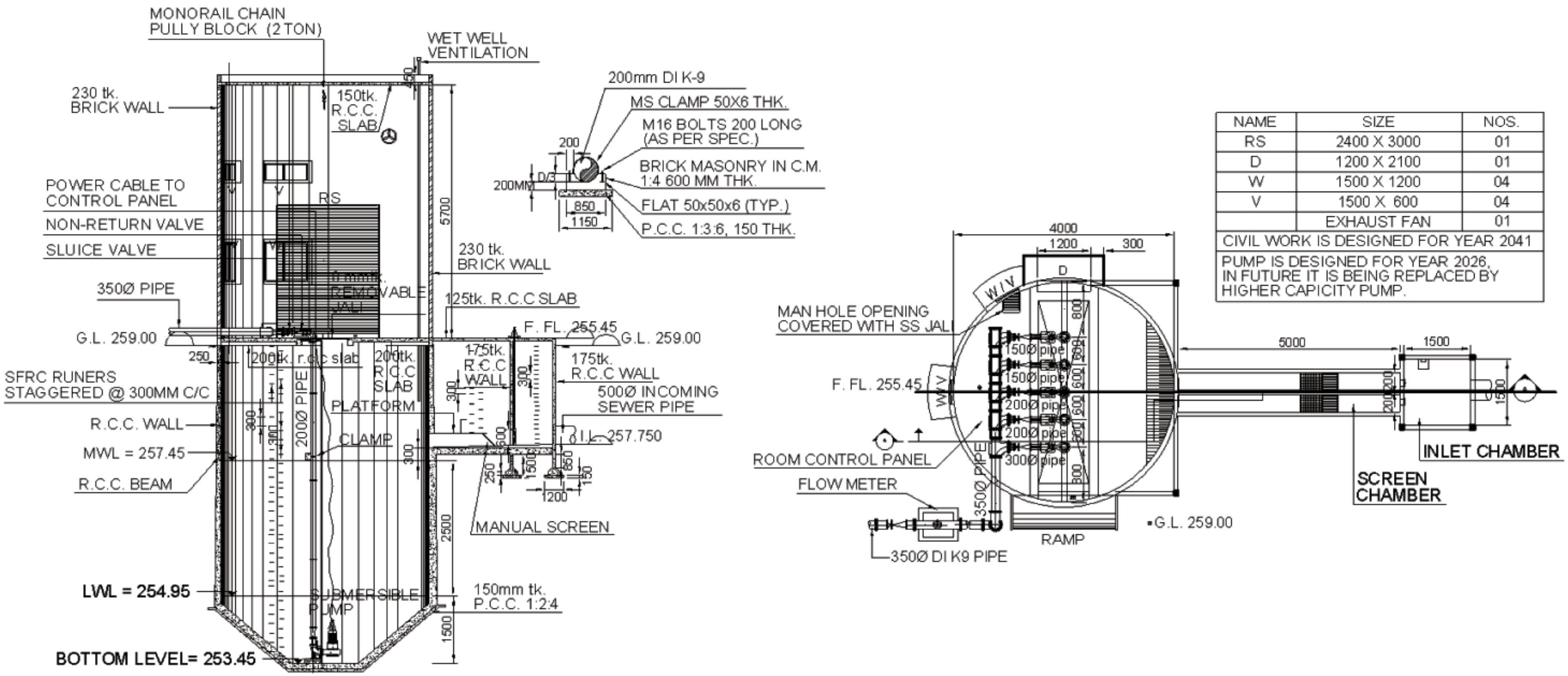
However, pumps to be mounted vertically are supplied as sub-assembled. For the installation of these pumps, the proper sequence of assembly has to be clearly understood from the drawing of the pump manufacturer.

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

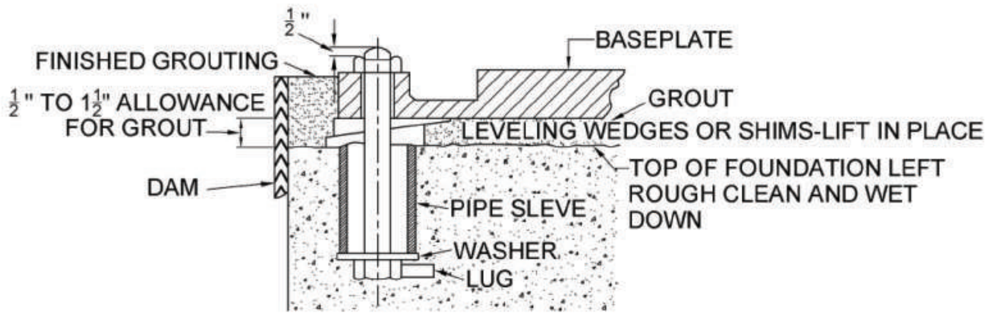
The foundation should be sufficiently substantial to absorb vibrations and to form a permanent, rigid support for the base plate.

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



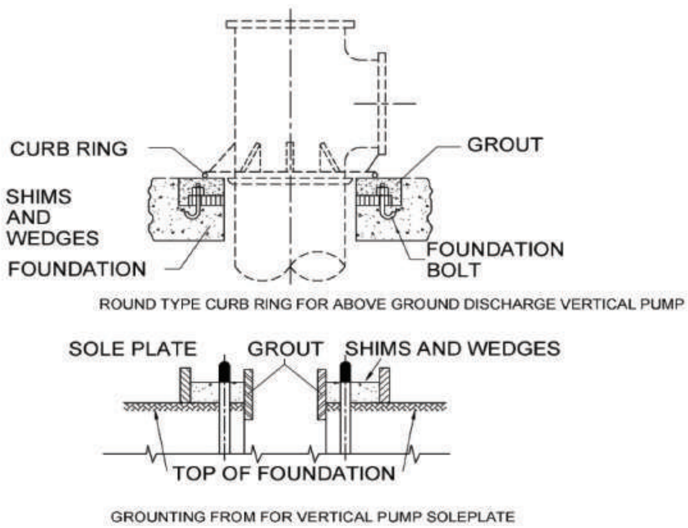
Source: CPHEEO, 1993

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

$$\text{kW required} = Q \times H / 100.5 / \acute{\eta} \tag{4.3}$$

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.

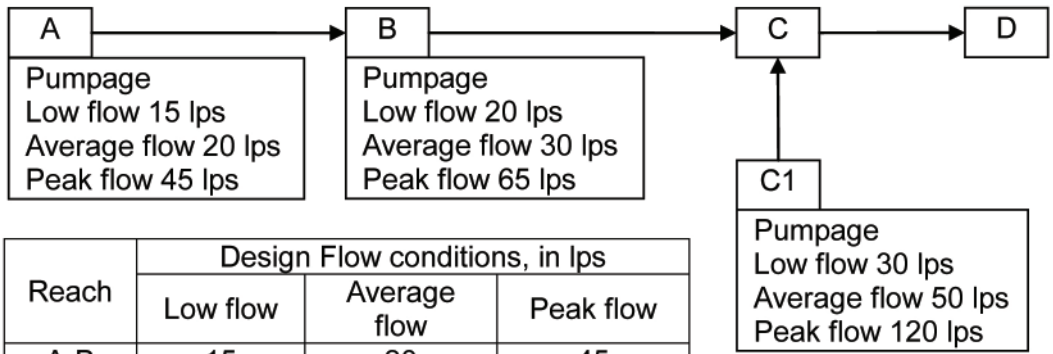
Whenever the velocities are too small, the organic materials get settled out and undergo anaerobic decay and release the sulphide, which later combines with the moisture and forms sulphurous and sulphuric acid.

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.



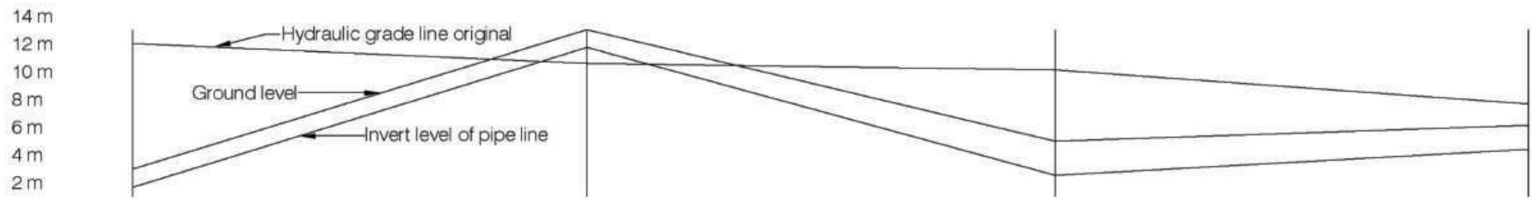
Reach	Design Flow conditions, in lps		
	Low flow	Average flow	Peak flow
A-B	15	20	45
B-C	35	50	110
C1-C	30	50	120
C-D	65	100	230

Figure 4.15 Illustration of pumping main hydraulics on serial pumping mains

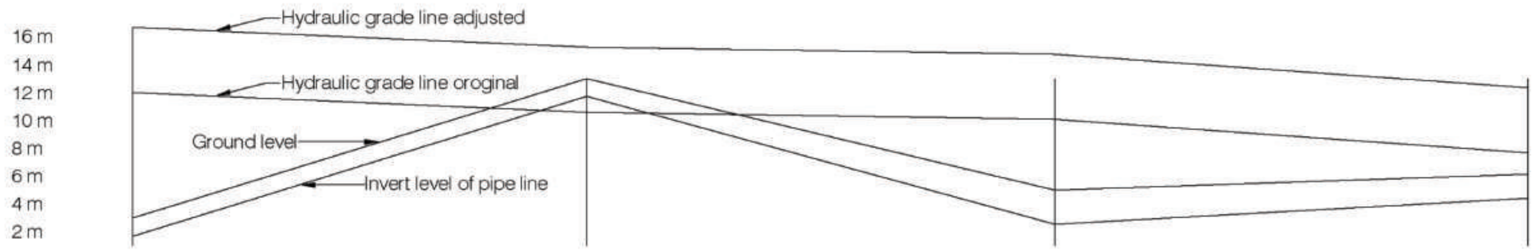
a) The first step is to calculate the friction loss and fittings loss in the “spine” pipe line which will be A-B-C-D with incrementing flows in each segment for a given pipeline diameter by using Appendix 4.4 for each segment and adding up all these from A to D and establish the preferred diameter from 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.

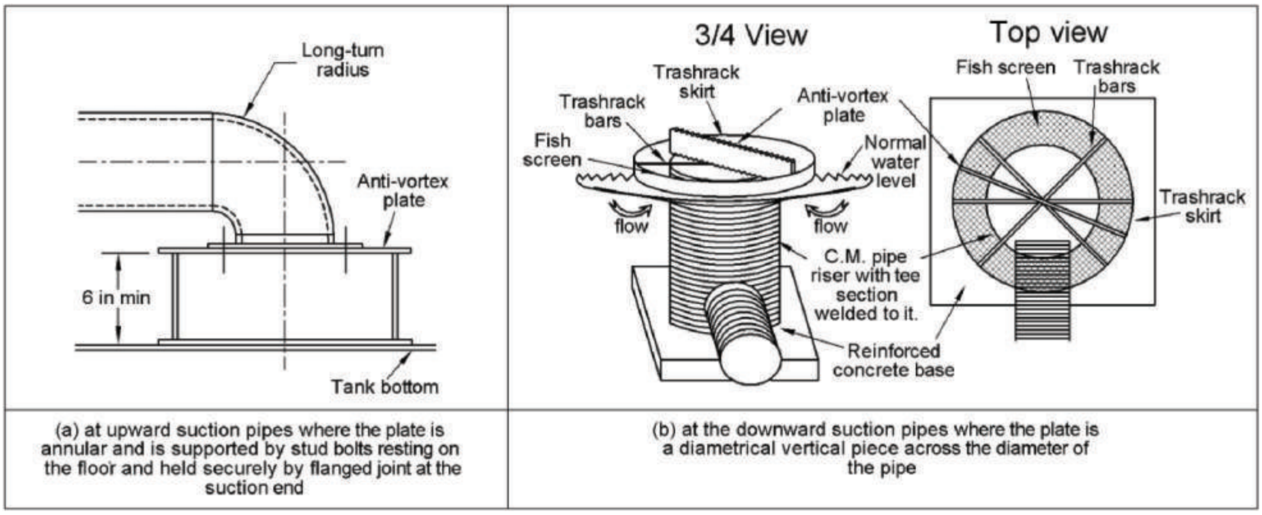


Figure 4.17 Typical anti vortex plate

There are many other variations and can be sourced from the market.