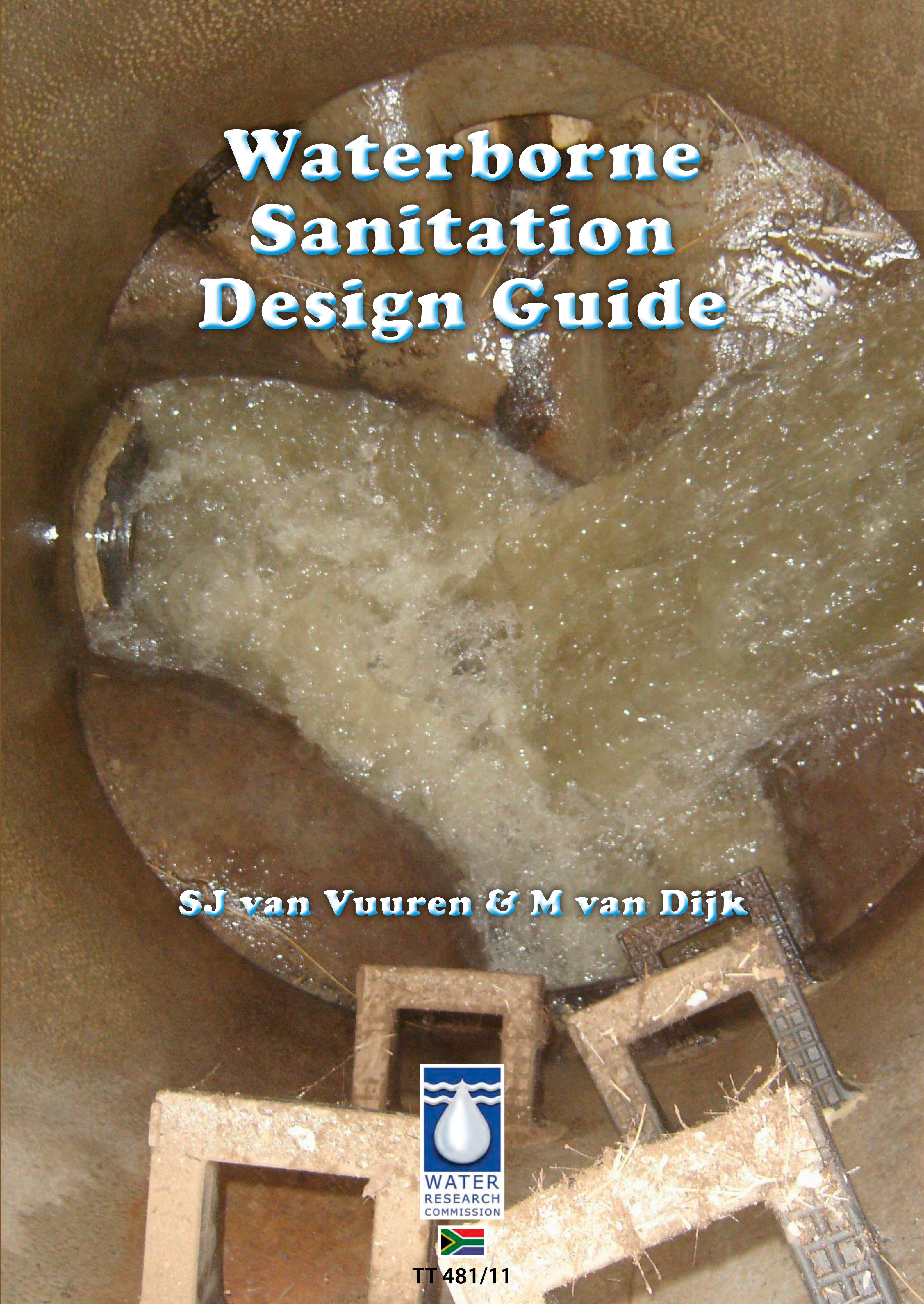

Waterborne Sanitation Design Guide

SJ van Vuuren & M van Dijk

WATER RESEARCH COMMISSION

TT 481/11

WATERBORNE SANITATION DESIGN GUIDE

Report to the

WATER RESEARCH COMMISSION

By

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Executive Summary

The general health of the population improves when people have access to basic clean water supply and sanitation. The safe disposal of human excreta and greywater is vitally important in the control of infectious and other communicable diseases and the design and construction of appropriate sanitation systems is of paramount importance in contributing to the safe disposal of human excreta. However, on its own, the proper planning and construction of sanitation systems does not provide a guarantee that the general health of the population will improve. A holistic approach to health care is required, with the provision of suitable sanitation being just one of the necessary components thereof.

A three-pronged approach was followed to source the information required to produce this guide. At the outset, the authors collaborated with a number of local authorities in South Africa and gathered information regarding the design and operation of their sewer systems. The main concerns raised by the managers of the various sewer networks in South Africa were also noted and were addressed in compiling this report. Secondly, the standards and guidelines used in practice in the design and operation of waterborne sanitation systems were reviewed. Thirdly, many sources of information were consulted and a synthesis of the material was tailored to South African conditions to produce a comprehensive guide on waterborne sanitation systems. In particular, the following documents were heavily relied on: Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development *(CSIR, 2003);* Alternative Sewer Systems (WEF Manual of Practice, 2008)*; the USEPA (1991) manual entitled* Gravity Sanitary Sewer Design and Construction; *and the* Sewer Design Manual *(ASCE, 1982).*

This report summarizes the available knowledge, information and advancements of all waterborne sanitation systems used in South Africa. The objective of the report is to provide a concise guide for the analysis and design of waterborne sanitation systems.

In order to streamline the planning and design process in South Africa a three-tier philosophy is proposed for sewage collection system planning and design. As described by Jacobs and Van Dijk (2009) the philosophy used originates from the field of transport engineering where three different 'solution levels for design procedures' *are documented in the* South African Code of Practice for the Design of Highway Bridges and Culverts (*Department of Transport, TMH 7*)*. Adopting this concept for the planning and design of sewage collection systems leads to three technical tiers. This three-tiered philosophy could be used as a basis to derive a best management practice for sewer system planning and design.*

❶ General information
❷ Detailed design
❸ Specialized/Advanced design

Throughout this guide the information provided is partitioned into these three tiers.

The four main waterborne sanitation systems which are described in this guide are:

- *Conventional gravity sewer*
- *Vacuum sewer systems*
- *Small-bore sewer*
- *Simplified sewerage*

A summary description with advantages and disadvantages of these systems is provided. A technical design criterion for designing each of these systems is given with a worked example guiding the designer through the process to be followed.

To provide further classification and background information photos, videos, software and additional literature were included on the accompanying DVD. Where this icon is shown in this report movie clips, photos, additional literature or software are available in the **SewerAid** *DVD to visually enhance the understanding of the specific concept.*

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Waterborne Sanitation Design Guide

Table of Contents

LIST OF TABLES

LIST OF FIGURES

APPENDICES

GLOSSARY OF TERMS

Term		Description
Aerobic	:	A process in which dissolved oxygen is present.
Anaerobic	:	A process in which dissolved oxygen is not present.
Appurtenance	:	Machinery, appliances, structures and other parts of the main structure necessary to allow it to operate as intended, but not considered part of the main structure.
Attenuation	:	The reduction in magnitude/intensity/concentration of a substance dispersed in a liquid medium.
Average dry weather flow	:	The average non-storm flow over 24 h during the dry months of the year. It is composed of the average sewage flow and the average dry weather inflow/infiltration.
Average wet weather flow	:	The average flow over 24 h during the wet months of the year on days when rainfall occurred on that or the preceding day.
Base flow	:	The portion of the wastewater flow, including inflow and infiltration, that corresponds to the minimum flow recorded in a sewer. It typically equates to the 'minimum night flow' concept in water distribution systems.
Blockage	:	A deposit in a sewer resulting in restriction or stopping of flow.
Branch sewer	:	A sewer that receives wastewater from a relatively small area and discharges into a main sewer.
Bulk main	:	See Collector main.
Bypass	:	A pipe, valve, gate, weir, trench or other device designed to allow all or part of the wastewater flow to be diverted from the usual channels or flow. It provides an alternative route for the wastewater whilst the facility or device is being maintained.
Cesspool	:	A covered watertight tank used for receiving and storing sewage from premises which cannot be connected to the public sewer and where conditions prevent the use of a small sewage treatment works, including a septic tank.
Cleanout	:	An access opening (usually covered or capped) in a wastewater collection system used for inserting tools, rods or snakes while cleaning a pipeline or clearing a blockage. It remains permanently accessible after completion of the drainage installation.
Cleaning eye	:	See Cleanout.
Collection system	:	A network of pipes, manholes, cleanouts, traps, siphons, lift stations and other structures used to collect all wastewater from an area and convey this to a treatment plant.
Collector main	:	In collection systems, this is a larger pipe in which smaller branch and sub-main sewers are connected.
Collector sewer	:	The intermediate sized pipelines that convey the effluent from the reticulation to the main outfall sewers. These are usually in sizes ranging from 150 mm to 450 mm in diameter (PIPES, 2009)

Term		Description
Combined sewer system	:	A wastewater collection and treatment system where domestic and industrial wastewater is combined with storm runoff.
Conservancy tank	:	A covered tank that is used for the reception and temporary retention of sewage and that requires emptying at intervals.
Debris	:	Any material in wastewater found floating, suspended, settled or moving along at the bottom of a sewer. This material may cause blockage or settle out in a sewer.
Detention	:	The process of collecting and holding back stormwater or combined sewage for delayed release to receiving waters.
Discharge	:	The release of wastewater or contaminants to the environment. A direct discharge of wastewater flows from a land surface directly into surface waters, while an indirect discharge of wastewater flows into surface waters by way of a wastewater treatment system.
Diversion structure	:	A type of regulator that diverts flow from one pipe to another.
Domestic wastewater	:	Human-generated sewage that flows from homes and businesses.
Drain	:	A conduit, generally underground, designed to carry wastewater and/or surface water from a source to a sewer; a pipeline carrying land drainage flow or surface water from roads (Stephenson and Barta, 2005).
Drop manhole	:	A mainline or house service line lateral entering a manhole at a higher elevation than the main flow line or channel.
Dry well	:	A dry room or compartment in a lift station, near or below the water level, where the pumps are located.
Effluent	:	Treated water, wastewater or other liquid flowing out of a treatment facility.
Extraneous flow	:	Water entering the sewer from sources other than intended water used and wasted, or leaking, at source (e.g. stormwater and groundwater infiltration). Extraneous flows make up most of the base flow in most sewers.
Exfiltration	:	Liquid wastes and liquid-carried wastes which unintentionally leak out of a sewer pipe system and into the environment via cracks or malfunctioning pipe joints
Force main	:	A pipe that carries wastewater under pressure from the discharge side of a pump to a point of gravity flow downstream. Also called pressure main.
French drain	:	A conventional absorption field that comprises a trench that is filled with suitable material and that is used for the disposal of liquid effluent from a septic tank or wastewater.
Grease	:	In a sewage collection system, grease is considered to be the residues of fats, detergents, waxes, free fatty acids, calcium and magnesium soaps, mineral oils and certain other non-fatty material which tend to separate from water and coagulate as floatables or scum (NEIWPCC, 2003).

Term		Description
Grease trap	:	A device that is designed to cool down incoming hot wastewater to below 30°C, to enable grease and fat to separate from the water and to solidify at the surface level of the wastewater, and that prevents grease and fat from entering the sewer (also referred to as a grease interceptor).
Greywater	:	Wastewater from the bath, shower and possibly the washing machine that is 'less polluted' than waste from the other sources (e.g. the toilet and kitchen sink).
Grit	:	The heavy mineral material present in wastewater such as sand, coffee grounds, eggshells and gravel. Grit tends to settle out at flow velocities of below 0.6 m/s and accumulates in the invert of the pipe.
Grit trap	:	A permanent structure built into a manhole (or other convenient location in the collection system) for the accumulation and removal of grit.
Groundwater infiltration	:	Infiltration of groundwater (that typically enters the sewerage system through pipe defects located below the normal groundwater table).
Gully	:	A pipe fitting that incorporates a trap into which wastewater is discharged and that is normally connected to a drain.
Hydraulic grade line (HGL)	:	The surface or profile of water flowing in an open channel or a pipe flowing partially full. If a pipe is under pressure, the HGL is at the level water would rise to in a small tube connected to the pipe.
Infiltration	:	The ingress or seepage of groundwater into a drain or sewer system through defects in pipes, joints or manholes (Stephenson and Barta, 2005).
Inflow	:	Water discharged into a sewer system and service connections from such sources as, but not limited to, roof leaders, yard and area drains, around manhole covers or through holes in the covers, surface runoff, etc. Inflow differs from infiltration in that it is a direct discharge into the sewer rather than a leak in the sewer itself.
Inspection chamber	:	A chamber not deeper than 1 m and of such dimensions that permanent access may be obtained to a drain without a person being required to enter into such a chamber.
Interceptor sewer	:	A sewer that receives flow from a number of other large sewers or outlets and conduits the waters to a point for treatment or disposal.
Invert	:	The bottom of the inside of a pipe.
Lag	:	An interval of time before additional flow enters the system.
Lateral sewer	:	A sewer that discharges into a branch or other sewer and has no other common sewer tributary to it. It is also sometimes called a 'street sewer' because it collects wastewater from individual houses.
Lift station	:	A wastewater pumping station that lifts the wastewater to a higher elevation usually discharging into a downstream gravity sewer.

Term		Description
Load	:	Any matter transported by the flow in sewers (typically this would be sewage).
Main sewer	:	This is a larger pipe in which smaller branch and sub-main sewers are connected. It may also be called a trunk sewer.
Manhole	:	A chamber of depth exceeding 750 mm and of such dimensions that a person can enter such chamber to obtain access to a drain.
Offset	:	A combination of elbows or bends which brings one section of a line of pipe out of line with, but into a line parallel with, another section.
Outfall	:	The point, location, or structure where wastewater or drainage discharges from a sewer, drain or other conduit.
Outfall sewer	:	A sewer that receives wastewater from a collecting system or from a treatment plant and carries it to a point of final discharge. These are usually from 450 mm in diameter and larger.
Overflow manhole	:	A manhole which fills and allows raw wastewater to flow out onto the street or environment.
Peak dry weather flow (PDWF)	:	The peak non-storm flow during the dry months of the year. It is composed of the peak sewage flow and the peak dry weather inflow/infiltration.
Peak wet weather flow (PWWF)	:	The peak flow during the wet months of the year on days when rainfall occurred on that or the preceding day.
Pig	:	Refers to a poly pig which is a bullet-shaped device made of hard rubber or similar material used for cleaning of sewer lines.
Plumbing	:	The system of pipes and fittings required for the sanitation of a building (to the stand boundary where the plumbing joins the sewer).
Pumping station	:	This is usually an underground structure that the sewage is discharged into. The types vary but in smaller systems these comprise of a wet well, into which the sewage is discharged, and the wet well also houses submersible pumps which pump the sewage to its destination. In a larger station there may be a separate dry well, adjacent to the wet well, which houses the pumps. On some pumping stations the pumps may be housed above ground near the wet well.
Raw sewage	:	Untreated wastewater.
Regulator	:	A structure that controls the flow of wastewater from two or more input pipes (trunk lines) to a single output (usually a larger interceptor line). Regulators can be used to restrict or halt flow, thus causing wastewater to be stored in the conveyance system until it can be handled by the treatment plant.
Relief sewer	:	A sewer built to carry flows in excess of the capacity of an existing sewer.

Term		Description
Reticulation	:	This is the smallest element of a sanitation system and consists of the small-diameter pipelines that convey the effluent from the individual properties and along streets. They are usually in sizes ranging from 100 mm to 225 mm in diameter (PIPES, 2009).
Rising main	:	See Force main.
Rodding eye	:	A permanent access opening to the interior of a drainage installation that permits full-bore access to the interior of a drain for internal cleaning, but does not include an inspection eye or manhole.
Sanitation system	:	In the context of this guide all the components including the pipelines that convey sanitary wastewater away from where it is generated to the outfall works where it is treated and purified before it is discharged into the natural watercourses (PIPES, 2009).
Screen	:	A large sieve used for the purpose of trapping large objects in sewage.
Sediment	:	Solid material settled from suspension in a liquid.
Sedimentation	:	The process of settling and depositing of suspended material carried by wastewater. Sedimentation usually occurs by gravity when the velocity of the wastewater is reduced below the point at which it can transport the suspended material.
Septage	:	In the United States, the partially treated waste stored in a septic tank is called septage.
Septic tank	:	An underground tank used for the deposition of domestic wastes. Bacteria in the wastes decompose the organic matter, and the sludge settles to the bottom. The effluent flows through drains into the ground. Sludge is pumped out at regular intervals.
Sewage	:	Wastewater, soil water, industrial effluent and other liquid waste, either separately or in combination, but excluding stormwater. Usually classified as wastewater derived from human communities – toilet, bathroom and kitchen waste.
Sewer	:	A pipe or conduit that is the property of the local authority and that is used for the conveyance of sewage.
Sewer gas	:	Gas developing in collection sewer lines as a result of the decomposition of organic matter in the wastewater. When testing for gases found in sewers, test for lack of oxygen and also for explosive and toxic gases (NEIWPCC, 2003).
Sewer main	:	A sewer pipe to which building laterals are connected.
Sewer system	:	Collectively, all of the property involved in the operation of a sewer utility. It includes land, wastewater pipes, pumping stations, treatment plants, and general property. It may also be called sewerage or wastewater system.
Sewerage	:	System of piping with appurtenances for collecting, moving and treating wastewater from source to end discharge.
Silt trap	:	See Grit trap. Also called sand trap.

Term		Description
Siphon	:	A pipe or conduit through which water will flow above the HGL under certain conditions. Siphons also called depressed sewers are designed to carry flow underneath an obstruction and to regain as much pressure head as possible after the obstruction has been passed.
Sludge	:	The suspended matter in industrial effluent or sewage remaining after partial drying.
Sludge removal	:	This is the process of removing sludge from treatment systems or tanks and can be carried out manually or automatically.
Soffit	:	The top of the inside of a pipe.
Storage	:	A method for controlling combined sewer overflows by storing the combined sewage until the rainstorm subsides, then releasing it back into the conveyance system to be treated at the usual treatment plant.
Sullage	:	Wastewater emanating from baths, kitchen sinks, laundries and showers.
Surcharge	:	Sewers are surcharged when the supply of wastewater to be carried is greater than the capacity of the pipes to carry the flow. The surface of the wastewater in manholes rises above the top of the sewer pipe and the sewer is under pressure or a head (NEIWPCC, 2003).
Surcharged manhole	:	A manhole in which the rate of wastewater flow entering the manhole is greater than the capacity of the outlet under gravity conditions. When the wastewater level in the manhole is higher than the top of the outlet pipe the manhole is said to be 'surcharged' (NEIWPCC, 2003).
Suspended solids	:	Small particles of organic or inorganic materials that float on the surface of, or are suspended in, sewage or other liquids and which cloud the water. The term may include sand, mud, and clay particles as well as waste materials.
Terminal manhole	:	A manhole that is placed at the upstream end of a sewer and having no inlet pipe.
Trunk main	:	See Collector main.
Trunk sewer	:	This is a larger pipe in which smaller branch and sub-main sewers are connected. It may also be called a main sewer.
Wastewater	:	A community's used water and water-carried solids that flow to a treatment plant. Stormwater, surface water and groundwater infiltration may also be included in the term wastewater. The term 'sewage' usually refers to household wastes.
Waterborne	:	Transported by water.
Wet well	:	A compartment or tank in which wastewater is collected. The suction pipe of a pump may be connected to the wet well or a submersible pump may be placed inside the wet well.

LIST OF SYMBOLS

A	:	Internal sectional area of pipe (m^2)
B	:	Peak-flow rate (ℓ/s)
C	:	The required capacity (m^3)
Df	:	Design-flow rate (ℓ/s)
H_1	:	Elevation of pipe invert at house connection manhole/rodding eye (m)
H_2	:	Elevation of pipe invert at main sewer line connection point (m)
H_f	:	Friction head (m)
H_s	:	Static head (m)
H_v	:	Vacuum head (m)
L_{1-2}	:	Length of pipe between Point 1 and Point 2 (m)
n	:	Manning's roughness coefficient
R	:	Hydraulic radius, for a pipe flowing full, $R = D/4$, where D is the internal diameter of the pipe
Q	:	Flow capacity of pipe (m^3/s)
Q_a	:	Station average flow (ℓ/s)
Q_{dp}	:	Discharge pump capacity (ℓ/s)
Q_{max}	:	Station peak flow (ℓ/s)
Q_{min}	:	Station minimum flow (ℓ/s)
Q_{vp}	:	Vacuum pump capacity (m^3/min)
t	:	System pump-down time (min)
TDH	:	Total dynamic head (m)
V	:	Flow velocity (m/s)
V_{ct}	:	Collection tank volume (m^3)
V_o	:	Collection tank operating volume (m^3)
V_p	:	Piping system volume (m^3)
V_{rt}	:	Reservoir tank volume (m^3)
V_t	:	Total system volume (m^3)
V_{tank}	:	Volume of septic tank (ℓ)
W	:	The width of the tank (m)
ΔS	:	Average slope of pipe (m/m)

ABBREVIATIONS

AC	:	Asbestos cement
ACS	:	Alternative collection systems
ADWF	:	Average dry weather flow
AWWF	:	Average wet weather flow
ASCE	:	American Society of Civil Engineers
CCTV	:	Closed circuit television
CSIR	:	Council for Scientific and Industrial Research
du	:	Dwelling unit
EDU	:	Equivalent discharge unit
IDP	:	Integrated development plan
HDPE	:	High density polyethylene
HGL	:	Hydraulic grade line
LCA	:	Life Cycle Analysis
NPSH	:	Net positive suction head
O&M	:	Operation and maintenance
PE	:	Polyethylene
PVC	:	Polyvinyl chloride
PWWF	:	Peak wet weather flow
RCB	:	Reinforced concrete box
RCP	:	Reinforced concrete pipe
SABS	:	South African Bureau of Standards
SANS	:	South African National Standards
SBGS	:	Small-bore gravity sewer
SDG	:	Small diameter gravity
SFD	:	Single family dwelling
SFS	:	Solids-free sewer
STEG	:	Septic tank effluent gravity
STEP	:	Septic tank effluent pumping
TDH	:	Total discharge head
U.S.EPA	:	U.S. Environmental Protection Agency
VC	:	Vertical curve
VCP	:	Vitrified clay pipe
VIP	:	Ventilated Improved Pit
WRC	:	Water Research Commission
WSDP	:	Water Services Development Plan
WWTW	:	Wastewater treatment works

Waterborne Sanitation Design Guide

1. INTRODUCTION

Historical records include many references to engineering feats undertaken by ancient civilizations to collect and convey water. Archaeological explorations indicate that an understanding of drainage principles existed very early in history. For example, a sewer arch constructed about 3750 B.C. was unearthed in an excavation at Nippur, India. Another excavation in Tell Asmar, near Baghdad, exposed a sewer constructed in 2600 B.C. (OCPA, 1997).

The first sewers in Rome were built between 800 B.C. and 735 B.C., preceding the first aqueduct by about 500 years. Most renowned of these early construction efforts were the aqueducts of Rome. The water carried by these aqueducts was used primarily for drinking. The aqueducts were also used to carry sewage through Rome's main sewer, the Cloaca Maxima (the name means Greatest Sewer). Built in 800 B.C., and constructed mainly of stone masonry and natural cement, the Cloaca Maxima was the first known man-made waterborne method of sewage disposal. After 2 800 years, sections of this stone sewer are still being utilized. Crude, but functional, sewers also existed in the ancient cities of Babylon, Jerusalem, Byzantium and Paris.

As described by OCPA (1997), very little theoretical pipeline technology existed prior to the 19th century. The precursor of the modern formula for relating velocity of flow and head loss due to friction in open-channel flow was developed by Antoine Chezy, a French engineer and mathematician. Principles of sanitation developed by Edwin Chadwick, an Englishman, were refined by engineers of that time and contributed to the design of properly sized and aligned sewers, with adequate facilities for cleaning and maintenance.

Sewage disposal methods did not improve until the early 1840s when the first modern sewer was built in Hamburg, Germany. It was modern in the sense that houses were connected to a sewer system. For the first time, sanitary sewers were separate from storm sewers. Paris officials had begun to design sewers at the start of the 19th century to protect its citizens from cholera.

The modern toilet is widely credited to Thomas Crapper (who was only improving on the original design developed by Sir John Harrington in 1596), who installed one for Queen Elizabeth I in 1880. In the 1820s, the first flush toilet was invented by Albert Giblin, acting as a forerunner to today's modern cistern.

In South Africa the first waterborne sanitation system, with sewers, was used in the Great Karoo town of Matjiesfontein, founded in 1884 by a Scottish man named James Douglas Logan. The first flushing toilet was installed in his home.

The sanitary sewer system is a major capital investment made by a community. The system's function is only vaguely recognized by the public due to its underground installation, except for the manhole covers or when the system doesn't function properly.

Sanitary systems are essential to protect the public health and welfare in all development areas. Every community produces wastewater of domestic, commercial and industrial origin. Sanitary systems perform the vitally needed functions of collecting these wastewaters and conveying them to points of treatment and disposal (ASCE, 1982).

It is generally accepted that the general health of the population improves when people have access to basic clean water supply and sanitation. The safe disposal of human excreta is vitally important in the control of infectious and other communicable diseases. The construction of appropriate sanitation systems is of paramount importance in contributing to the safe disposal of human excreta.

However, the proper planning and construction of these sanitation systems alone does not provide a guarantee that the general health of the population will improve. A holistic approach to health care is required, with the provision of suitable sanitation being just one of the necessary components thereof. Sanitation is a complex system of interrelated factors and is successful when the factors affecting the health and social organization of the community are effectively linked. A sanitation system is deemed suitable when it is:

- Reliable
- Acceptable
- Appropriate
- Affordable

In the late 1960s, the cost of conventional gravity systems in smaller communities was found to be high compared to the cost of treatment and disposal. According to WEF (2008) the capital cost of a conventional sewage collection system was averaging almost four times the cost of treatment. The operation and maintenance costs followed similar trends because of the greater number of pump stations required per unit length of pipe, owing to the increased lengths of pipe needed to service these less densely populated areas. In this guide some of the more commonly used alternative collection systems (ACS) such as vacuum, small-bore and simplified sewerage systems are also described and guidelines are provided for the design of these systems. Because of their relative newness, all ACS types have shown lack of proper design, installation, management, use and application. Some engineers might be hesitant to recommend these new technologies and thus the purpose of this guide is to provide sufficient information on all available technologies and useful tools to assist engineers in overcoming these concerns and to guide them in selecting, planning and designing the most appropriate system to solve existing wastewater problems and reduce the cost of wastewater management for new developments.

In order to develop a guide for the design and operation of waterborne sanitation for South Africa a good understanding of the existing waterborne sanitation standards and specifications is required. A number of local authorities were visited and data gathered in order to determine the various standards applicable throughout South Africa.

Information has been synthesized from a wide variety of sources and tailored to South African conditions.

Some existing sources were incorporated when compiling this guide. These include the documents *Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development* (CSIR, 2003); *Alternative Sewer Systems* (*WEF Manual of Practice*, 2008); the USEPA (1991) manual entitled *Gravity Sanitary Sewer Design and Construction;* and the *Sewer Design Manual* (ASCE, 1982).

This guide provides a complete overview of all waterborne sanitation systems used in South Africa.

2. PLANNING CONSIDERATIONS AND POLICIES

The health, social, and environmental benefits of improved sanitation are maximised when sanitation is planned for and provided in an integrated way with water supply and other municipal services. The central mechanism to achieve integrated planning and development is the municipality-driven Integrated Development Planning (IDP) process (of which the Water Services Development Plan (WSDP) is a component).

There are critical linkages between the provision of health and hygiene education and sanitation services, water supply services, solid waste management and housing. It is, for example, not feasible to propose reticulated waterborne sewer systems for dispersed settlements, nor practical to propose on-site full pressure water supply when there is no adequate water resource capacity.

Uncoordinated planning is largely responsible for the current lack of consistency in the sanitation sector.

Integrated planning with an increased emphasis on a '*package of services*' approach will allow for more realistic decisions to be taken with regard to technical options and affordability (DWAF, 2001). This will assist in ensuring that development of the various types of infrastructure and the provision of health and hygiene awareness and education takes place in a more co-ordinated and more sustainable way. The focal mechanism for achieving integrated planning is the municipality-driven IDP process.

Selection of the sanitation system is inter-dependent on other services and the full range of sanitation needs within the community. This could range from on-site systems (chemical toilets, VIP, septic tanks) to full waterborne systems (conventional gravity, small-bore, etc.). In other words, the sanitation package must not only include the households, but must also consider the needs of institutions such as schools, places of worship, bus-stops, taxi ranks, etc.

There is a wide range of technical options to choose from. These range from various improved latrines, septic tanks, composting latrines to full waterborne sanitation systems.

The choice of sanitation system should review (DWAF, 2001):

- The affordability to the household
- Operation and maintenance requirements
- Sustainability
- Improvements to health
- Compliance with environmental protection regulations
- The ability of community-based contractors to implement

Johannesburg Water has adopted a sanitation policy that recommends the use of a number of generic sanitation technologies (Johannesburg Water, 2007; Burke, 2002). These may be tailored to meet the affordability and convenience of the customer. According to Johannesburg Water this policy adopts the principle that the level-of-service of water supply should be consistent with the applicable sanitation technologies.

Table 2.1 outlines the generic water and sanitation service-packages with those in bold representing the preferred options for the various service levels.

Table 2.1: Generic water and sanitation service-packages (Johannesburg Water, 2007)

Service package	Sanitation option	Water supply level
Emergency or temporary services	**Communal VIP** (temporary < 12 months measure) Chemical toilets should be avoided, except in emergency situations)	**Communal tank or standpipe** Water tankers may be used under emergency conditions
Basic level	**Household VIP** (on-plot dry) Composting-desiccating systems may be used where the advantages and community acceptance are proven	**Communal standpipe** Yard connections may be considered for low population density
Low level	**LOFLOS** (on-site low flush)	**Low-volume yard connection** (such as yard tanks) Communal standpipe
Intermediate level	**Shallow sewerage** (off-site medium flush) Small-bore sewerage may be considered	**Yard connection** House connection may be used if this is affordable
High level	**Full-bore sewerage** (off-site full flush) Septic (or conservancy) tanks may be used in sparsely populated areas. Small-bore sewerage may be considered.	**Yard or house connections** These may range from trickle feed to full-pressure systems, depending upon affordability and water availability

Planning a wastewater collection system requires the following steps:

- Evaluate and select potential collection systems
- Prepare the layout of the collection system
- Estimate future population and wastewater flow
- Perform a preliminary design (pipe sizes, interceptor tanks or vacuum stations, etc.) for each of the potential alternative systems
- Determine the cost and conduct a Life Cycle Analysis (LCA) to compare the alternatives

This would place the designer in a position to make a decision on what the appropriate sanitation system for the specific application would be.

Sewage collection system planning is regarded as the systematic process of planning a sewer network in terms of cost, applicability, hydraulic and operational performance while subjecting it to existing and potential future development loading. The approach followed in planning sewer systems in South Africa is usually one of *ad hoc* application of whatever level of service is considered necessary, affordable or the system that the designer feels comfortable with designing.

In order to streamline the planning and design process in South Africa a three-tier philosophy reflected in this guide is proposed for sewage collection system planning and design. As described by Jacobs and Van Dijk (2009) this philosophy used originates from the field of transport engineering where three different '*solution levels for design procedures*' are documented in the *South African Code of Practice for the Design of Highway Bridges and Culverts* (TMH 7) (Department of Transport, 1986). Adopting this concept for the planning and design of sewage collection systems leads to three technical tiers. This three-tiered philosophy could be used as a basis to derive a best management practice for sewer system planning and design.

❶ General information
❷ Detailed design
❸ Specialized/Advanced design

Throughout this guide the information provided will be partitioned into these three tiers (levels).

Level 1 – General information ❶
The first tier is termed Level 1 and comprises presenting of the most basic information and design rules. The approach is intended to provide information on planning and design for use in cases where limited technical skill is available, or the scope of work is relatively small with negligible risk. This approach is sufficient only in smaller municipalities and small towns with limited sewer infrastructure.   Very often Level 1 is dominated by minimum requirements for some parameters rather than hydraulic considerations. A typical example would be use of a minimum pipe diameter without conducting any hydraulic analysis, being driven simply by the required size needed for rodding and to prevent clogging.

Level 2 –Detailed design ❷
Level 2 entails a more sophisticated approach incorporating design theories that take into account the hydraulics of system elements, requiring a basic analysis of the system or parts thereof. This level will utilize typical design parameters and standards and apply these in the planning and design of the sewage collection system.

Level 3 – Specialized/Advanced design ❸
Level 3 is the most advanced and requires advanced skill and software tools to conduct specialist and detailed analysis of components and/or the sewer distribution system. The planning and design for Level 3 would be, for instance, where a siphon is designed which requires a detailed hydraulic analysis of the various flows through the siphon systems ensuring cleaning velocities on the upward limb and the calculation of head losses through the inlet and outlet structures. The planning, design and analysis of complicated sewer systems with a variety of users and discharge patterns would also fall under Level 3.

A Water Research Commission (WRC) project entitled *Sewer System Planning Made Simple – For Small Local Authorities* (WRC Report No. 1828/1/10) would be a good starting point to understand the fundamental principles relating to sewer planning with useful tools for the managers/engineers to assist in the planning and design process (Jacobs et al., 2010). In the planning of a sewer distribution system the designer embarks on the selection of a layout by selecting an outlet (defined by the lowest point), determining the tributary area, locating trunk and main sewers and determining the need for and location of pumping stations and rising mains. Preliminary layouts are usually based on topographic maps and other pertinent information. The outlet is located according to the circumstances of the particular project and this could be the treatment works, a pumping station or a connection to a trunk/main sewer.

Drainage district boundaries usually conform to watershed or drainage-basin areas. Trunk mains and interceptor sewers are normally located at the lower elevations in a given area.

Reassessment/modelling of a sewer system are required when proposed development intensifies the land use. The following three scenarios must be reviewed:

- Existing Condition – to identify existing deficiencies in the system
- Existing Condition with Proposed Development – to identify additional deficiencies created by the proposed development
- General Plan Build-Out Condition – to identify the ultimate pipe size for improvements

Sewer modelling is used to identify the specific project's impact on the rest of the sewer system. Development in areas with a deficient sewer downstream could be restrictive. Where uses are discontinued on a property to allow for new development, new development up to the sewer generation rate of the previous use on the property will be allowed in sewer-deficient areas.

A developer should or could be instructed by the Local Government to make the required improvements to the sewer system at his/her own cost and request a reimbursement agreement to recover a portion of the costs from other developments that tie into the system and benefit from the improvements.

A general planning-level overview of all collection alternatives is provided in **Table 2.2**. Specific local conditions may defy the table entries when detailed design is undertaken.

Table 2.2: General overview of collection alternatives (adapted from WEF, 2008)

Issue	Conventional sewer	Vacuum sewer	Small-bore		Simplified sewer
			STEG	STEP	
Annual inspections—suggested preventative maintenance	No	Yes	Yes	Yes	Yes
Septage pumping from onsite septic tank, as required	No	No	Yes	Yes	No
Onsite electrical connection required		No	No	Yes	No
Discharge wastewater characteristics Strength (Why?) Flow (Why?) Corrosion/odour potential (Why?) FOG	 Medium Diluted High High infiltration and inflow Low to high ƒ(rising mains) Medium	 High Undiluted Low Low infiltration and inflow Low Aeration in volatile solids High	 Low STE Low Low infiltration and inflow High Sulphides from septic tank Low	 Low STE Low Low infiltration and inflow High Sulphides from septic tank Low	 Medium Diluted High High infiltration and inflow Low to high ƒ(rising mains) Medium
Terrain effects Discharge above source Discharge below source Undulating terrain	 Yes, with lift stations Yes Yes, with GP	 Yes Yes Yes, with GP	 No Yes Yes, use a mix of both	 Yes No	
Discharge to Conventional sewer # Biological treatment Constructed wetlands	 Yes Yes Yes	 Yes Yes* Yes*	 Yes Yes Yes	 Yes Yes Yes	 Yes Yes Yes

Notes: # Extra design considerations
* Pretreatment needed

The design engineer should consider the community's choice of collection system type during the planning stages of a sewage collection system project. The final choice should be based on the results of a cost-effectiveness analysis although sometimes this is purely based on the preference of the client or community. Where the terrain is applicable to a gravity system, the engineer may not even consider other systems. However, while gravity systems may appear to be less costly in these situations many factors considered collectively may result in one of the other alternative systems actually being the proper choice.

3. WATERBORNE SANITATION SYSTEMS

3.1 Introduction

The objective of this chapter is to reflect material applicable to the study focus and to recapitulate the trends and procedures currently used.

The vision of providing sanitation in South Africa as set out in the Draft White Paper on Water Services (DWAF, 2002a) reads as follows:

Water is Life, Sanitation is Dignity
- *All people living in South Africa have access to adequate, safe and affordable water and sanitation services, practise safe sanitation and use water wisely.* - *Water supply and sanitation services are sustainable and are provided by effective and efficient institutions that are accountable and responsive to those whom they serve.* - *Water is used wisely, sustainably and efficiently in order to promote economic growth and reduce poverty.*

Water and sanitation are interdependent and interrelated in terms of sanitation provision in South Africa. Although waterborne sanitation systems are usually perceived as the highest level of service, these systems may not necessarily be sustainable due to the lack of water resources.

Both conventional waterborne sanitation systems and bucket systems are expensive and need well-run organisations to make sure they are safe for users and the environment. Because of this, other sanitation systems have been used in developing areas, and there is currently a range of systems that can be used in different situations.

The selection of the most suitable sanitation system is not a simple decision to be made only by engineers. The National Sanitation Policy (DWAF, 1996), reflects a number of important points to consider when selecting a system:

- *Is the proposed system affordable to the user, service supplier and the government?*
- *What kind of organisation will be needed? How complicated must it be?*
- *What will be the risks to the environment?*
- *Is it acceptable to people (bearing in mind the cost to them)?*
- *What is the water supply like? Is it adequate? Can it support the proposed sanitation system?*
- *Will the system be reliable in this situation?*
- *How much of the system can be built and maintained by local people using materials locally available?*

- *Can or should it be upgraded, when people can afford a more expensive system?*
- *Does the housing layout make some systems more difficult to build or operate/maintain?*

❶

Improving household sanitation is not something which happens once in a lifetime. It is a continuous process in which a family should be able to obtain the type of sanitation for which it is willing to pay and use the system correctly to ensure proper functioning thereof.

The National Sanitation Policy (DWAF, 1996) is the point of departure for successfully interpreting and implementing the *Water Services Act* (Act 108 of 1997). *Its objective is to ensure that in the provision of sanitation:*

- *end-users play a central role in all decisions which affect them;*
- *the service is appropriate to the environmental conditions in an area;*
- *the service is sustainable and cost effective to the users, on a long term basis; and*
- *the service results in improved hygiene and environmental health conditions*

A sanitation service needs to offer a complete, holistic and developmental approach to the community, which includes health and hygiene improvements, environmental health considerations and infrastructure development.

3.2 Classification of sanitation systems

One way of classifying sanitation systems has been to distinguish between the use of water and the disposal procedure for the excreta (CSIR, 2003). This classification reflects the four groups shown in **Table 3.1**.

Table 3.1: Classification of sanitation systems (adapted from CSIR, 2003)

Use of water	Conveyance of excreta to a wastewater treatment works	Treatment of excreta on site
No water added	**Group 1** Bucket latrines; chemical toilets	**Group 2** Unimproved pit latrines; ventilated improved pit latrines; reed odourless earth closet latrines; ventilated improved double pit latrines; ventilated vault pit latrines; continuous composting latrines; anaerobic digesters; biological/electric toilets
Water added	**Group 3** Conservancy tank systems; shallow sewers; conventional waterborne sewerage systems; vacuum sewerage	**Group 4** Conventional septic tank systems; aqua-privies; biogas digesters; solids-free sewer systems/small-bore sewer

A further classification to distinguish between different sanitation systems is also to include whether the waste treatment is aerobic or anaerobic (CSIR, 1991).

As described in the National Sanitation Policy (DWAF, 1996) the hierarchy of **adequate sanitation** options can be viewed in different ways (**Table 3.2**). When viewed from the point of the user, it is generally associated with progressively higher costs (initial and ongoing), greater use of water for flushing and improved convenience and status. If viewed from the point of the organisation responsible for managing the system, it is associated with both higher costs to be recovered from users, and increasing operations and maintenance complexity (see **Table 3.2**).

Table 3.2: Hierarchy of adequate sanitation technologies (adapted from DWAF, 1996)

System	Degree of complexity	Approximate water (ℓ/flush)*
VIP	Simple, but needs proper design and construction; periodic desludging or relocation	Nil
LOFLOS	Some types use mechanical flushing; soakaway or soakpit needs proper design; periodic desludging	0.5 to 1.0
Septic tank	Soakaway needs proper design and construction; periodic desludging	6 to 15
Solids-free sewerage	Needs reticulation and treatment works; periodic desludging	3 to 15
Simplified sewerage	Needs reticulation and treatment works	6 to 15
Conventional sewerage	Needs reticulation and treatment works	6 to 15
Vacuum sewerage	Requires vacuum pump station and treatment works	6 to 15

* Please be careful of marketing ploys where the water use is used to justify the selection of a particular sanitation option. Water use is primarily dependent on the household.

This guide will provide the necessary information and guidance in the design of the most appropriate system, but should be read in conjunction with the *Waterborne Sanitation Operation and Maintenance Guide* (Van Vuuren and Van Dijk, 2011).

3.3 History of design standards and criteria

In South Africa most of the sewerage infrastructure has been designed in accordance with the relevant standards, applicable local requirements and municipal by-laws pertinent to the specific development. The changes in design standards were usually brought about by changes in the applicable technologies and political or socio-economic changes (Stephenson and Barta, 2005).

A summary of the main standards applicable to waterborne sanitation design, are provided in **Table 3.3**.

Table 3.3: Design standards of sanitation systems*

Title	Authors	Description	Year
Guidelines for the Provision of Township Services in Residential Townships ('Blue Book')	CSIR for Department of Community Development	The document provides practical guidance for the design with relevant formulae and descriptions	1983
Towards Guidelines for Services and Amenities in Developing Communities ('Green Book')	CSIR	Mostly used as a planning guide	1988
Proposed Development Guidelines for Housing Projects ('Brown Book')	Cape Provincial Administration	A design guide	-
Water Supply and Sanitation in Developing Countries	Institution of Water Engineers and Kawata	A companion to the *Manual of British Water Engineering Practice* used as British Standard	1983
RSA/KwaZulu Guidelines	RSA/KwaZulu Development Programme (RKDP)	It provides design standards for several different systems founded primarily on the *Guidelines for the Provision of Township Services in Residential Townships* and was developed for Durban and Pietermaritzburg metropolitan areas.	1990
Guidelines for the Provision of Engineering Services and Amenities in Residential Township Development ('Old Red Book')	CSIR	A manual widely used in the design and development of municipal services. The manual was based on the *Guidelines for the Provision of Township Services in Residential Townships* and *Towards Guidelines for Services and Amenities in Developing Communities*.	1994
Guidelines for Human Settlement Planning and Design ('New Red Book')	CSIR supported by the SA Department of Housing	A design manual widely used in the design and development of municipal services although lacking in technical guidance and definite design criteria.	2000 (Revised in August 2003)

Note: * Adapted from Stephenson and Barta (2005)

Some local authorities would in general prescribe the *Guidelines for Human Settlement Planning and Design* (CSIR, 2003) adding some additional criteria and guidelines applicable to their specific needs for a waterborne sanitation system. A few of these guidelines and other standards are listed in **Table 3.4**.

Table 3.4: Other relevant guidelines and standards

Title	Authors	Year
Standardized Specification for Civil Engineering Construction Section LD: Sewers. SANS 1200 LD:1982 (SABS, 1982a)	The South African Bureau of Standards	1982
Code of Practice for Use with Standardized Specifications for Civil Engineering Construction and Contract Documents Parts 2 to 5 Section LD: Sewers. SANS 10120-2 to 5 (SABS, 1982b)	The South African Bureau of Standards	1982
SANS 10252-2:1993 - *Water Supply and Drainage for Buildings, Part 2: Drainage Installations for Buildings* (SABS, 1993)	The South African Bureau of Standards	1993
General Principles and Guidelines for Design and Construction of Water and Sanitation Systems in the City of Tshwane Metropolitan Municipality Area	City of Tshwane Metropolitan Municipality (CTMM)	2007
City of Tshwane Metropolitan Municipality: Sanitation By-Laws (CTMM, 2003)	City of Tshwane Metropolitan Municipality	2003
Guidelines and Standards for the Design and Maintenance of Water and Sanitation Services (Draft)	Johannesburg Water (Pty) Ltd	2007
Guidelines for the Design of Foul-Water Sewers	Ethekwini Water and Sanitation	1987
Design Standards for Waterborne Sanitation	City of Cape Town	2007

These guidelines and standards have assisted in the planning and design of numerous waterborne sanitation systems in South Africa. A summarized description of all of the waterborne sanitation systems is provided in the paragraphs that follow with more detailed design aspects covered in **Sections 5 to 8**.

3.4 Conventional gravity sewer system

3.4.1 General description

A conventional gravity sewer system requires a water supply connection, a sewer reticulation system, bulk sewer lines and a wastewater treatment works. Water is used to flush the excreta from the toilet into the sewer line. A water seal is created in the toilet pan to prevent odours from entering the house, see **Figure 3.1**. The sewer reticulation system is usually situated outside the erf boundary.

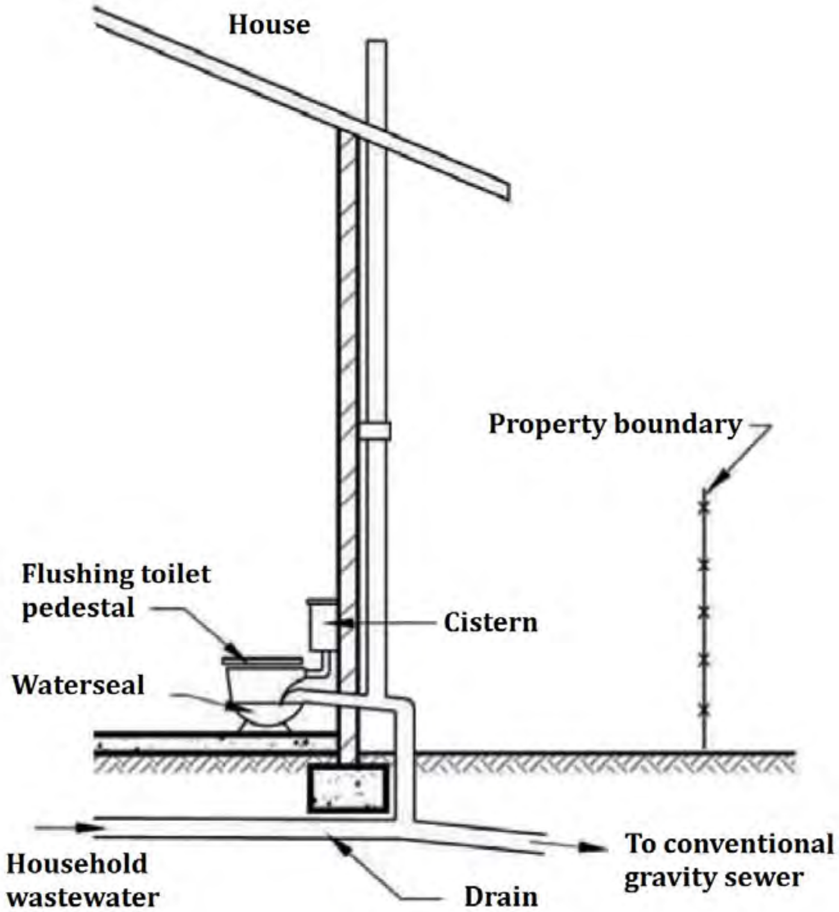

Figure 3.1: Conventional gravity sewer system (CSIR, 2003)

The conventional gravity sewer network connection system which is commonly found in developed urban areas is known as the '*Conventional Gravity Sewers*'. A main gravity sewer distribution network in a developed area conveys accumulated sewage via house connections to a main disposal facility (i.e. sewage treatment works/plant). The connection pipe diameter varies between 100 mm and 150 mm, whilst the main sewer line varies from 300 mm to larger diameters, depending on flow volumes accumulated upstream. The main sewer line forms part of a network of sewer lines that gravitates to a wastewater treatment plant (Little, 2004).

A typical house connection will consist of a pipe connected directly from the house to the main sewer pipe. No interceptor/septic tank or vacuum facility is required between the house and the collector or main sewer. All sanitation and sewage effluent is disposed of directly from the house through the single connection pipe into the collector or the main sewer line.

Figure 3.2 is a schematic representation of a typical gravity sewer network connection system.

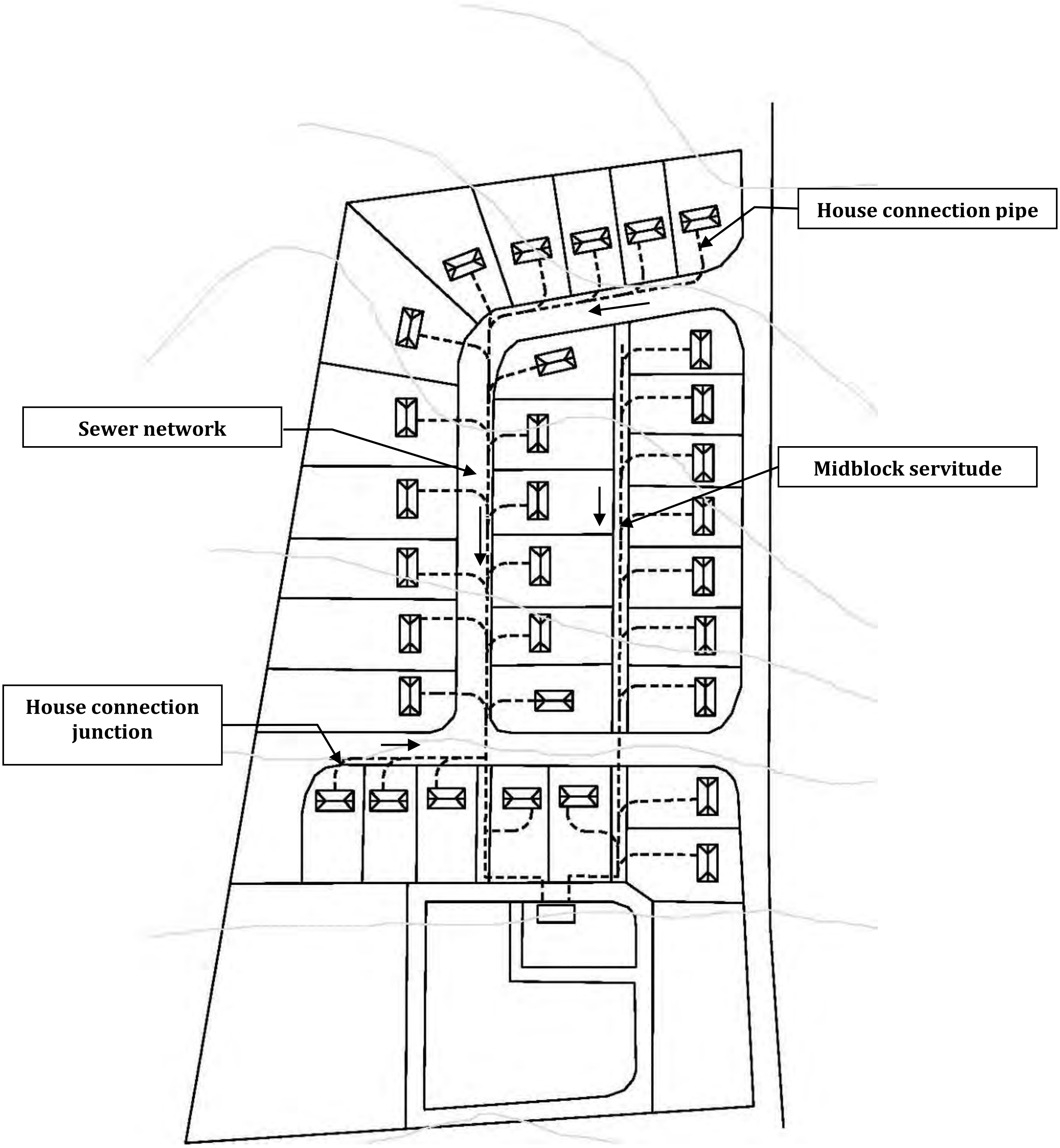

Figure 3.2: Typical gravity sewer network

3.4.2 Technical design criteria

The components of a conventional gravity sewer system are:

- *Pedestal with flush mechanism*

  The standard flush volume in South Africa is between 10 ℓ and 15 ℓ per flush. In Europe and SA the flush volumes have been reduced to 6 ℓ per flush (Palmer Development Group, 1993), with dual flush systems. In the *Guidelines for Human*

Settlement Planning and Design (CSIR, 2003), these standard flush volumes are indicated to be usually between 8 ℓ and 9 ℓ per flush and it is indicated that research has shown that the system is not adversely affected if flushing volumes are reduced.

Figure 3.3 illustrates a typical flushing toilet (cistern, flushing toilet pedestal or pan).

Figure 3.3: Typical flushing toilet (dual flush)

Once the toilet has been flushed the sewage flows through the on-site sewer connections. The other household wastewater pipes are also usually connected to the on-site sewer system and assist in the cleaning thereof by flushing the solid matter further down the system.

On-site sewer connection

The on-site sewer connection is usually designed with a minimum diameter, minimum slope and minimum cover or depth to link up with the main sewer line outside the erf boundary (see **Figure 3.4**).

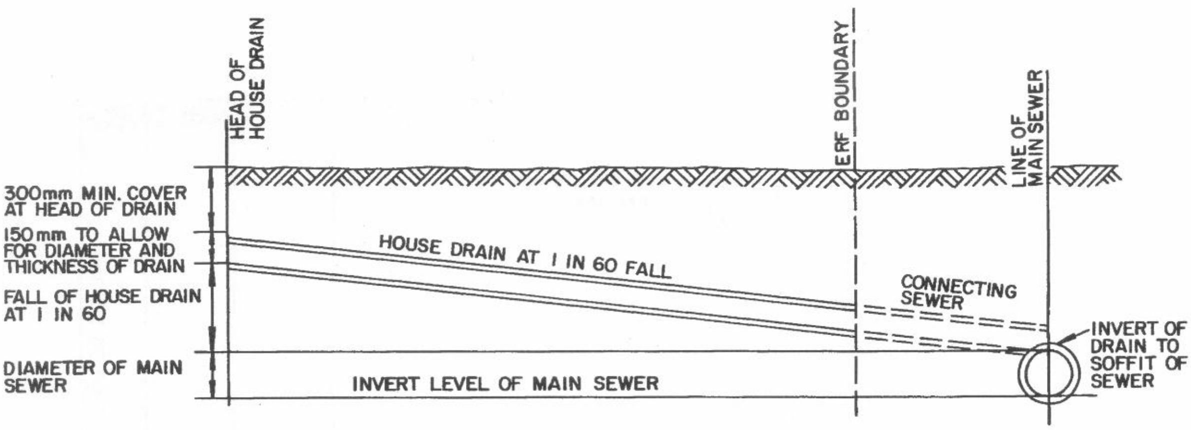

Figure 3.4: Diagrammatic illustration of elements in design of depth of main sewer to accommodate house connections (SABS, 1982a)

Internal reticulation (municipal reticulation)

This is the sewer reticulation laid within the residential area. The sewers are either laid in the streets (pavements) or 'mid-block' (which is at the back of the residential properties). Access to mid-block sewers for maintenance purposes may be difficult for the local authorities.

❶

Sewers are usually laid in straight lines and constant grades between manholes or could have a curved alignment. Steep drops in sewers should preferably be avoided but could be accommodated by connecting a number of 1/16 (6.25°) bends.

It is recommended that the internal sewer reticulation be laid at a depth to provide cover of 0.6 m to 1.4 m depending on the location (in servitudes or in sidewalks) (CSIR, 2003). Deviation from this recommendation is allowed when, for instance, structurally stronger sewer pipes are used or a concrete slab is placed over the pipe or additional earth filling is placed over the pipe for protection.

Sewer pipes are usually laid according to the requirements of SANS 1200 L:1983 (SABS, 1983a), SANS 1200 LB:1983 (SABS, 1983b) and SANS 1200 LD:1982 (SABS, 1982a) (trenching, bedding and backfilling).

Design flow

The design flow is usually based on the type of residential unit drained, potential infiltration, peak factors and attenuation in the network. Stephenson and Barta (2005) indicate that dry weather flow (DWF), average dry weather flow (ADWF), peak dry weather flow (PDWF) and peak wet weather flow (PWWF), as well as estimates of groundwater infiltration and stormwater inflows should be accounted for in determining the capacity of a sewer system.

DWF is calculated by taking the population served multiplied by the average domestic wastewater contribution plus infiltration plus the anticipated industrial effluent discharged into the sewer.

The 'Red Book' (CSIR, 2003) bases its average daily dry weather flow (ADWF) per single-dwelling unit on the income groups (higher, middle and lower). By adding a peak factor which is dependent on the population served (2.5 for a population of 1 500) and a typical infiltration rate of 15% a design flow could be determined for each of the income groups:

$Q_{design} = 0.0167\ \ell/s \cdot du$ for lower income group
$Q_{design} = 0.0250\ \ell/s \cdot du$ for middle income group
$Q_{design} = 0.0333\ \ell/s \cdot du$ for higher income group

Municipalities usually develop their own design standards as shown in **Table 3.5** for the City of Tshwane Metropolitan Municipality (from Stephenson and Barta, 2005), and the designer may be required to use these values.

Table 3.5: Design sewage outflows for urban sanitation systems

Item	Zoning/category	Measuring unit/day	Design sewage outflow
1.	**RESIDENTIAL**		
1.1	Low cost housing – erf up to 250 m^2	kℓ per erf	0.6
1.2	Small sized erf up to 500 m^2	kℓ per erf	0.7
1.3	Medium sized erf up to 1000 m^2	kℓ per erf	0.8
1.4	Large sized erf up to 1 500 m^2	kℓ per erf	0.8
1.5	Extra-large erf in excess of 1 500 m^2	kℓ per erf	0.8
1.6	Cluster housing up to 20 units/ha	kℓ per unit	0.7
1.7	Cluster housing up to 40 units/ha	kℓ per unit	0.6
1.8	Cluster housing up to 60 units/ha	kℓ per unit	0.6
1.9	High-rise flats (± 50 m^2 per unit)	kℓ per every 50 m^2	0.6
1.10	Guest and boarding houses, hostels, hotels, retirement centres and villages, orphanages, etc.	kℓ per 100 m^2 development	0.9
1.11	Agricultural holdings (house plus out building)	kℓ per holding	1.4
2.	**BUSINESS/INDUSTRIAL DEVELOPMENT**		
2.1	General business with an FSR	kℓper 100 m^2	0.8
2.2	Warehousing (including up to 20% offices)	kℓ per 100 m^2	0.4
2.3	Industrial (dry)	kℓper 100 m^2	0.3
2.4	Industrial (wet)	kℓ per 100 m^2	Specific
2.5	Garage or filling station	kℓ per 100 m^2	1.0
2.6	Car wash facility	kℓ per wash bay	10.0

Similar to the information provided in **Table 3.5** design flows for other developments are also available such as churches and schools, etc.

Main design guidelines

Some other design guidelines for conventional waterborne sewerage systems are:

- There are minimum and maximum flow velocities with typical values ranging from ±0.7 m/s to 2.5 m/s, respectively.

- There are recommendations regarding the maximum and minimum flow depths. The design guidelines in the 'Red Book' (CSIR, 2003) indicate that sewers should be designed to flow full during peak flows although making an allowance of 15% for infiltration of stormwater. Some local municipal guidelines do sometimes design for a maximum flow depth of between 70% and 80% of the pipe diameter (to have surplus capacity for stormwater).

- The minimum sewer diameter is usually at least 150 mm (absolute minimum is 100 mm).

- The minimum sewer gradient is indirectly specified since the minimum full-bore velocity is prescribed, see **Table 3.6**. In general the sewers could follow the slope of the ground (except if it is horizontal or adverse) provided that the minimum full-bore velocity is maintained and the hydraulic design capacity is sufficient.

Table 3.6: Minimum sewer gradients

Internal sewer diameter (mm)	Minimum gradient
100	1:120
150	1:200
200	1:300
225	1:350
250	1:400
300	1:500

A costing analysis is usually carried out to determine whether or not it would be beneficial to increase the diameter and thus reducing the excavation required, since the sewer can now be placed at a flatter gradient.

Manholes

Manholes are placed along the sewer reticulation system to allow for effective maintenance of the sewer system. Manholes can be either precast concrete segments, constructed with brickwork or a plastic unit depending on the specification of the client.

- Manholes are usually placed at all junctions and/or changes of grade and/or direction. In cases where manholes are placed within flood plains the manholes should be raised to a level above the 1:50 year flood level to prevent ingress of flood water. Manholes are placed a maximum of 150 m apart where the local authority has power rodding machines, or 100 m apart where only hand-operated rodding equipment is available.

- The size of the manhole shafts and chambers should be designed to allow for easy access and working space. Depending on the shape (circular or rectangular) minimum internal dimensions are prescribed, shown in **Table 3.7**. The aim is to provide sufficient space for a person to work in.

Table 3.7: Minimum manhole dimensions (internal)

Shape	Shaft	Working chamber
Circular	750 mm	1 000 mm
Rectangular	610 mm	910 mm

- Benching in the manhole is required to allow a maintenance worker to easily stand safely on such benching while working in the manhole and for hydraulic reasons to prevent deposition (self-cleansing).

Figure 3.5: Typical manhole with benching (circular)

- *Connector service*

  These would be described as the bulk/main/outfall sewer lines and are used to transport the sewage from the residential area to the wastewater treatment works (WWTW). It might be necessary to pump sewage if no route is available for the sewage to flow under gravity to the WWTW. Most of the guidelines and criteria as provided for the internal reticulation will still be applicable for the connector services. The bulk sewer line will, however, be designed hydraulically and not just using a prescribed minimum self-cleansing velocity.

3.4.3 Construction

Herewith a summary of the simplified construction process of a conventional gravity sewer system:

Step 1 - Surveying and pegging of route. This entails the physical marking of the route. Establish line and grade for pipe and appurtenances. Verify location and elevation of manholes and other sewer components.

Step 2 - Excavation of trenches to various depths, to the correct slope. Additional trimming, clearing, identification of obstructions and preparation of the trench foundation and bedding are required.

Step 3 - Main and lateral gravity sewer pipes are laid (installed and joined) to the correct grade. Pipes should form a sewer with a smooth, uniform invert.

Step 4 - Backfill and compact around the installed main and lateral gravity sewer lines.

Step 5 - Construction of manholes and other appurtenances, as well as making connections between manholes and the sewer pipes.

Step 6 - Restoration and testing – The sewer route will be restored to the pre-construction original conditions and the sewers will inspected and tested.

Figure 3.6 depicts the construction of the on-site sewer connection whilst **Figure 3.7** depicts that of the connector service.

Figure 3.6: Construction of on-site sewer connection (conventional gravity sewer)

Figure 3.7: Construction of connector service (conventional gravity sewer)

3.4.4 Advantages and disadvantages

There are some advantages and disadvantages of conventional gravity sewer systems. These include:

- ✓ A high level of user convenience is obtained.
- ✓ When designed correctly and operated as intended it has few problems.
- ✓ If well-constructed and maintained, it has little impact on groundwater quality.
- ✓ Can handle grit and solids in sanitary sewage.
- ✓ Can maintain a minimum velocity (at design flow), reducing the production of hydrogen sulphide and methane. This in turn reduces odours, blockages and pipe corrosion (USEPA, 2002).
- ✓ Very suited to high-density areas although formal structure/layout is usually required.
- ✓ Easy access to the system.

- ✗ The most expensive sanitation option, in terms of both capital and operating costs.
- ✗ The slope requirements to maintain gravity flow can require deep excavations in hilly or flat terrain, driving up the construction costs.
- ✗ Manholes associated with conventional gravity sewers are a source of inflow and infiltration, increasing the volume of wastewater to be carried as well as the size of pipes and pumping stations, and ultimately increasing costs.
- ✗ Up-front connection fees are often unaffordable for low-income households.

- Can only be designed and installed by trained professionals.
- Problems have been experienced in drainage of low-lying areas
- Requires more water compared to other alternatives to operate effectively.
- Vandalism and theft of manhole covers which could be overcome by replacing with concrete lockable covers.
- Illegal stormwater connections to sewer systems increasing flows and capacity requirements of WWTW.
- Insufficient resources are committed to the operation and maintenance of sewer reticulation, resulting in deterioration of the capital asset and increased maintenance requirements costs. However, this is true for most waterborne sanitation systems.
- Misuse of the system (flushing of incorrect papers and other objects) resulting in blockage (see **Figure 3.8**).

Figure 3.8: Overflowing sewer (RHP, 2002)

- Shortage of skilled personnel which has a knock-on effect on the deterioration of the asset. However, this is true for most waterborne sanitation systems.
- Pressure on consultants and contractors to reduce costs due to financial constraints sometimes results in less robust and/or poorly constructed systems which is aggravated by people misusing it. However, this is true for most waterborne sanitation systems.
- Poorly operated and maintained sewer reticulation systems result in a high frequency of blockages and spills, causing raw sewage to flow into rivers, streets, reservoirs and the sea.
- Only applicable where residents are able to afford the full maintenance and operation costs.

3.5 Vacuum sewer systems

3.5.1 General description

Thirty years ago, vacuum sewers were regarded as 'new' and only to be used as a system of last resort. The technology has improved significantly and is now basically accepted as an alternative sewage collection system. The U.S. Environmental Protection Agency (U.S. EPA) manual, *Alternative Wastewater Collection Systems* (USEPA, 1991), characterized vacuum sewers as lagging behind other collection types. However, at the moment the vacuum system is viewed on par with other collection system types. The lessons learnt from the early systems resulted in better design and operating guidelines and the technology has advanced, resulting in more reliable and efficient systems.

Vacuum sewers use differential air pressure to move the wastewater. A central vacuum pump station is required to maintain a vacuum (negative pressure) on the collection system, see **Figure 3.9**. As described in WEF (2008) the system requires a normally closed vacuum-gravity interface valve at each entry point to seal the lines, so that the vacuum can be maintained. These valves, located in valve pits, open when a predetermined amount of wastewater accumulates in collecting sumps. The resulting differential pressure between the atmosphere and vacuum becomes the driving force that propels the wastewater towards the vacuum station.

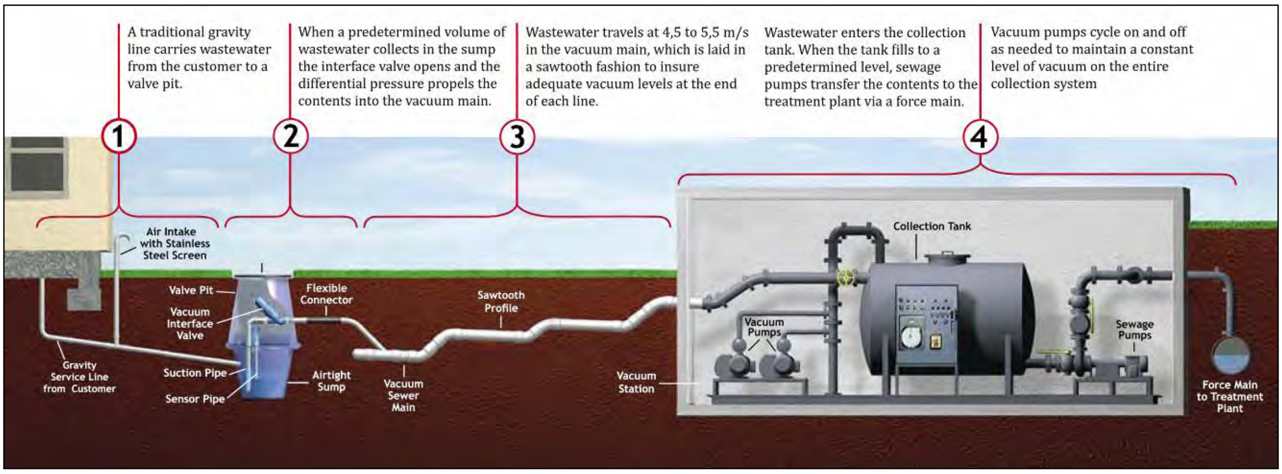

Figure 3.9: Vacuum sewer system (adapted from AIRVAC, 2005b)

A vacuum sewer system consists of three major components – the valve pit, vacuum mains, and vacuum station (see **Figure 3.9**).

Herewith the general conditions conducive to the selection of vacuum sewers:

- Unstable soil or rock
- Flat terrain
- Rolling land, with many small elevation changes
- High water table

- Restricted construction condition
- New urban development in rural areas
- Existing urban development, where built-out conditions exist
- Sensitive ecosystem

Experience has shown that, for vacuum systems to be cost-effective, a minimum of 75 to 100 customers per vacuum station is generally required. The average number of customers per station in systems presently in operation is approximately 200 to 500 (WEF, 2008). Vacuum systems are limited somewhat by topography. The vacuum produced by a vacuum station is generally capable of lifting wastewater 4.5 m to 6 m. This amount of lift may be sufficient to allow the designer to avoid all or many of the lift stations that would be required in a conventional gravity system and the operation and maintenance requirements that they present.

The largest installed vacuum sewer system is at Palm Jumeirah in Dubai, **Figure 3.10**. The system supports more than 2 000 villas and consists of 900 collection chambers and more than 40 km of vacuum sewer lines all connecting to a central Roevac vacuum station.

Figure 3.10: Palm Jumeirah in Dubai utilizes a vacuum sewer system

3.5.2 Technical design criteria

There are some general requirements for the design of vacuum sewer systems as presented in *Alternative Sewer Systems* (WEF, 2008):

- *The entire vacuum sewer system, including the individual valve pits, shall be owned, operated, and maintained by a single operating entity.*

- *The vacuum piping network shall be designed with the intent to keep the bore of the entire pipeline open. Designs where sections of the pipeline are purposely sealed are not allowed.*

- *The vacuum sewer system must be designed to remain operational during the loss of vacuum.*

- *For routine and emergency operation and maintenance of a vacuum sewer system, the public entity responsible for the system shall have the right of access to an adequate supply of spare valves, pumps, parts, and service.*

- *The vacuum sewer system air–to–liquid ratio shall be a minimum of two parts air to one part liquid.*

As indicated in **Figure 3.9** the three major components of a vacuum system are the valve pit, vacuum mains and vacuum station.

- *Valve pit and vacuum valve* - Valve pits and sumps are needed to accept the waste from the houses. The pit may consist of one unit with two separate chambers as shown in **Figure 3.11**. The upper chamber houses the vacuum valve, and the bottom chamber is the sump, into which the building sewer is connected. These two chambers are sealed from each other. The combination valve pit–sump is typically made of fibreglass and is able to withstand traffic loads.

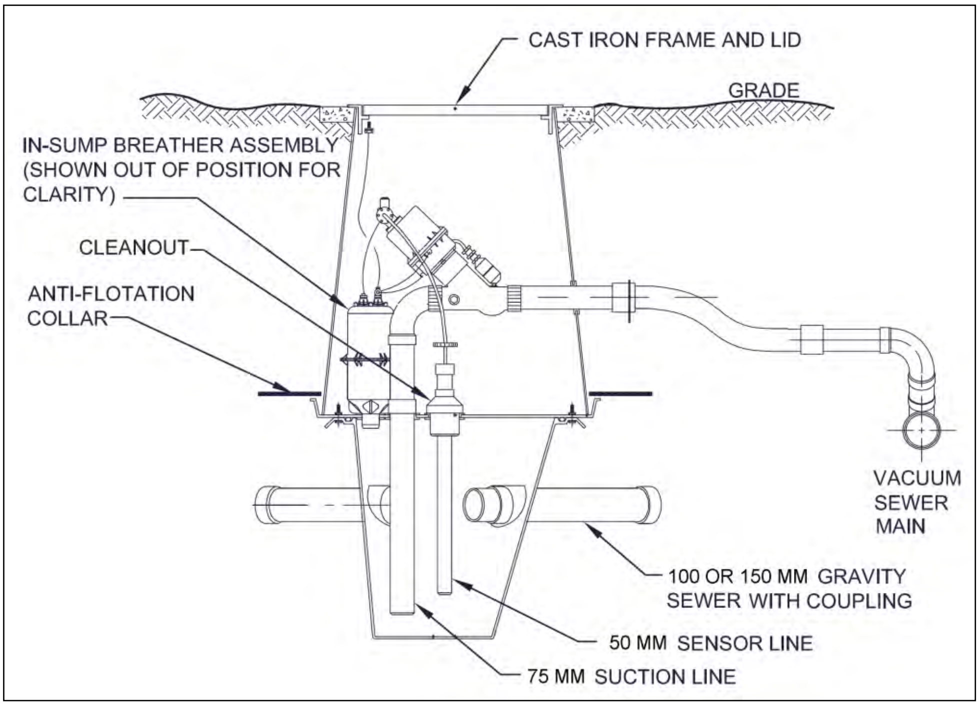

Figure 3.11: Valve pit (WEF, 2008)

The vacuum valve provides the interface between the vacuum in the collection piping and the atmospheric air in the building sewer and sump. The system vacuum in the collection piping is maintained when the valve is closed. With the valve opened, the system vacuum evacuates the contents of the sump.

An air-intake is installed on the homeowner's building sewer, downstream of all of the house traps. This air-intake is necessary to provide the volume of air that will follow the wastewater into the main.

Design requirements (adapted from WEF, 2008)

- A single valve pit should serve a maximum of four EDUs, but no more than a maximum EDU equivalent to 0.2 ℓ/s and the peak flow to any valve pits is limited to a maximum of 0.2 ℓ/s
- The valve pit arrangement shall have a receiving sump, with a minimum of 190 ℓ of storage
- Vacuum-valve pits shall be designed to prevent entrance of water in the sump and for the vacuum valve to remain fully operational if submerged
- Air-intakes, a minimum of 100 mm in diameter, shall be provided for each individual gravity line and shall extend a minimum of 0.6 m above ground level and be protected against physical damage and flooding. Air-intakes shall be screened to prevent the entry of rodents, insects, and debris.
- Vacuum valves shall have the ability to pass a 75 mm spherical solid
- Valves are to be vacuum-operated on opening and spring-assisted on closing
- Valve configuration shall be arranged so that the collection system vacuum ensures positive valve seating. Valve plunger and shaft shall be arranged to be completely out of the flow path when the valve is in the open position.
- The valve shall be equipped with a sensor-controller that shall rely on atmospheric air and vacuum pressure from the downstream side of the valve for its operation, thereby requiring no other power source
- With the exception of the gravity lateral line air-intake, there shall be no other external sources of air necessary or permitted as a part of the valve assembly
- An internal sump breather unit arrangement shall connect the valve controller to its air source and provide a means of ensuring that no liquid can enter the controller during system shutdowns and restarts.

- *Vacuum mains* - The piping network connects the individual valve pits to the collection tank at the vacuum station. Usually polyvinyl chloride (PVC) pipe is used. Earlier systems used solvent-welded joints, but most recent systems use O-ring rubber gasketed pipe with gaskets for vacuum conditions. PVC pressure fittings are needed for directional change and for the crossover connections from the service line to the main line.

  Lifts or vertical profile changes are used to maintain shallow trench depths and for uphill liquid transport. These lifts are made in a sawtooth profile design as shown in **Figure 3.9**. Division valves are used to isolate various sections of vacuum mains, thereby allowing operations personnel to troubleshoot maintenance problems.

Design requirements (adapted from WEF, 2008)

- The general design configuration shall be based on the sawtooth profile design concept. Other vertical pipeline design profiles may be considered, if justified by appropriate engineering data.
- A minimum pipe diameter of 100 mm is required for vacuum sewer mains and gravity service laterals
- Vacuum sewer lines must have a minimum slope of 0.20%
- The maximum design flows (i.e., peak flows) for vacuum pipe sizing are as follows:
  - 100 mm pipe shall be 2.4 ℓ/s
  - 150 mm pipe shall be 6.6 ℓ/s
  - 200 mm pipe shall be 13.2 ℓ/s
  - 250 mm pipe shall be 23.7 ℓ/s
- Lift heights for a single lift should in no case exceed 0.9 m in height. A series of lifts should be made with consistent lift heights.
- For changes in horizontal alignment, two 45° bends connected by a short section of piping are required, rather than one 90° bend
- Isolation valves are required at every branch connection and at intervals no greater than 460 m on main lines. The valves shall be installed with a valve box or other approved apparatus, to facilitate proper use of the valve.
- Recommended piping and fittings for the vacuum collection system is PVC with push-on gasketed joints for piping 100 mm in diameter and larger. Recommended piping and fittings for diameters less than 100 mm is PVC with push-on gasketed joints or with socket-type fittings (solvent-welded).

- *Vacuum stations* - Vacuum stations function as transfer facilities between a central collection point for all vacuum sewer lines and a pressurized line leading directly or indirectly to a treatment facility. **Figure 3.12** shows the major components of the vacuum station. Vacuum pumps are needed to produce the vacuum necessary for liquid–air transport.

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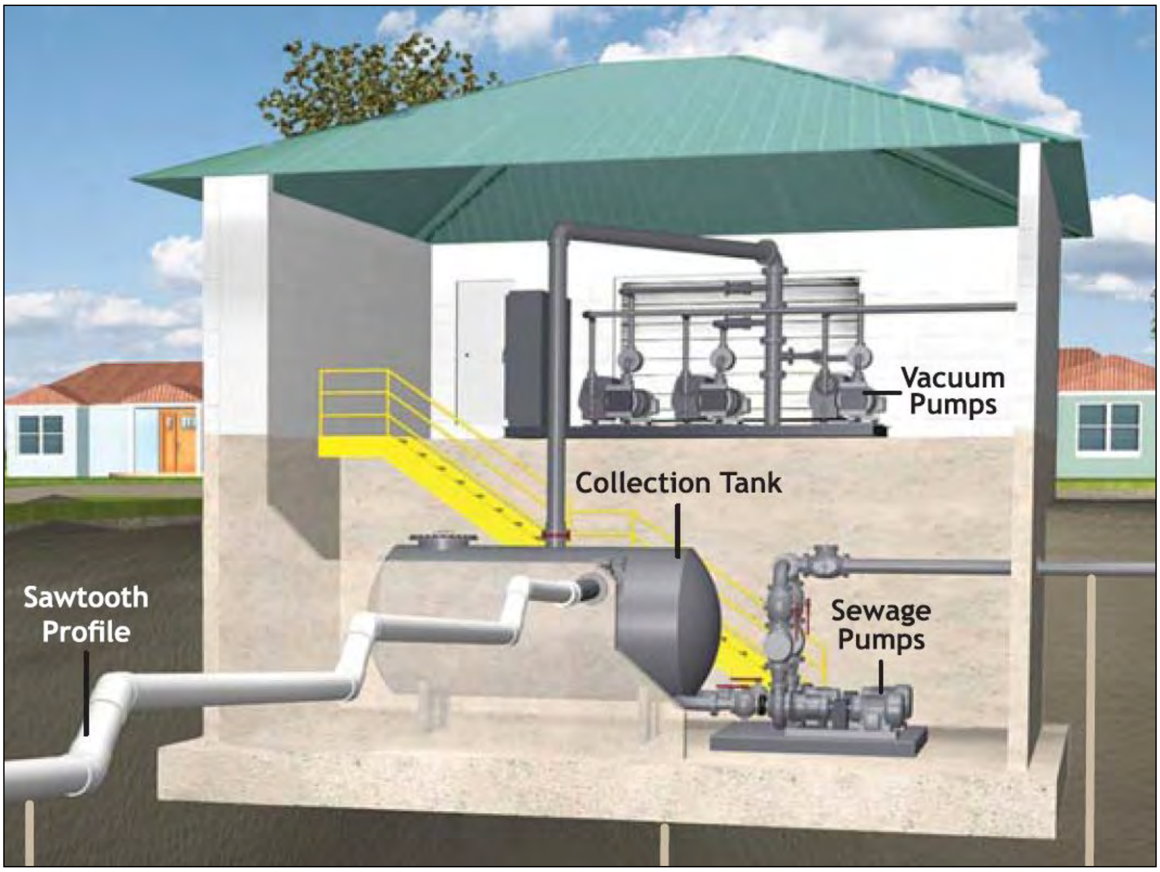

Figure 3.12: Vacuum station (Bock, Fulton, and Slifer, 2009)

Design requirements (adapted from WEF, 2008)

- A minimum peak-flow-to-average-flow ratio of 3.5:1 is recommended for vacuum pump station component sizing
- Standby power shall be capable of handling 110% peak loading. A standby generator is recommended for all vacuum stations.
- A minimum of two pumping units shall be provided for both the vacuum pumps and the wastewater pumps, with each being capable of handling peak-flow conditions with the other out of service
- An alarm system, with the capability to notify staff operators remotely (i.e., telemetry system), shall be provided
- Check valves are required on each wastewater pump discharge line
- Isolation valves are required between the vacuum collection tank, vacuum pump(*s*), influent line, and raw wastewater discharge pipe
- Shutoff valves are required on both the wastewater pump suction and discharge piping
- Necessary pipes, fittings, and valves shall be provided, to allow for emergency pumping out of the vacuum collection tank
- The minimum recommended vacuum pump size is 4.3 m^3/min (7 460 W)
- A two-step process shall be used to size the vacuum pumps. Vacuum pumps shall first be sized according to peak flow. The adequacy of this initial sizing shall then be checked to see that the system pump-down time (t) is between 1 min and 3 min.

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3.5.3 Construction

Herewith a summary of the construction process of a vacuum sewer system:

Step 1 - Surveying and pegging of route. This entails the physical marking of the route.

Step 2 - Right-of-way trimming and clearing. This stage could take a few days per street and may involve cutting and removing driveway sections, drainage structures and other obstructions located in public rights-of-way or dedicated utility easements.

Step 3 - Directional drilling - In selected cases, rather than using an open trench for pipes, work crews will use small drilling machines to install portions of the pipe network without significantly disturbing the area.

Step 4 - Mainline piping – Residents could expect inconveniences during installation of the sewer pipelines. Construction teams will cross driveways, walkways and may excavate entire streets. The contractor will excavate trenches, **Figure 3.13**, of various depths with other teams following, assembling and installing lengths of pipe. The length of pipe installed in one work day varies from 100 m to 450 m depending on ground conditions. In cases where there is a high water table, groundwater 'dewatering' is often necessary.

Figure 3.13: Excavating trenches (QUA-VAC, 2009)

Step 5 - Collection system component installation – The contractor will install a vacuum valve, as shown in **Figure 3.14**, in a valve pit in the right-of-way and extend gravity service laterals from the pits to each resident's property line. The valve pits are fibreglass tanks that range in height from 1.5 m to 2.4 m.

Figure 3.14: Contractor installing vacuum valve in valve pit

Step 6 - Restoration – The contractor will rely on the construction plans and the pre-construction videotape when restoring construction areas to original conditions. Restoration activities that commence once the pipe installation is complete may take several weeks, or even months.

Step 7 - Construction of vacuum stations – Vacuum stations are usually concrete block buildings on concrete foundations with minimum plan dimensions of approximately 7.5 m to 9.0 m. Part of the structure is constructed below grade to accommodate entry of the vacuum sewer. Two alternating liquid ring or sliding vane vacuum pumps withdraw air from a vacuum reservoir tank, which is connected in turn to a fibreglass or steel collection tank. The reservoir tank provides a vacuum reservoir to limit the number of vacuum pump starts and it prevents the vacuum pumps from being in contact with the air/wastewater being collected into the collection tank.

3.5.4 Advantages and disadvantages

The advantage of vacuum collection systems may include substantial reductions in water use, material costs, excavation costs, and treatment expenses. In short, there is a potential for overall cost-effectiveness. The following advantages are evident (WEF, 2008):

- ✓ Small pipe sizes – typically 100 mm to 250 mm are used.
- ✓ No manholes are necessary.
- ✓ The toilet can be placed indoors although this is true for all waterborne sewage systems that have a water seal.
- ✓ Field changes can easily be made, as unforeseen underground obstacles can be avoided by going over, under, or around them, i.e. flexibility of piping.
- ✓ Installation of smaller-diameter pipes at shallow depths eliminates the need for wide, deep trenches, reducing excavation costs and potential dewatering costs.
- ✓ High scouring velocities are attained, reducing the risk of blockages and keeping wastewater aerated and mixed.
- ✓ Elimination of the exposure of maintenance personnel to the risk of hydrogen sulphide gas hazards.
- ✓ The system will not allow major leaks to go unnoticed, resulting in reduced environmental damage from exfiltration of wastewater.
- ✓ Only one source of power, at the vacuum station, is required. No onsite power demand exists at valve pits.
- ✓ Flexible system lends itself to modular design.
- ✓ The elimination of infiltration permits a reduction in size and cost of the treatment plant.
- ✓ Vacuum stations can be designed to blend with the surroundings more than traditional lift stations.
- ✓ Valve pits are more concealable on the customer's property than are grinder pump stations.
- ✓ A single-source responsibility exists, as one operating entity operates and maintains the entire system, including the on-lot valve pit and valve.
- ✓ Aeration of sewage.
- ✓ Minimal maintenance is required.
- ✓ Low energy consumption.
- ✓ Closed system with no leakage or odour.
- ✓ No groundwater pollution (no exfiltration).

There are some disadvantages to vacuum systems such as:

- Multiple house hook-ups can be a source of neighbourhood friction unless the pit is located in public property.
- It requires permanent easements from one of the property owners which could be difficult to obtain.
- Can only be designed and installed by trained professionals.
- Although the mains are small and require shallow burial depth they are limited to approximately 6 m of head.
- The sewer lines require a specific profile of pockets or running traps, so installation requires the same attention to grade as gravity sewers.
- The central vacuum station requires a large capital investment, so for small systems (less than 50 homes) it is not economically feasible.

3.6 Small-bore sewer

3.6.1 General description

Small-bore systems or small diameter gravity (SDG) sewers or solids-free sewers (SFS) are also called septic tank effluent gravity (STEG) sewers, and these systems convey effluent by gravity from an interceptor tank (or septic tank) to a centralized treatment plant or pump station from where it is conveyed to another collection system. Another variation on this alternative sewer system is the septic tank effluent pumping (STEP) concept. All these systems utilize smaller-diameter pipes placed in shallow trenches following the natural contours of the area, thus reducing the capital cost of the pipe as well as excavation and construction costs.

A solids-free sewer (SFS) system is a system of effluent conveyance which uses an onsite tank to settle solids out of the sewage, and conveys the liquid effluent only to a central treatment and/or disposal point by means of a sewer network (Pisani, 1998c), as depicted in **Figure 3.15**. The onsite tank could be a septic tank, interceptor tank, aqua-privy or anaerobic digester. Most of the suspended material is removed from the wastewater flow by this tank reducing the risk of clogging to occur. The sludge remaining in the tank must be removed by means of a vacuum tanker and transported to the treatment works. The diameter of the pipes downstream of the tank, the lateral and the sewer main can thus be reduced. Cleanouts are used to provide an access point for flushing and manholes are rarely constructed. Air-release risers are required at summits in the sewer profile. Odour control is important since wastewater from the tank still has the potential to release odours.

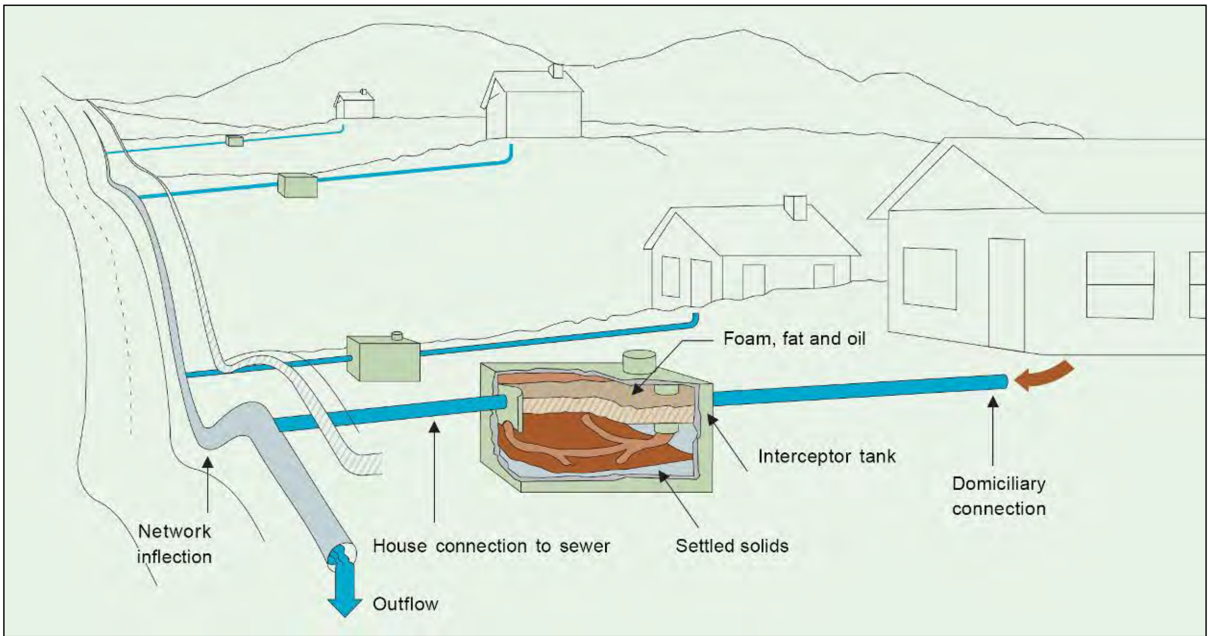

Figure 3.15: Solids-free sewer system (Alvéstegui, 2005)

The small-bore sewer system is commonly used in South Africa's rural and peri-urban areas, as well as in developed countries like Australia and the United States. The small-bore system is similar to a gravity sewer system, collecting sewage from households and discharging it into a main disposal facility through gravity (Little, 2004).

The difference between a small-bore system and a gravity system is the inclusion of an interceptor/septic tank between the house and the main sewer distribution pipeline. The interceptor tank prevents gross solids from entering the main sewer system. The connection pipe that conveys effluent from the house to the tank is typically 100 mm or 150 mm in diameter. The outlet pipe to the distribution network is designed to convey only the liquids discharging from the tank, resulting in a more economically reduced pipe diameter.

Unlike conventional gravity sewers which are designed for open-channel flow, small-bore sewers may be installed with sections depressed below the hydraulic gradient line, i.e. have adverse slopes. Thus, flow within a small-bore sewer may alternate between open-channel and pressure flow. The sewer pipes can therefore be laid at flat gradients as they do not carry solids that require transportation at scouring velocities (TAG, 1985).

Refer to **Figure 3.16** for a schematic representation of a typical small-bore sewer distribution system:

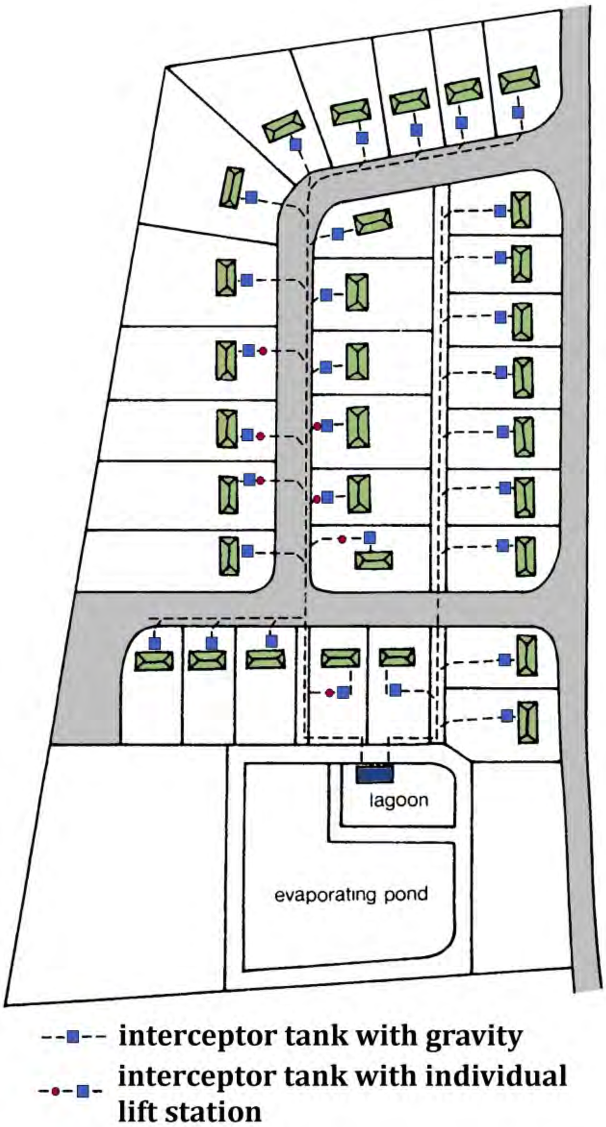

Figure 3.16: Small-bore sewer distribution system (TAG, 1985)

3.6.2 Technical design criteria

A small-diameter gravity sewer system conveys settled wastewater to the treatment facility utilizing the difference in elevation between the upstream connections and the downstream outlet. It must be set deep enough to receive flows by gravity from the majority of the service connections and have sufficient capacity to transport the anticipated peak flows. In sections where the differences in elevation are too small to allow for gravity flow energy must be added by means of a lift station.

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- *Hydraulic design*
  Design-flow estimates – The sewer must be designed to carry expected peak hour flow. There is less infiltration/inflow and attenuation is created in the interceptor tanks and thus the average daily wastewater flow per capita is approximately 190 ℓ/d. The instantaneous peak flows are typically 0.03 ℓ/s to 0.06 ℓ/s (USEPA, 1991).
  Flow velocities – In STEG systems, the primary treatment provided in the interceptor tanks upstream of each connection point removes grit and most grease and settleable solids. The remaining solids which enter the collectors and slime growth which develops within the sewer are easily flushed away at velocities of 0.15 m/s.

- *Collector mains*
  Layout – Small-bore sewer distribution systems are most often installed in front of the properties underneath the pavement.
  Alignment and grade – The horizontal alignment does not need to be straight, i.e. it can be installed to avoid obstacles. The vertical alignment does not need to be uniform provided the hydraulic grade line (HGL) does not rise above any upstream interceptor tank outlet invert during peak-flow conditions.
  Pipe diameter – Determined by means of a hydraulic analysis. Minimum diameter is 100 mm but 50 mm diameter pipes have been used with some modifications to the interceptor tank outlets.
  Depth – The depth for the burial of the collector mains is determined by the elevation of the interceptor tank outlet invert elevations or is based on trench loadings. Where gravity drainage from a residential connection is not possible STEP lift stations are used. Minimum depth is typically 0.75 m.
  Pipe materials – PVC plastic pipe is the most commonly used pipe material in STEG systems.

- *Service laterals*
  Typical service laterals between the tank and the sewer line are 100 mm diameter PVC pipes. The connection is usually made with tee or wye fittings.

- *Interceptor tanks*
  Location – The interceptor tanks should be located where they are easily accessible for periodic removal of accumulated sludge.
  Design – Prefabricated, single-compartment septic tanks are typically used for interceptor tanks in STEG systems. Inlet and outlet baffles, **Figure 3.17**, are provided in conventional septic tanks to retain solids within the tank.

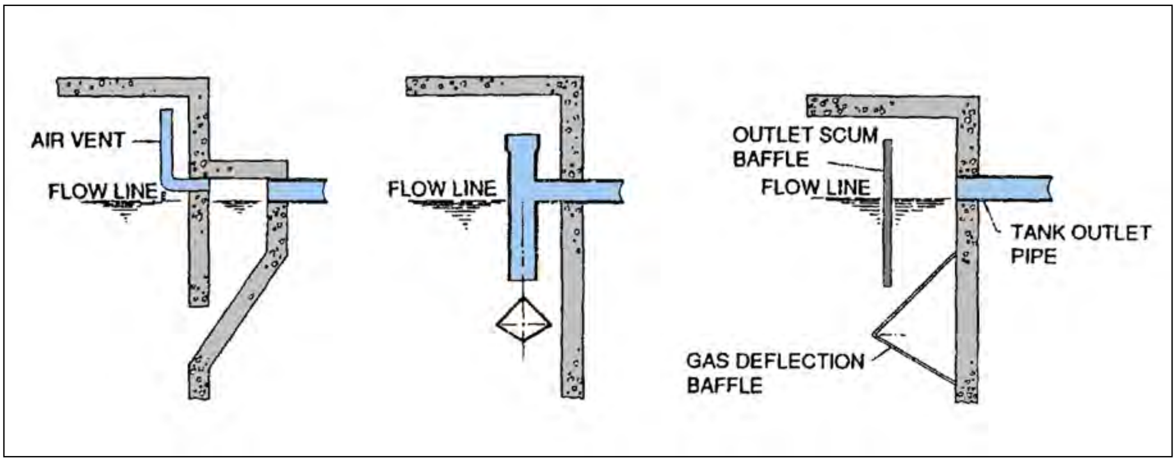

Figure 3.17: Typical interceptor tank outlet baffles (USEPA, 1991)

Material – The tanks are usually made in reinforced concrete, coated steel, fibreglass and high-density polyethylene. All tank joints must be watertight.

- *Manholes and cleanouts*
  In most small-bore sewer distribution systems, cleanouts are used instead of manholes, except at major junctions at mains.

- *Valves*
  Air-release, combination air-release/vacuum, **Figure 3.18**, and check valves are used in small-bore sewers. Air-release and combination air-release/vacuum valves are used for air venting at summits in mains that have inflective gradients. Check valves are sometimes used on the service connections at the point of connection to the main to prevent backflow during surcharged conditions.

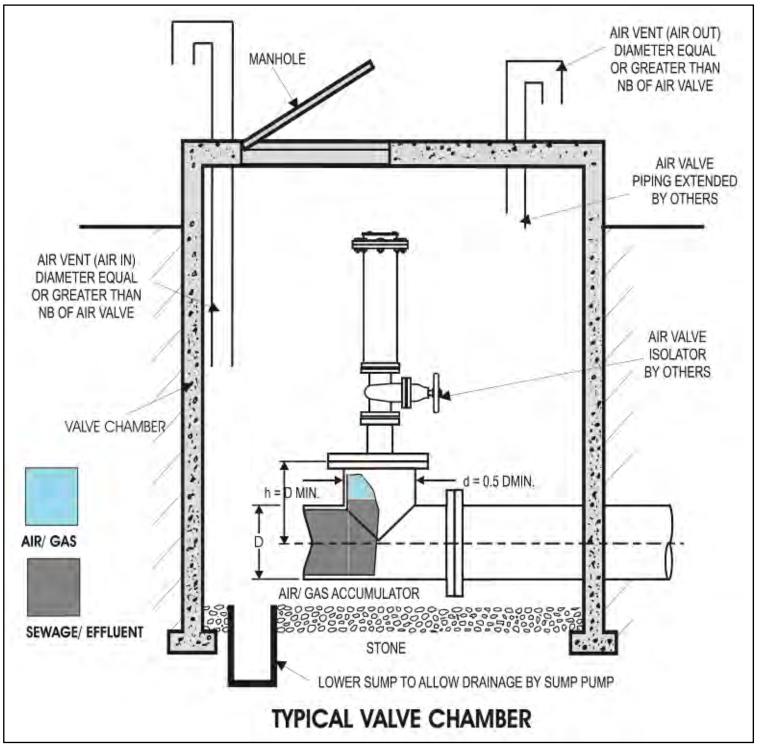

Figure 3.18: Combination air-release/vacuum valve (Vent-O-Mat, 2009)

- *Odours and corrosion*

  Odours are a commonly reported problem with small-bore sewage collection systems. The settled wastewater collected by small-bore systems is septic and therefore contains dissolved hydrogen sulphide (H_2S) and other malodorous gases. These gases tend to be released to the atmosphere in quantity where turbulent conditions occur such as in lift stations, drop cleanouts or hydraulic jumps which occur at rapid and large changes in grade or direction in the collector main. The odours escape primarily from the house plumbing stack vents, manholes or wet-well covers of lift stations.

  The odours are controlled by minimizing turbulence and sealing uncontrolled air outlets. Air-tight lift station covers should be installed if odours are persistent and odour control provided for the fresh air vent. An effective odour-control measure is to terminate the vent in a buried gravel trench. Manholes should be replaced with cleanouts; however, if used, the manholes should have air-tight covers.

  The atmosphere created by the released gases is very corrosive. Corrosion is a common problem in lift stations. Corrosion-resistant materials must therefore be used. More recent small-bore sewer systems are using wet-well/dry-well design for lift stations to reduce the exposure of mechanical components to the corrosive atmosphere.

3.6.3 Construction

Construction of small-bore sewers is similar to construction of conventional gravity sewers except that strict horizontal and vertical control of main alignment is not required (USEPA, 1991). Small-bore sewer systems require that a significant portion of the work be performed on private property to install the interceptor tank and service lateral.

Prior to the start of any work, rights-of-way, work areas, clearing limits and pavement cuts should be laid out to protect adjacent properties. Access roads, detours and protective barricades should be laid out and constructed as required in advance of construction.

- *Mainline sewers*

  Thermoplastic pipes, most commonly PVC but also low-density polyethylene, are used for small-bore sewers. Their advantages include light weight, long laying lengths, high impact strength, corrosion resistance, flexibility and ease of cutting in the field.

  Line changes – Determining the route of the collector main should be performed with the objective of minimizing site restoration costs. All obstacles in the intended path of the main sewer may not have been identified on the plan sheets during initial surveys. Since straight alignment is not required for small-bore sewers, changes in the alignment within maximum pipe deflection limits can be made in the field to avoid large trees, fences, pavement, etc.

Most changes can be made by the construction manager, but major changes in alignment should be evaluated by the design engineer.

Grade control - Strict vertical control of small-bore sewers during main-line construction is not necessary. In most cases, the pipe may be joined above ground and laid in the trench. However, the pipe should be laid as uniformly as is reasonable to minimize head losses and potential points where gas can collect.

Trench construction - Trenching may be done by backhoe or trenching equipment. Over-excavation is not a critical concern if the change in the pipe invert elevation is not greater than one pipe diameter or so sudden that the integrity of the pipe is threatened. Selective backfill for bedding and surrounding the pipe is necessary if the native trench spoil contains cobble or does not fill the area around the pipe snugly. Granular materials such as medium or coarse sand are usually used. Local requirements may control the use of backfill materials. Pipeline markers which relate the pipe to existing permanent above-ground structures should also be used.

HPDE piping allows two simple installation options – open cut (digging small, shallow trenches) or trenchless technology (TT) using horizontal directional drilling (HDD), as shown in **Figure 3.19**.

Figure 3.19: Directional drilling and installing of small-bore sewer

Both installation options cost significantly less than traditional sewers, can be installed in a fraction of the time using local labour, and will have a minimal impact on the environment.

- *Service connections*

  Service connections include the building sewer, interceptor tank and service lateral to the collector main. Usually, the municipality is responsible for installation of the interceptor tank and service lateral whilst the user is responsible for installing the building sewer and its connection to the interceptor tank.

The building sewer should be a 75 mm to 100 mm diameter pipe installed at a uniform negative gradient sufficient to transport faecal solids but not so great as to strand solids in the line. Recommended gradients are 1 in 30 (75 mm pipe) and 1 in 40 (100 mm pipe). Bends greater than 45° should be provided with a cleanout.

- *Interceptor tanks*

In developing countries interceptor tanks are most commonly constructed in brick or block work on a concrete base, and rendered with cement mortar internally. If available, prefabricated tanks of precast concrete, glass-fibre-reinforced plastic or thermoplastics may be used. Precast concrete tanks can be readily made at a central casting yard, and are usually made in flat or cylindrical sections for ease of transport and subsequent erection on site.

Location - The tanks should be located where they can be reached easily for routine removal of solids. The tanks should be clear of vehicular traffic areas unless the cover is adequately reinforced to withstand live traffic loads.

Inlet and outlet piping - All joints at the tank should use rubber gaskets or be sealed with a durable, watertight, flexible material.

Bedding and backfilling - The tank must be set level on undisturbed soil at an elevation that allows at least a 2.5% slope in the building sewer.

Flotation collars - If the tanks have to be set in soil that may be saturated at any time, flotation collars should be used to prevent flotation when the tank is desludged.

Existing tanks - Existing tanks may be used to reduce construction costs if they are in good repair. A careful inspection of each tank is required.

- *Cleanouts and manholes*

Cleanouts can be easily made up from standard PVC pipe fittings or they can be made as small inspection boxes in cement-rendered brick with suitable benching and airtight covers.

Manholes are best made in concrete or cement-rendered brick; standard manhole designs (for conventional sewer networks) should be used. Covers of manholes should be sealed to prevent ingress of water into the sewer system.

- *Lift stations*

Effluent pumps - Centrifugal submersible effluent pumps are most commonly used with small-bore sewer systems. All pumps should be of cast iron, bronze and/or plastic construction and mounted on a base.

Discharge piping - Because of the corrosive nature of the wastewater, only plastic pipe should be used. Quick-disconnect couplings should be provided to allow easy removal of the pump for repairs.

Level sensors - Mercury level control switches have been found to be the most trouble-free of the several types of switches readily available.

3.6.4 Advantages and disadvantages

Some of the numerous advantages of small-bore sewage collection systems include:

- ✓ Low flush can be used since solids only need to be conveyed over a short distance.
- ✓ The tank functions as an interceptor for unwanted material being flushed down the system.
- ✓ Existing systems (conservancy or septic tanks) can be easily upgraded to an SFS system.
- ✓ Simpler treatment process for settled sewage.
- ✓ The smaller pipes used in small-bore sewer systems can be designed to flow full and in some cases pressurized.
- ✓ Flatter slopes (thus less excavation) and velocities are acceptable.
- ✓ Elimination of costly manholes – these can be replaced with cheaper cleanouts.
- ✓ Low operation and maintenance cost.
- ✓ Water use can be limited thus reducing operating cost.

However, the system has certain disadvantages such as:

- ✗ The desludging of the tank and disposal required by the local authority.
- ✗ The build-up of scum in the tank which inhibits flow and decomposition.
- ✗ Backing up of wastewater if problem at tank outlet.
- ✗ Surface flooding or seepage.
- ✗ Greenhouse gases.
- ✗ If a blockage does occur it could be difficult to locate in the closed system.

3.7 Simplified sewerage / Shallow sewerage / Condominial sewerage

3.7.1 Background and general description

Simplified sewerage is an off-site sanitation technology that removes wastewater from the household environment. Conceptually it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local circumstances.

Initially the technology for shallow sewer systems was developed to provide a service for the poorer communities. Literature, however, indicates that in some countries it has been developed as the standard option. As described earlier, the minimum acceptable sanitation option in South Africa is the Ventilated Improved Pit (VIP) latrine and the highest level of service perceived is conventional waterborne sewerage. Shallow sewerage falls somewhere between these two boundaries.

Many of the conventional sewerage design standards, such as minimum diameter, minimum slopes, and minimum depths, are relaxed in shallow sewer systems, and community-based construction, operation and maintenance are allowed.

3.7.2 Technical design criteria

According to Pegram and Palmer (1999), shallow sewer systems '*require a relaxation of traditional design and construction standards and an associated education of the technical personnel who are responsible for their implementation and management*'. Eslick and Harrison (2004a) provide new guidelines and suggestions:

- Technical design standards for sewers, the 'Red Book' (CSIR, 2003), South African Bureau of Standards (SABS, 1982a) and others should be relaxed.

- Site-specific designs of block feeders and trunk mains to minimize cost of the system should be encouraged.

- Building codes for household fittings and house connections should be relaxed, allowing the installation of connections, with less stringent connection requirements, by locals. The connection into the trunk main will still require a quality control process.

- However, it remains important to consult and educate the residents in cases where traditional standards are relaxed, to ensure ownership and proper use of the system.

- In cases where the construction or management of the system is delegated to a small contractor, capacity building is still required to ensure the transfer of skills into the community.

As detailed in Pegram and Palmer (1999), the shallow sewer system approach borrowed from and adapted the principles of simplified sewerage. A detailed description of the various design components of shallow sewer systems is provided in UNCHS-HABITAT (1986) and a summary is provided below.

- *System layout*

The layout of the sewer system is an important element of the shallow sewer design approach (UNCHS-HABITAT, 1986). As described in Pegram and Palmer (1999) and Watson (1995) the shallow sewer system within a residential area consists of three main components, namely house connections, block feeder sewers and trunk mains as indicated in **Figure 3.20**.

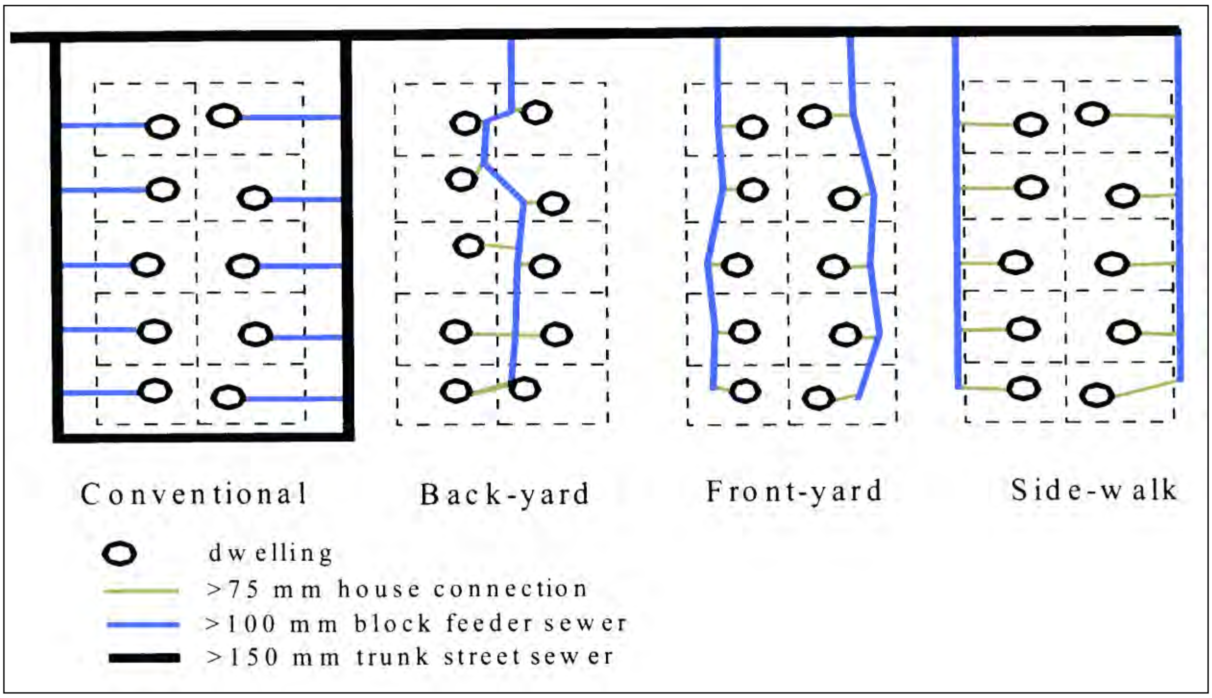

Figure 3.20: Layout of a conventional and shallow sewer system (Watson, 1995)

As indicated in **Figure 3.20** there are three options for placing of the shallow sewer, called the condominial sewer. The sewer can either be laid mid-block in the back yards (back-yard system) of the houses, or in the properties of the houses (front-yard systems) or the sewer can be laid in the street sidewalks (under the pavements). The various options have different advantages and disadvantages when comparing cost, maintenance and accepting ownership.

- *House connections*

The house connections will include all the in-house plumbing fixtures and the pipe work connecting these to the inspection chamber on the block feeder sewer. As detailed in Pegram and Palmer (1999) the in-house plumbing fixtures usually consist of a low-volume pour-flush or cistern-flushing toilet and wash basin, although the low volume could be problematic.

Other fixtures in the house such as the kitchen sink or showers may also be connected. The toilet is connected to the inspection chamber which has a ventilation pipe, by a 100 mm diameter connector pipe; 75 mm could also be considered, laid at a minimum gradient of 1:50. The installation of the house connections is the responsibility of the households themselves. Where conventional systems use 8 ℓ to 9 ℓ/flush the shallow sewer only uses 1.5 ℓ/pour flush.

Block feeder system

The block feeder sewers are trenched in either of three positions depicted in **Figure 3.20**. The block feeders are laid in straight lines between inspection chambers, at a constant gradient greater than 1:167, with at least 400 mm cover. The minimum diameter of the block feeder pipes is 100 mm.

Block feeder systems are different from conventional street sewers in mainly three ways, pipe diameters (100 mm vs. 150 mm), gradients (1:167 compared with 1:60) and inspection chambers are used instead of manholes.

Inspection chambers

Inspection chambers are provided along the feeder sewer line at regular intervals, located in open areas. The inspection chambers provide house connections as illustrated in **Figure 3.21** and access for sewer maintenance. The size of the chamber depends on the depth of the sewer line and the chambers are fitted with a tight-fitting cover.

Figure 3.21: Household inspection chamber (Pegram and Palmer, 1999)

- *Trunk street sewer system*

The trunk street sewers could be designed according to the standards used for conventional, simplified or small-bore sewer design principles. As indicated in Pegram and Palmer (1999) many of the pilot shallow sewer systems are implemented on a single street block which is then connected to communal septic tanks and/or waste stabilization ponds providing treatment. Trunk sewers are usually laid under street sidewalks, deep enough to receive the sewage from the block feeder system or with approximately 1.0 m cover. Trunk sewers are very similar to conventional sewers except for the shallower depths at which the block sewers discharge into the trunk sewer.

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- *Main design guidelines*

  - The minimum self-cleansing velocity in shallow sewers is 0.5 m/s (compared with 0.7 m/s in conventional waterborne systems). The pressure force of the water backing up behind the solids in the smaller pipes flushes the solids down the system and this is the main reason for the relaxation of the minimum self-cleansing velocity. A maximum flow velocity of 4.0 m/s has been set although the effect of these high velocities, for short periods, on the sewer pipes itself is insignificant.

  - Based on accepted practice, a minimum flow depth of 20% of the pipe diameter (ensuring solids transport) and a maximum flow depth of 80% of the pipe diameter (to have surplus capacity) is suggested

  - Shallow sewers use 100 mm piping, where possible, only increasing when estimated peak sewage dictates

  - To determine the minimum sewer gradient the Manning equation can be simplified, based on the reduction in minimum velocity and minimum flow depths mentioned above and assuming a Manning roughness coefficient of $n = 0.013\ s/m^{1/3}$ (UNCHS-HABITAT, 1986):

$$S = 0{,}01Q^{-\frac{2}{3}} \qquad \text{... (3.1)}$$

    where:

    S = pipe gradient (m/m)

    Q = flow rate (ℓ/s)

    This indicates that the minimum sewer gradient is independent of the pipe diameter for a given flow rate. Later studies by Mara (1996) indicated even flatter gradients discussed in more detail later in this guide.

  - In South Africa the peak sewage flows can be calculated utilizing the information as provided in **Section 5** of this guide for conventional gravity sewers.

Assuming an infiltration rate of 15%, a peak factor of 2.5 and that approximately 80% of the household water consumption will be return flow, the peak-flow rate will be between 0.004 ℓ/s·du and 0.01 ℓ/s·du.

- The sewer pipe should have at least 400 mm cover to prevent damage. **Figure 3.22** depicts the excavation of a shallow sewer.

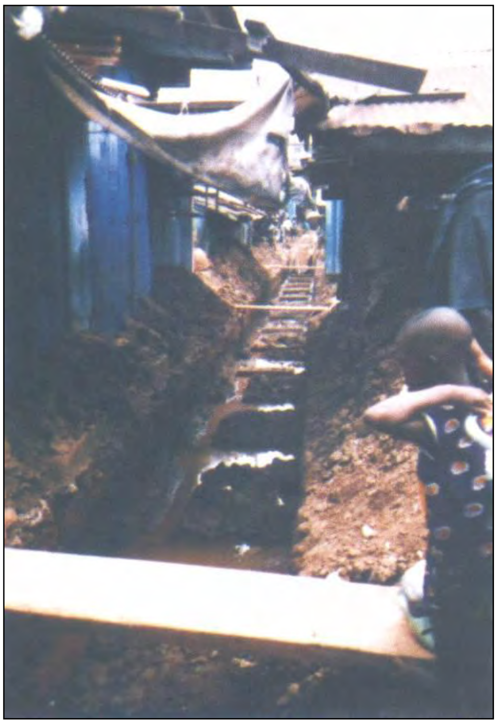

Figure 3.22: Excavation of shallow sewer (Pegram and Palmer, 1999)

3.7.3 Construction

With some basic training of the community they could install the pipelines themselves and maintain them thereafter, see **Figure 3.23**. Similarly, the inspection chambers could be constructed, **Figure 3.24**, and house connections made.

Figure 3.23: Installing the shallow sewer (Eslick and Harrison, 2004a)

Figure 3.24: Construction of inspection chambers (Eslick and Harrison, 2004a)

3.7.4 Advantages and disadvantages

There are some advantages and disadvantages of shallow sewer systems. These include:

- ✓ Initial capital outlay is lower than that of conventional waterborne sanitation and thus more people would be able to benefit from the available budget.
- ✓ There are fewer labour costs although costs for training should be included.
- ✓ Fewer manholes are required
- ✓ Maintenance costs are reduced since the community will be responsible for their section of the system if it needs to be cleared or maintained.

- ✓ These systems allow easy access into confined spaces compared with other systems.
- ✓ Suitable for low-income urban and peri-urban areas.
- ✓ Lower monthly costs to operate and therefore more affordable.
- ✓ Same level of service as conventional waterborne sanitation alternative.
- ✓ Comfortable and convenient and seen as the upliftment to the level of service of the developed world.
- ✓ Theoretically a water-saving system.
- ✓ May be the only system available in very dense retro-fit applications.
- ✓ Improve and maintain the environment.

- ✗ On the negative side it still requires a conventional main sewer system into which the shallow sewer can be connected.
- ✗ It requires from the community the willingness to work together.
- ✗ There is a mismatch of the shallow sewer methodology of community 'self-help' vs. community expectation of 'government will provide' (Eslick and Harrison, 2004b).
- ✗ There are still some legal conflicts such as ownership of the system, contractual difficulties and conflicts with existing standards and regulations.
- ✗ Access problems in small inspection chambers.
- ✗ According to Eslick and Harrison (2004b) the transfer of expertise to new homeowners, i.e. those who were not owners at the time when the project was implemented, also remains problematic.
- ✗ Other project management, condominium management and political influences could also influence the success of the system.

3.8 Other systems

Alternative systems which also utilize water include a conservancy tank, septic tanks, aqua-privies, anaerobic digesters which can be connected to a conventional soakaway, or a small-bore sewer system (solids-free sewer).

3.8.1 Conservancy tank system

3.8.1.1 General description

This is a system which is used extensively in the Far East as well as by several local authorities (CSIR, 1991), such as Glentana in the Southern Cape. This system consists of a standard flushing toilet which drains into a conservancy tank situated on the property. In some cases a number of properties could be drained into one larger tank. The tank can take both excreta and greywater.

In some cases the conservancy tank, sometimes referred to as the vault, is used to collect excreta plus flushing water only. A separate facility is then provided for sullage disposal on-site. The vault is designed to be emptied by vacuum tankers, i.e. tankers equipped with suction pumps. The tankers typically have storage capacities of between 1 500 ℓ and 5 000 ℓ. The appropriate volume of the conservancy tank should be determined based on the planned number of periodic emptying cycles and the anticipated waste generated.

The vault must thus be accessible to the vacuum tanker with a centralized disposal wastewater treatment works also within reach for the tanker. Additionally, maintenance facilities for the vacuum tanker should also be provided.

The implementation of such a sewerage system will depend on the availability of working vacuum tankers ('honey suckers') in the local authority. The owner of the property served by such a tank shall provide and maintain at his own expense a suitable road and access to enable the vehicle used by the local authority to reach and empty such tank.

Payment should be made to the local authority for the clearance of such tank in accordance with their tariff structure.

The design and construction of a conservancy tank is usually done by trained or skilled professionals.

3.8.1.2 Technical design criteria

An illustration of a typical conservancy tank is provided in **Figure 3.25**.

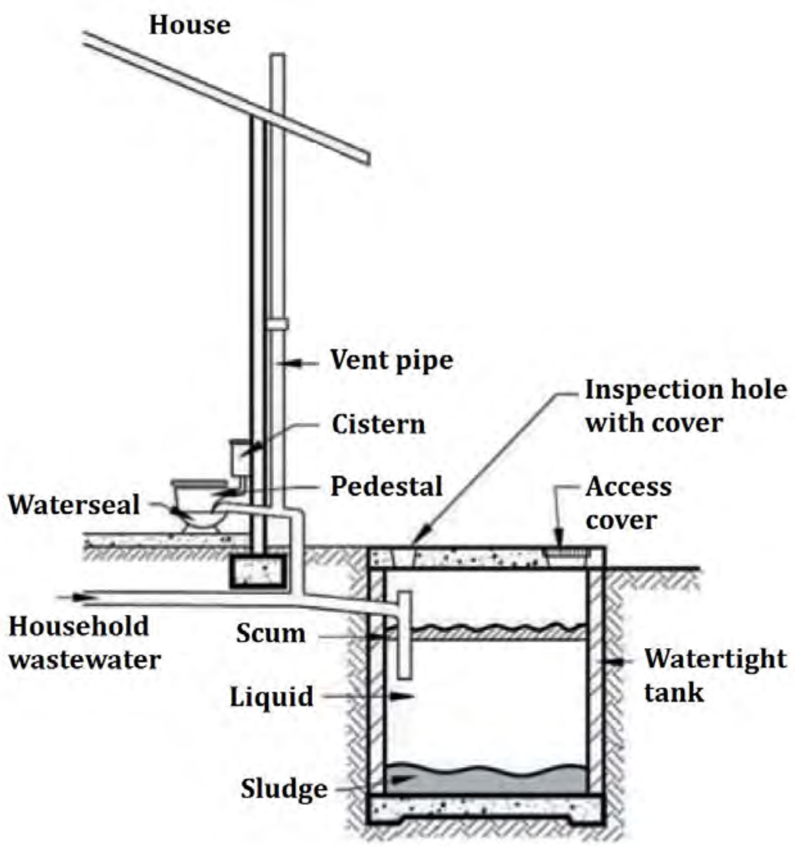

Figure 3.25: Flushing toilet with conservancy tank (CSIR, 2003)

Tank geometry

The conservancy tank should be designed to allow easy access for emptying purposes. An inspection hole for determining whether emptying is required is useful. The tank should be watertight and sited on the premises so as not to become a source of nuisance or danger to health and not to endanger the structure of any building. The sizing is dependent on the planned emptying cycle and estimated quantity of waste generated.

The City of Cape Town (2007) specifies the following requirements for a conservancy tank:

- Tank to be constructed with 215 mm brick or 150 mm reinforced concrete walls on a foundation slab of mass concrete not less than 150 mm thick
- Tank shall be at ground level and shall be provided with one or more air-tight manhole covers to allow access to the tank for cleaning
- Floor of the tank shall be graded to a point which is vertically below one of the manhole covers. At this point a small sump should also be provided with a depth between 150 mm and 225 mm and a sump area of between 300 mm x 300 mm and 450 mm x 450 mm.
- Capacity of the tank shall be not less than 5 400 ℓ or the maximum amount of sewage likely to be discharged into it over a period of 2 d, whichever is the greatest. The tank size should be an exact multiple of 5 400 ℓ (dependent on vacuum tanker capacity).

Clearing services

Provision should be made for an approved clearing service. The frequency of emptying of the tank will depend on the size and is directly related to the level of water supply to the residential building. A vacuum tanker, **Figure 3.26**, would empty the tank and convey the sewage to a central wastewater treatment works for purification.

Figure 3.26: Vacuum tanker for emptying conservancy tank

3.8.1.3 Advantages/disadvantages

There are some advantages and disadvantages of conservancy tank sanitation systems. These include:

- ✓ An advantage of the conservancy tank is that it requires less frequent emptying than, for instance, the bucket-cartage system.
- ✓ System can be used in high-density areas.
- ✓ The conservancy tank system could be easily upgraded to a solids-free sewer system.
- ✓ It provides for a high degree of flexibility in matching collection facilities with the demand.

- ✗ There is a possibility of surface water and groundwater contamination.
- ✗ Initial capital cost to construct the watertight tank could be high for the homeowner.
- ✗ Operating cost could be high since it is dependent on a skilled labour force to empty the tank and transport the contents to a WWTW.
- ✗ Fuel costs need to be considered and are dependent on distance to the WWTW with road transport.

3.8.2 Septic tanks

3.8.2.1 Background

Septic tanks form part of the sewage disposal system which can be connected to the outlet of any water-flush latrine. An advantage of a septic tank is that the household has all the advantages of the conventional waterborne sanitation system without the need for extensive/expensive wastewater treatment, except for the periodic removal of sludge. The cost of the system is carried by the household.

The septic tank's function is to slow down discharges from the building's plumbing fixtures so that solid material can settle to the bottom of the tank and greases and scum can rise to the top (see **Figure 3.27**). A stable biological system within the tank promotes the conversion of organic solids to soluble organic chemicals and gases. The result is a relatively uniform quality seepage that will proceed to the soakaway/leaching field or SFS. There is no need to introduce any commercial additives to the tank to promote biological growth although it helps initially to start the process.

The biological decomposition process in a septic tank does not purify wastewater to potable water standards; it treats wastewater to some degree, but largely serves as a primary treatment storage vessel. Although the outflow from a septic tank to the absorption field looks clear, it can contain many disease-producing bacteria. Final treatment of the sewage and the destruction of disease-causing organisms occur in the soil.

3.8.2.2 General description

The tanks are usually designed for a fairly short water-retention time of 1 d to 2 d and therefore the pathogen removal is rather poor. Septic tanks are usually connected to soakaways (see **Figure 3.28**). The design of septic tanks can be such that it can handle the sullage water from bathrooms and kitchens or only the effluent from toilets. In the case where the sullage does not pass through the septic tank it discharges straight into the soakaway after passing through a grease trap.

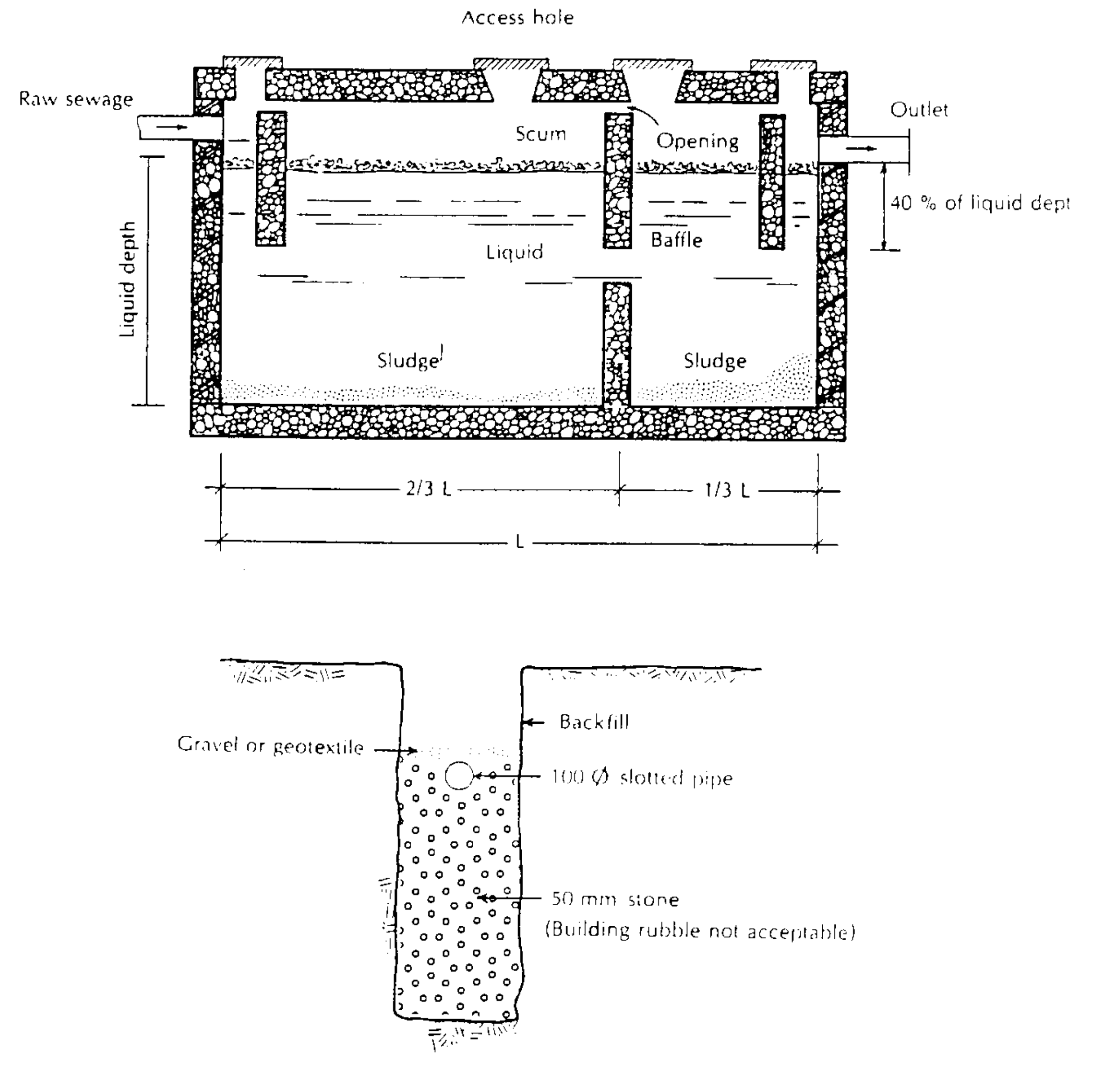

Figure 3.27: Example of a septic tank system (CSIR, 1991)

The septic tank can take the discharge from a conventional, low-flush or pour-flush system. The solids remain in the septic tank and it is required to periodically remove the sludge from the tank and dispose of it at a WWTW.

3.8.2.3 Technical design criteria

- *Geometry of the septic tank*

The geometry of the septic tank influences the flow velocity at which the sewage passes through the tank, the accumulation of sludge as well as the possible presence of stagnant areas in the tank. It is usually recommended to use double compartments which will reduce the peak flow due to increased surface area and allow the higher concentration of solids to settle first.

A good design would typically include:

- A liquid depth of between 1.0 m and 1.8 m
- Rectangular shape with length three times the width
- The first compartment should be twice the volume of the second compartment

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Inlet and outlet arrangements

It is recommended that the inlet to the first compartment be a sanitary T-piece or baffle wall. The vertical portion of the T-piece should extend below the surface liquid, to minimize incoming turbulence. The lower vertical arm of the inlet should be submerged between 30% and 40% of the liquid depth. The upper vertical arm of the T-piece should extend at least 50 mm above the crown of the inlet and end 15 mm below the cover of the tank. The invert of the inlet pipe should be between 50 mm and 75 mm above the surface of the liquid (SABS, 1993).

The sewage in the tank passes from the first compartment to the second through a mid-depth opening.

The outlet from the second compartment should also be a sanitary T-piece or baffle wall. All arms of the T-piece should have an inside diameter of half to three-quarters of that of the inlet pipe, thus damping peak inflows. The invert of the outlet pipe should be between 50 mm and 75 mm below that of the inlet pipe (SABS, 1993).

Some design criteria indicate that septic tanks shall have an effluent filter placed at the outlet in place of the outlet baffle. The purpose of the filter is to trap suspended solids that are not heavy enough nor have had time enough to sink to the bottom of the tank (as in a tank that hasn't been pumped in a timely manner and contains significant amounts of material that reduce its effective volume). Filters must, however, be periodically cleaned so that they do not plug and back sewage into the house.

Capacity of tank

The capacity of the septic tank should be adequate to store sludge and scum, as well as to retain liquid for at least 24 h just prior to the tank requiring desludging. The flow to the septic tank is directly related to the level of water supply to the residential building. Therefore the level of water supply to the building can be used to determine the capacity required. There are basically three methods to determine the capacity of the tank (SABS, 1993):

- For non-residential systems, estimate the average daily flow from the establishment. The capacity of the septic tank has to be 3 times the estimated average daily flow.

- For dwellings or dwelling units with full in-house water reticulation, relate the capacity of the septic tank required to the number of beds or bedrooms (see SANS 10252-2:1993 (SABS (1993)).

- For special residential systems such as multi-home systems or dwelling units without full in-house water reticulation, relate the capacity of the septic tank required to the number of persons to be served by the system (see SANS 10252-2:1993 (SABS (1993)).

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The tank is sized to provide sufficient capacity to limit the periodic removal of sludge to between 2 years and 5 years, i.e. creating user convenience and being economical.

Absorption field/Soakaway

The septic tank only partly treats sewage and it is the function of an absorption field or leaching field or soakaway to provide the final treatment and disposal of the effluent in a safe manner. The objective of an absorption-field design should be to maximize the use of soil surface that is expected to provide the highest infiltration rate. There are, however, a number of criteria which will influence the design of a suitable soakaway such as:

- General topography and flood plains
- Land slopes
- Vegetation
- Soil absorption rate
- Soil texture and classification
- Estimated flow rate from tank

The absorption field (soakaway) is the most important part of the onsite sewage disposal system. The absorption field is where the actual 'disposal' of the liquid occurs. There are a large variety of absorption-field designs.

Typically, an absorption field consists of a series of sewage distribution lines placed in trenches topped with soil (**Figure 3.28**). The conventional design uses a perforated pipe buried in a gravel-filled trench and backfilled with topsoil. The second is a newer design using plastic leaching chambers instead of gravel to hold the liquid effluent until it is filtered through and absorbed by the surrounding soils. There are many variations and different types of absorption fields in use today.

The soil in the absorption field absorbs and filters the partially treated liquid sewage. Other bacteria that live in the soil attack and digest the liquid. After additional bacteriological action and filtering in the soil, the once liquid sewage is basically water that returns to natural underground water, is evaporated to some extent or taken up by plants. The disposal area must be large enough to absorb the liquid effluent discharged to it. If the area provided is too small, liquid sewage will ooze to the surface or back up into the house through the sewer, eventually discharging into the house at the lowest plumbing fixture. This can become a nuisance and a health hazard for the entire community.

Clean-out frequency

It is recommended that septic tanks be cleaned every 2 years to 5 years (pumped out). The frequency should be based on the occupancy of the home and how quickly sludge builds up in the tank. When being cleaned, septic tanks should not be washed, scrubbed or disinfected as it is necessary to leave solid matter inside to restart the digestive process.

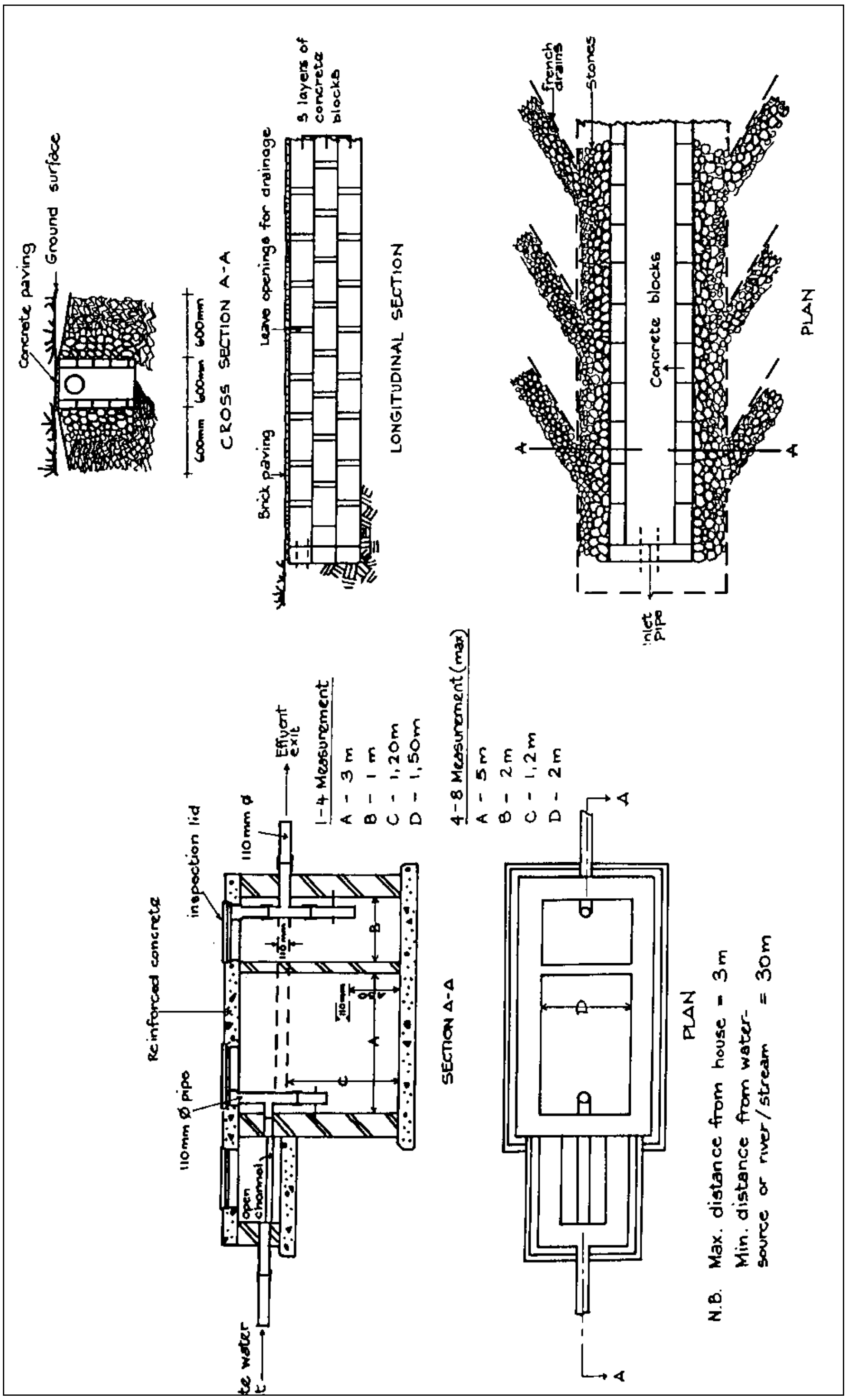


Figure 3.28: A typical example of a septic tank system designed as specified by a West Coast local authority (Wright, 1999)

3.8.2.4 Advantages and disadvantages

There are some advantages and disadvantages of septic tank systems with soakaways. These include:

- ✓ Comfortable and convenient most of the time (except during desludging).
- ✓ Relatively inexpensive compared with conventional sewerage systems.
- ✓ No central WWTW is required.

Septic tanks are prone to certain technical problems which could include:

- ✗ Poor pathogen removal.
- ✗ Inadequate soakaway facilities or the blocking of soakaways.
- ✗ The periodic relocation of soakaways.
- ✗ The desludging of the tank and disposal required by the local authority.
- ✗ The build-up of scum in the tank which inhibits flow and decomposition.
- ✗ Odour nuisance.
- ✗ Backing up of wastewater.
- ✗ Surface flooding or seepage.
- ✗ Local watercourse pollution.
- ✗ Groundwater pollution.

3.8.3 Anaerobic digester

This is basically a modification of the aqua-privy system described below. Commercial systems have been developed which require minimal water for flushing. The retention time of the liquid in an anaerobic digester is usually 30 d to 50 d which improves pathogen removal. The system can be connected to a solids-free system which removes the effluent for off-site disposal or to a soakaway keeping the effluent on-site.

3.8.4 Aqua-privies

This is a system where the toilets are located directly above or slightly offset to a watertight holding tank as shown in **Figure 3.29**. Flushing water is thus not required other than to keep the tank topped up (can use greywater for this purpose). The system can be connected to a solids-free system which removes the effluent for off-site disposal or to a soakaway keeping the effluent on-site.

The main function of the aqua-privy is to provide settlement, stabilization and anaerobic treatment of the solid waste. A watertight tank is essential to keep the wastewater sealed off and to prevent odours from being released.

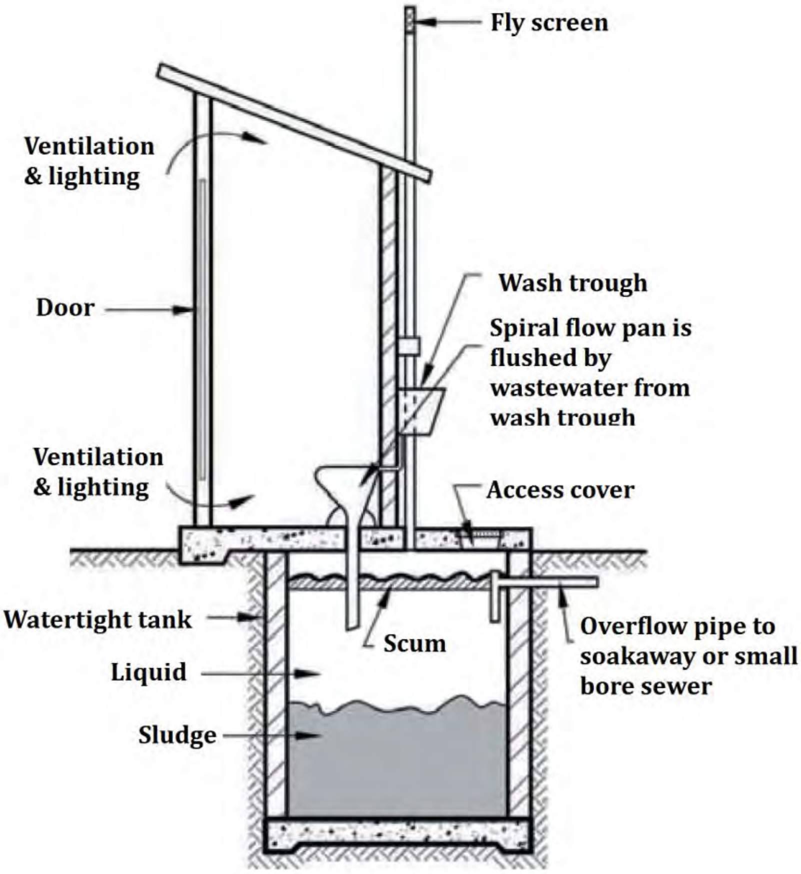


Figure 3.29: Illustration of a sectional view of an aqua-privy (CSIR, 2003)

3.9 Summary

The aim of this section was to provide a summary of some of the most commonly used waterborne sanitation options in South Africa. Although not all systems described above fall within the conventional definition of waterborne (i.e. transported or carried by water) the systems that utilize water were included in this summary.

A more detailed look into each one of these sanitation systems is covered in **Sections 5** to **8** of this guide. Planning information, technical design criteria and worked examples are provided to assist in the design of the most appropriate sanitation system.

4. COSTING OF WATERBORNE SANITATION SYSTEMS

The funding of the capital cost of new sanitation projects in South Africa is provided by the Municipal Infrastructure Grant (MIG). The operation and maintenance costs are required to come from the municipal budget.

As described by Still et al. (2009) waterborne sanitation is more popular with users and politicians, but there is a cost. While it is possible to build the on-site structure and the sewer connection and local reticulation for not much more than a VIP latrine (R7 000 to R9 000 per site is a reasonable budget figure), the additional costs of bulk water and bulk sewer provision and the costs of wastewater treatment can increase the real cost of waterborne sanitation to well over R30 000 per site (Still et al., 2009).

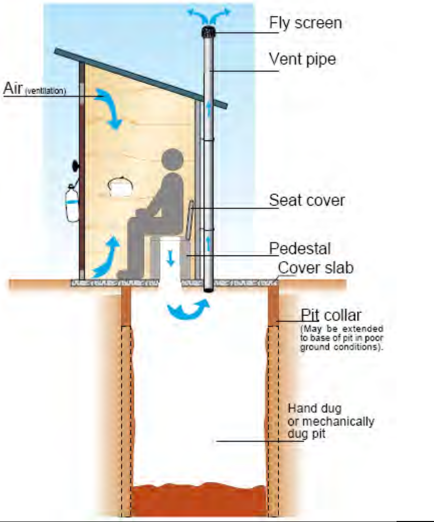
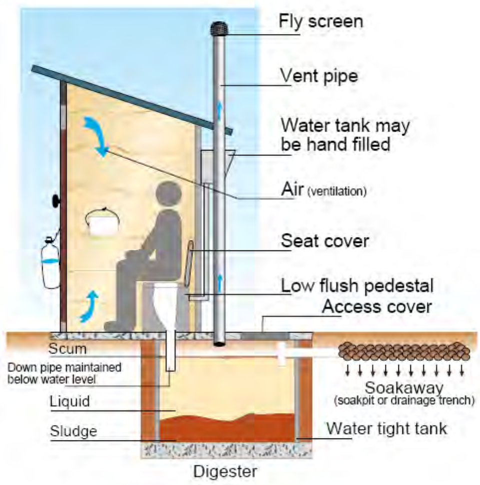
It is obvious that sanitation costs are very site-specific and depend on numerous factors such as:

- Size of population
- Density of settlement
- Design factors
- Methods of construction
- Cost of materials
- Cost of labour
- Soil conditions
- Time factor

The aim of determining the costs of the various alternative options is to provide a point of comparison between sanitation systems. In the costing comparison, the costs that would typically be expected in providing a sanitation system will be reflected. This would include capital costs as well as O&M costs. As part of a research project undertaken by Still et al. (2009) a user-friendly sanitation decision support tool (*Which San?* software) was developed. The tool enables a user to investigate the social, technical and financial feasibility of any sanitation option (grouping all waterborne sanitation options into one). The programme is simple to use, with the user being prompted for data appropriate to the situation in question, and progressively excluding options which are not feasible according to the data provided.

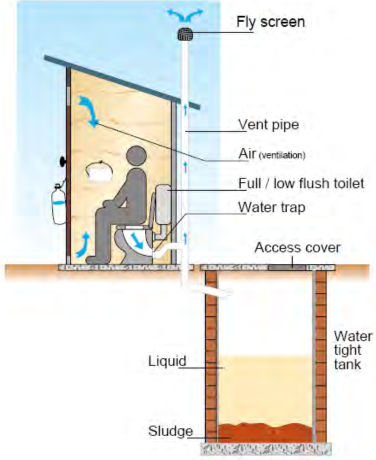
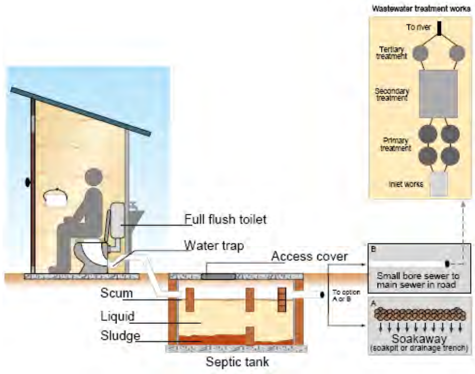
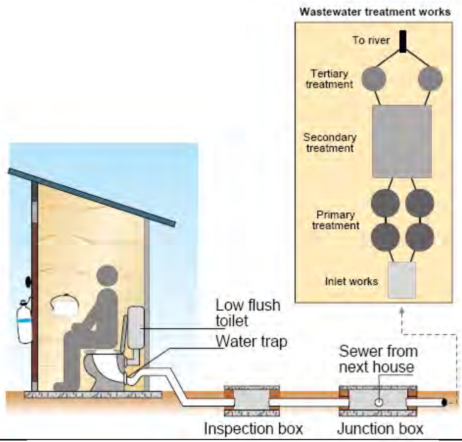
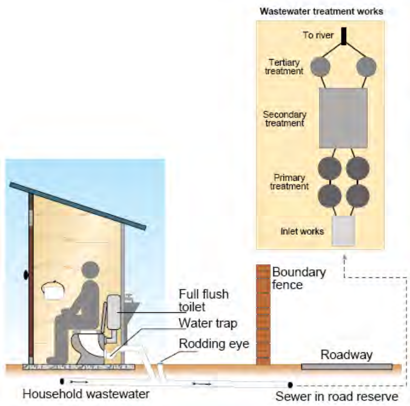
A comparison provided by the Department and Water Affairs and Forestry (DWAF, 2002b) is summarized as shown in **Table 4.1** (based on 2002 cost values and escalated with CPI to 2010 values).

Table 4.1: Cost comparison of different sanitation systems

<table>
<tr><th>Sanitation option</th><th>Capital cost</th><th>Operating cost</th><th>Schematic</th></tr>
<tr><td>Ventilated Improved Pit (VIP) latrine*</td><td>R875 – R4 400 depending on householder input and choice of materials</td><td>R90 per year if emptied every 5 years</td><td>
</td></tr>
<tr><td>Pour flush</td><td>R3 000 – R5 000 which can increase where soils are not suited to drainage</td><td>R220 – R450 per annum where subsoil drainage is available</td><td>
</td></tr>
<tr><td>Aqua-privy and soakaway</td><td>R3 000 – R5 000 which can increase where soils are not suited to drainage</td><td>R220 – R450 per annum where subsoil drainage is available</td><td>
</td></tr>
</table>

Note: * Included simply for comparative purposes since this is the minimum acceptable standard required

Table 4.1: Cost comparison of different sanitation systems (continued)

Sanitation option	Capital cost	Operating cost	Schematic
Conservancy tank	Depends on size and emptying frequency R3 000 – R7 300 depending on top structure and tank volume	R800 per household per annum (based on an estimated emptying cost of R260 per tank) assuming the tank is emptied, on average, 3 times per year	
Septic tank and soakaway or **Septic tank and small-bore solids-free sewer**	R10 200 – R12 400 If septic tank exists then similar, otherwise capital cost even higher	R300 – R650 per emptying, depends on emptying frequency	
Shallow sewerage	R3 600 – R4 400, savings of up to 50% on conventional sewerage capital costs	R440 – R650 assuming all maintenance is provided by the service provider. Reduces to R450 where residents are responsible for operation and maintenance of block sewers	
Full-bore waterborne sewerage	R8 750 – R10 200 taking bulk water and bulk sewer provision and sewage treatment costs into account	R580 – R1 160 per annum	

5. DESIGN GUIDELINES FOR CONVENTIONAL GRAVITY SEWER

5.1 Sewer system planning

The design engineer plans a sewer system layout by selecting an outlet, determining the tributary area, locating trunk and main sewers and determining the need for and location of pumping stations and rising mains. Preliminary layouts are usually made from topographic maps and other pertinent information. The outlet is located according to the circumstances of the particular project and this could be the treatment works, a pumping station or a trunk/main sewer.

Drainage district boundaries usually conform to watershed or drainage basin areas. Trunk mains and interceptor sewers are located at the lower elevations in a given area.

Modelling of the sewer system is required when proposed development intensifies the land use from the existing development on the site or proposed development requires a general plan amendment to a more intense use. The following three scenarios must be modelled:

- **Existing Condition** – to identify existing deficiencies in the system
- **Existing Condition with Proposed Development** – to identify additional deficiencies created by the proposed development
- **General Plan Build-Out Condition** – to identify the ultimate pipe size for improvements

Sewer modelling is required to identify the specific project's impact on the rest of the sewer system. Development in areas with a deficient sewer downstream could be restricted. Where uses are discontinued on a property to allow for new development, new development up to the sewage generation rate of the previous use on the property will be allowed in sewer-deficient areas.

A developer may make the needed improvements to the sewer system at his/her own cost and request a reimbursement agreement to recover a portion of the costs from other developments that tie into the system and benefit from the improvements.

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5.2 Gravity sewer system design (reticulation, link and main)

5.2.1 General

All sewers shall be designed in accordance with established design guides and standards and to accepted engineering principles. In all newly developed areas and/or in all existing areas where new gravity sewers are required, the design shall include the provisions that the sewer system size and capacity can adequately accommodate the ultimate anticipated conditions. It is recommended that under no circumstances any type or form of storm-drain system be connected to any gravity sewer system.

5.2.2 Design of sewers

5.2.2.1 Design criteria

These criteria have been collected from various resources and they are meeting accepted standards for gravity sewer design. The design engineer shall use these criteria to estimate design flows as accurately as possible. The calculations shall be submitted to the local authority for approval. These criteria shall be considered to determine the projected flow:

- Tributary areas
  Tributary area of a sewer shall include all areas that will contribute flow to the sewer system. It shall include flows from the developed area to the point of connection to the main line.

- Estimate of population
  A population estimate should be made for the proposed development and must be as accurate as possible.

- Land use
  Land use contributes and defines the densities of population and the type of users contributing to the flow within the tributary areas. To verify that the projection is reasonable, zoning maps and field reviews may be used.

- Use-specific flow rates for:
  - Residential
  - Commercial
  - Industrial

  Where uses are planned for an area, the average flow rates shown in **Table 5.1** shall be used to estimate flows. Industrial flow may vary significantly per industry type, size and the way in which wastewater is being discharged. The design engineer shall determine the magnitude of the industries' wastewater contribution in the area.

- Infiltration and inflow
  Infiltration/inflow shall be added to the design total flow

- Groundwater
- Major point-source discharge
  Major discharges from future point sources shall be incorporated in the design flow. Future development of major establishments should be ascertained from the available information, including the local authorities' general plan, zoning and land-use maps.

5.2.2.2 Design-flow calculations

Sewers are constructed primarily to convey the wastewater of a community to a point of treatment or ultimate disposal. Wastewater may be characterized as domestic, commercial or industrial in origin. Ideally other extraneous waters such as infiltration or inflow should be excluded insofar as practicable and through local legislation to prevent the discharging of roofs, yards, etc., into the sewer system.

The capacity of the sewer system must be determined from careful analysis of the present and probable future quantities of domestic, commercial and industrial wastewaters, as well as anticipated infiltration and extraneous inflow entering.

The design period for the sewer system needs to be determined and this could be different for the lateral sewers and the trunk mains. Once the design period has been determined the quantity of wastewater that will be conveyed can be calculated. The flow is largely a function of the population served, the population density and the water consumption and thus the sewers should be designed for peak-flow rates corresponding to predictions of these for that specific area. The SABS (1993) provide a set of anticipated sewage flows based on the different water users, see **Table 5.1**.

In South Africa, the average dry weather flow (ADWF) is based on the unit flow from either a single-family dwelling unit or the erven size. The nationally recognized approach adopted in the 'Red Book' (CSIR, 2003) refers to the ADWF per single family dwelling.

As described by Stephenson and Barta (2005) another approach is to apply unit sewer flows in urban areas based on the sewer flows generated by the different land uses by the so-called equivalent discharge unit (EDU = 100 ℓ/d). The EDU values are based on zoning and stand sizes since this allows flexibility in the allocation procedure and a closer calibration to actual flows experienced in the system.

As described by Stephenson and Barta (2005) the common procedure in determining design flow for the development (or enhancement) of a new (or existing wastewater system) is based on the application of a peak-flow factor and the unit average wastewater contribution:

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$$Q_{design} = (PFF)(URE) + INF + IED \qquad \text{...(5.1)}$$

where:

Q_{design} = design flow (ℓ/d)

PFF = peak-flow factor (between 1.3 and 2.5) depending on the land type and use

URE = average contribution from urban residential erf (ℓ/d·EDU or ℓ/d·SFD or specific by-laws unit)

IED = industrial effluent discharge (ℓ/d·erven)

INF = infiltration of groundwater and leakage from plumbing devices (ℓ/d) - see **Paragraph 5.2.2.3**

Development or enhancement of a wastewater system must take into consideration possible future reduction in wastewater flow volumes. Possible reasons why there could be a reduction are:

- Water conservation / demand management
- Increase in greywater reuse in households
- Reduction of infiltration (i.e. groundwater infiltration)
- Reduction of stormwater inflows

The peak-flow factor (PFF), which is the ratio of the expected peak design flow (PDF) to the calculated average daily flow (ADF), can be calculated by the formula developed by Harmon (1918):

$$PFF = 1 + \frac{14}{4 + \sqrt{POP}} \qquad \text{...(5.2)}$$

where:

POP = population in thousands

Some software programs (Sewsan, GLS Software) utilize contributor unit hydrographs to determine the inflow into each sewer pipe. Each land parcel or equivalent erven (EE) has a specific land use which is associated with a unit hydrograph in the numerical model. The unit hydrograph has a maximum flow of unity and describes the (dimensionless) 24 h flow pattern for that land use. The unit hydrograph peak method uses the expected peak flows associated with the land-use types to calculate the volume of the input hydrograph where the AADD method would base the volume of inflow on the annual average daily water demand of all the land parcels serviced by the pipe. In both methods the 24 h flow pattern is determined by combining the unit hydrographs *pro rata* to the number of land-parcel units associated with each land use.

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Table 5.1: Design flows (SABS, 1993)

Flow from dwelling houses or dwelling units with full in-house water reticulation	
Description	**Sewage flow (ℓ/d)***
Low-income group:	
Per dwelling unit, or	500
Per person per dwelling unit	70
Middle to upper-income groups:	
Per person per dwelling unit, or	160
Dwellings with 2 bedrooms	750
Dwellings with 3 bedrooms	900
Dwellings with 4 bedrooms	1 100
Dwellings with 5 bedrooms	1 400
Dwellings with 6 bedrooms	1 600

Note: * An allowance of 15% for stormwater infiltration and other contingencies should be incorporated in the design figures to be used for dwelling houses.

Sewage flow from dwelling units that do not have a full in-house water reticulation	
Level of water supply	**Sewage flow (ℓ/person·d)**
Public street standpipes	12 to 15
Single on-site standpipe with dry sanitation system	20 to 25
Single on-site standpipe with a WC pan connected to water supply	45 to 55
Single in-house tap with a WC pan connected to water supply	50 to 70

Sewage flow from non-residential buildings		
Type of establishment	**Unit**	**Daily sewage flow (ℓ/unit)**
Airports	Passenger	10
Bars	Customer	8
Boarding houses	Person	110
(additional kitchen wastes for non-residential boarders)	Person	23
Cocktail lounges	Seat	70
Country clubs	Visitor	30
	Employee	50
Day schools	Student	37
Department stores	Toilet	1 850
	Employee	40
Dining halls	Meal served	30
Drive-in theatres	Car space	9
Factories (exclusive of industrial waste)	Worker/shift	140
Hospitals, medical	Bed	500
	Employee	40
Hospitals, mental	Bed	400
	Employee	40
Hotels without private bathrooms	Person	110
Hotels with private bathrooms	Person	140
Motels	Bed	90
Offices	Worker/shift	70
Restaurants (toilet and kitchen wastes)	Patron	20
Service stations	Vehicle bay	10
Shopping centres	Parking space	5
	Employee	40
Swimming baths	Person	9
Theatres	Seat	10
Tourist camps or caravan parks with central bathhouse	Person	90

5.2.2.3 Extraneous flows

Extraneous flows can be defined as an excessive inflow/infiltration of water into the existing sewerage system due to uncontrolled surface inflow and/or groundwater infiltration on account of infrastructural deficiencies (e.g. missing manhole covers, damaged pipes due to poor trench bedding, etc.) or incorrect management practices of urban stormwater (Stephenson and Barta, 2005). The inflow of stormwater and infiltration of groundwater into sewers are considered common phenomena and in the South African context are seasonal, and dependent on the precipitation intensity, patterns of land use and other parameters of a drainage catchment. In cases where there is excessive inflow/infiltration this may lead to sewer surcharges, localized flooding and unnecessary pumping where required. At the WWTW, hydraulic overload may adversely affect both the physical and biological treatment processes. Wet weather periods may require overflow bypassing or additional storage capacity and/or treatment capacity. Typical conditions of extraneous flows are illustrated in **Figure 5.1**.

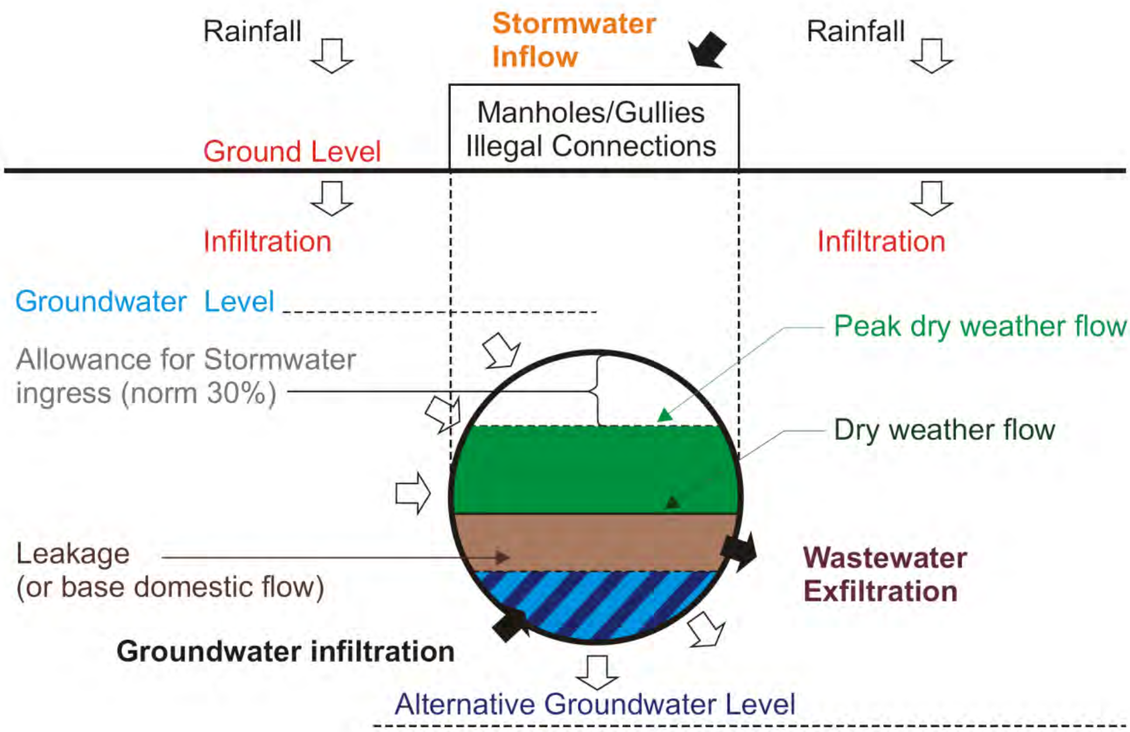

Figure 5.1: Concept of extraneous flows in a waterborne sewer (CTMM, 2009)

Sewer design capacity must include an allowance for extraneous water components which inevitably become a part of the total flow. Proper design and construction will reduce the extraneous water entering the sewer as infiltration through cracked pipes and defective joints or as inflow through cross connections, faulty manholes, illegal discharge points and submerged manhole covers.

Stormwater inflow

In South Africa, the design criteria for large-diameter separate sewers applied by some designers determine the pipe size in gravity mains such that the PDWF occupies 60% to 70% of pipe capacity. The remaining 30% to 40% of the pipe flow area is allocated to stormwater inflows.

Some common reasons for stormwater ingress into the sewer system are:

- Rise in groundwater table
- Illegal connections of house or business gutter down pipes to the sewer system
- Open manholes (mainly due to theft)
- Unwise man-made stormwater canalization and overgrown vegetation in natural channels
- Increase in natural runoff and flood plain due to increasingly larger impervious areas (urbanization)
- Swimming pool overflows may also be a contributing factor as overflows due to rainfall and backwash water are linked directly or indirectly to the sewer drainage system

The direct inflow of stormwater during a rain event can cause an almost immediate increase in flow rates in sewers. According to Stephenson and Barta (2005) a paved area of some 100 m^2 around a broken manhole cover can typically generate about 5 m^3 of stormwater inflow during 50 mm rainfall in one day. The effects of inflow on peak-flow rates that must be handled by a WWTW can be up to 5 times higher than the ADWF.

Groundwater infiltration

As described by Stephenson and Barta (2005) the sewers built in urban areas usually follow the watercourses in the valley close to (and occasionally below) the bed of a stream and could thus receive comparatively large quantities of groundwater, whereas sewers built at higher elevations will receive relatively small quantities of groundwater.

Although the elevation of the groundwater table varies with the quantity of rain percolating into the ground, leakage through defective joints, porous concrete and cracks could be large enough to lower the groundwater table to the level of the sewer.

The rate and quantity of infiltration depend on the length of the sewers, the area served, the soil and topographic conditions, and to a certain extent, the population density, which increases the number of house connections. The workmanship applied during sewer installation, type of pipe, number of joints and pipe size, together with the number and size of manholes all play a role in determining the infiltration and inflow.

The amount of groundwater flowing from a given area may vary from a negligible amount for a highly impervious area or an area with dense subsoil to 25% or 30% of the rainfall for a semi-pervious area with sandy subsoil permitting rapid passage of water. The percolation of water through the ground from rivers or other bodies of water sometimes has a considerable effect on the groundwater table, which rises and falls continually. The presence of a high groundwater table results in leakage into the sewers and an increase in the quantity of wastewater.

Table 5.2 below summarizes available information on infiltration rates obtained from different data sources and brought to the same unit of measurement (ℓ/min per metre diameter per metre pipe), Stephenson and Barta (2005).

Table 5.2: Typical groundwater infiltration values (ℓ/min per metre diameter per metre pipe) – (adapted from Stephenson and Barta, 2005)

Groundwater infiltration	Type of sewer	Remarks on sewer characteristics	Source of information
0.05	Separate	Monitored value from Johannesburg clay/concrete sewers typically 30 to 60 years old	Hine and Stephenson (1985)
0.10	Combined/ separate	Textbook value. No details known of sewer material and age.	Qasim (1986)
0.01-0.70	Combined/ separate	Internationally recognised range of values. No details known of sewer materials and age.	Metcalf and Eddy (1991)
0.05	Separate	Measured value from Cape Town clay/concrete sewers typically 20 to 40 years old	Pollet (1994)
0.03-0.04	Separate	Measured values from Pretoria clay/concrete sewers of 150 mm to 900 mm in diameter typically not older than 40 years	GLS Inc. (1997)
0.02-0.08	Combined	Estimated value for UK purposes predominantly for old clay sewer pipes	CIRIA (1998)
0.048 to add to design rate	Separate	Design allowance mainly for clay and concrete sewer pipelines	Johannesburg Water (Pty) Ltd
0.01	Separate	Permissible wastewater loss from new sewer	SABS (1982a)
15% allowance of ADWF to add	Separate	Predominately for clay and concrete sewer pipes	CSIR, 'Red Book' (2003)
0.04	Separate	-	CTMM (2007)

To prevent possible infiltration of groundwater at manholes the City of Tshwane Metropolitan Municipality prescribes that the contractor should use ‘687/617 Prostruct’ to seal the joints (CTMM, 2007).

Leakage

A proportion of inflow to sewers is generated primarily from leaking toilets and bathroom appliances, building foundation drains, etc. This inflow component is difficult to identify and it is commonly measured with infiltration.

Table 5.3 illustrates various approaches in sizing residential sewage outflow and leakage (or base flow) sewer flow components. The values indicated represent local and international methodology and field measurements should preferably be adopted rather than directly applying the values given (Stephenson and Barta, 2005).

Table 5.3: Sizing of residential water leakage into sewers (Stephenson and Barta, 2005)

Residential outflow (or ADWF)	Leakage from households	Type of sewer and location	Source of data
1.17 ℓ/min per household	0.06 ℓ/min per household	Separate in Johannesburg	Hine and Stephenson (1985)
60-80% of water input	Included in residential flow	Combined/ separate	Qasim (1986)
60-80% of water input	Included in residential flow	Combined/ separate	Metcalf and Eddy (1991)
0.01 ℓ/min per household	Included in residential flow	Separate in Cape Town	Pollet (1994)
0.60 ℓ/min per urban erf (UE)	0.15 ℓ/min per urban erf	Separate in Pretoria	GLS Inc (1997)
0.42 ℓ/min for every 100 m^2 of erf size	Included in residential flow	Average criterion for SA	CSIR, ‘Red Book’ (2003)

Wastewater exfiltration

The age of buried sewer pipes of a municipal wastewater system is considered to be the most significant characteristic governing exfiltration from sewers. Leaking sewers should be of great concern if they are located in any area with high groundwater vulnerability (e.g. close proximity to a groundwater aquifer). Exfiltration can occur when the level of the sewer liquid is above the groundwater table. The positive head created by such circumstances can cause raw sewage to exfiltrate through open joints into the surrounding ground (Stephenson and Barta, 2005).

Exfiltrating sewers may contaminate groundwater with a variety of contaminants including bacteria, nitrates, heavy metals, sulphate and organic compounds. According to (Stephenson and Barta, 2005) traces of the following compounds will indicate sewer-related groundwater pollution:

- Bacteria from domestic sewage (usually measured as faecal coliforms or *E. coli*)
- Inorganic nitrogen species (nitrate and ammonia) from domestic sewage
- Inorganic ions such as sulphate, chloride and potassium
- Phosphate and boron mainly from detergents

5.2.2.4 Hydraulics of sewers

Hydraulic capacity of sewers

The capacity of a wastewater system is based on assessing essential parameters including dry weather flow (DWF), average dry weather flow (ADWF), peak dry weather flow (PDWF) and peak wet weather flow (PWWF), as well as estimates of extraneous flows.

The hydraulic capacity of sewers (i.e. gravity sewers) is usually designed to accommodate the PDWF whilst flowing partially full, as shown in **Figure 5.1**. A portion of the pipe-flow area is allocated to extraneous flows. Over time, this allowance is commonly taken up by infiltration of groundwater leaving very little space for stormwater inflows.

As indicated, the maximum and minimum flow rates in a single day can vary greatly (**Paragraph 5.2.2.2**). There is also seldom control over the content of wastewater that must be conveyed to a treatment plant. The wastewater can contain dissolved solids as well as suspended solids that either settle or float. Most of the dissolved solids and the floating material are transported with the flow. The suspended solids that settle along the sewer pipe invert need careful consideration since the deposition of this material can cause a blockage.

Sewers are usually designed to flow full or nearly full at peak-flow rates and partially full at lesser flows with the flow surface exposed to the atmosphere and thus functioning as an open channel. During extreme peak flows, sewers could in fact surcharge the manholes and the sewers then become pressurized conduits.

There is thus variation in flow rates, deposition of material and frequent changes in slope, different pipe sizes, manholes and other hydraulic control structures that need to be considered in the hydraulic design of sewers.

Flow in sewers can be calculated using either the Manning or Kutter formula with 'n - roughness parameter' or the Colebrook-White Darcy Weisbach or Chezy equation with 'k_s - roughness parameter', see **Tables 5.4** and **5.5**.

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Table 5.4: Friction formulae

Formulae		Parameter and units
Manning	$Q = \frac{1}{n}\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}S^{\frac{1}{2}}$	Q = flow rate (m^3/s) n = coefficient of roughness ($s/m^{1/3}$) A = flow area (m^2) P = wetted perimeter (m) R = hydraulic radius (m) – A/P D = inner diameter (m) S = slope of the energy grade line (m/m) ν = kinematic viscosity of sewage (m^2/s) k_s = absolute roughness of conduit (m) g = gravitational acceleration (m/s^2)
Kutter	$Q = \left[\frac{\frac{1,81}{n} + 41,67 + \frac{0,0028}{S}}{1 + \frac{n}{\sqrt{R}}\left(41,67 + \frac{0,0028}{S}\right)}\right]A\sqrt{RS}$	
Colebrook-White Darcy-Weisbach	$Q = -2A\sqrt{2gDS}\ \log\left(\frac{k_s}{3,7D} + \frac{2,51\nu}{D\sqrt{2gDS}}\right)$	
Chezy	$Q = 18\log\left(\frac{12R}{k_s}\right)A\sqrt{RS}$	

Table 5.5: Recommended roughness parameters (Manning *n*-values)

Pipe	n ($s/m^{1/3}$)
Man-entry plastic lined sewer	0.012
Non man-entry plastic lined sewer	0.013
Plastic	0.013
Standard concrete sewer	0.015
Vitrified clay	0.014

Absolute roughness parameters are included in **Paragraph 9.2** (**Table 9.4**)

- Self-cleansing velocity

As indicated above the deposition of suspended material is of particular concern in the design of sewers. The deposited material at the bottom of the sewer does not remain there if the velocity and turbulent motion are sufficient to re-suspend or move the settled particles along the bottom (ASCE, 1982). This velocity which is sufficient to prevent the deposition of material is called the *self-cleansing velocity*. The calculation method for determining the velocity that is required in a pipe flowing full to transport the sediment is given later in this guide. It is not always feasible to conduct a detailed analysis to determine the minimum velocity and it is often accepted that the minimum self-cleansing full-bore velocity in a conventional sewer is 0.7 m/s.

- Minimum and maximum flow velocities

As indicated above the minimum self-cleansing full-bore velocity in conventional sewers is assumed to be **0.7 m/s**. A maximum flow velocity of **2.5 m/**s has been set although the effect of these high velocities, for short periods, on the sewer pipes itself is insignificant. These recommendations are for gravity and rising mains.

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Maximum and minimum flow depths

It is suggested that sewers should be designed to flow full during peak flows making an allowance for infiltration of stormwater as discussed in **Paragraph 5.2.2.3**. Some local municipal guidelines design for a maximum flow depth of between 60% and 70% of the pipe diameter using PDWF (to have surplus capacity). For partially-full circular channels, a convenient semi-graphical method of solution is provided by the curves describing proportional ratios of discharge, hydraulic radius, area and velocity expressed as a function of the relative depth to diameter d/D (**Figure 5.2**).

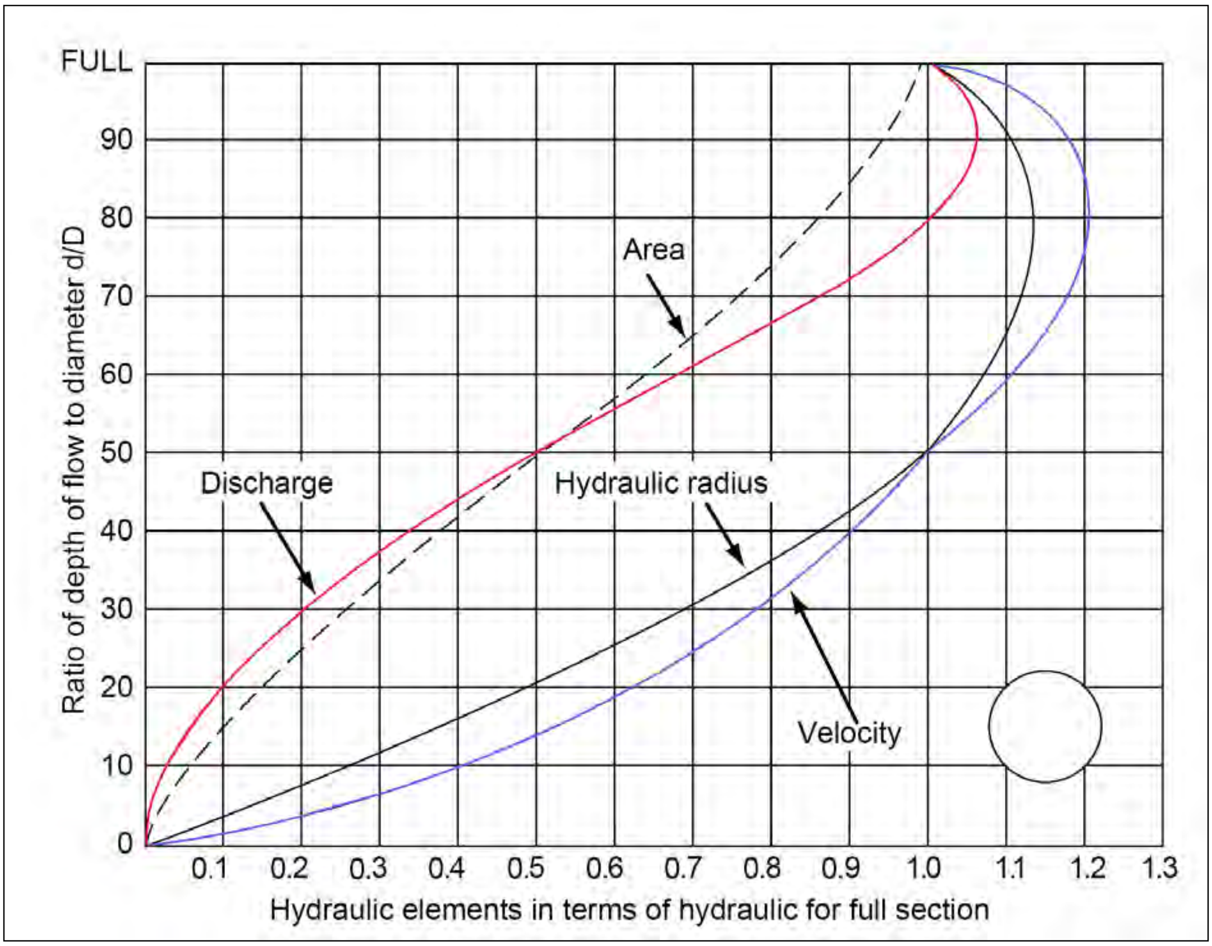

Figure 5.2: Hydraulic elements graph for circular conduits

Partially full sewer pipes

For a pipe that flows partially full the D in the Colebrook-White equation (Equation (5.3)) is replaced with $4R$, the hydraulic radius of the pipe given by the quotient of the cross-sectional area of the fluid in the pipe and the wetted perimeter. The pipe is not pressurized and thus the water surface is parallel to the pipe invert and so the hydraulic grade line (HGL) equals the pipe gradient, Equation (5.4).

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{k_s}{3,7(4R)} + \frac{2,51}{Re\sqrt{\lambda}}\right) \qquad \text{...(5.3)}$$

where:

λ = friction factor (dimensionless)
k_s = absolute roughness of pipe (m)
R = hydraulic radius (m) - $R = \frac{A}{P}$
A = flow area (m^2)
P = wetted perimeter (m)
Re = Reynolds number (dimensionless) - $Re = \frac{4RV}{\nu}$
V = flow velocity (m/s)
ν = kinematic viscosity (m^2/s)

$$S_f = S_o = \frac{h_f}{L} \qquad \text{...(5.4)}$$

where:

S_f = hydraulic grade line slope (m/m)
S_o = pipe gradient (m/m)
h_f = friction loss (m)
L = pipe length (m)

For a pipe flowing partially full the flow velocity can be determined using the Colebrook-White equation to determine the friction factor and a modified Darcy-Weisbach equation to determine the velocity as shown in Equation (5.5).

$$V = \sqrt{\frac{8gS_oR}{\lambda}} \qquad \text{...(5.5)}$$

Utilizing a further ratio of $V_p = \frac{V_d}{V_D}$ for proportional value of a partially full pipe with depth, *d*, and the full pipe depth (*D*) as shown in **Figure 5.3** provides:

$$V_p = \sqrt{\frac{\lambda_D}{\lambda_d} R_p} \qquad \text{...(5.6)}$$

Where the subscripts *p*, *D* and *d* refer, respectively, to the proportional value, the full depth (*D*) and the partially full depth (*d*). Similarly:

$$Q_p = \sqrt{\frac{\lambda_D}{\lambda_d} R_p} A_p \qquad \text{...(5.7)}$$

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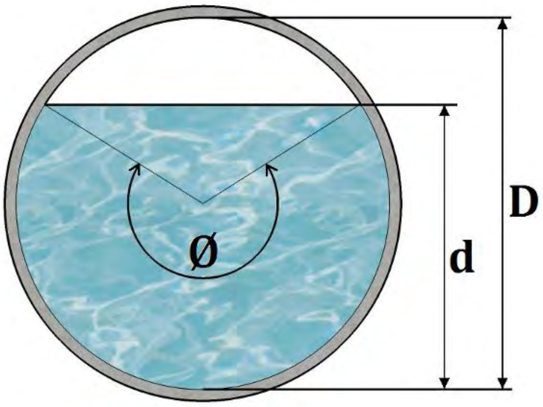

Figure 5.3: Pipe running partially full

For a circular pipe:

$$A_p = \left(\frac{Ø - \sinØ}{2\pi}\right) \qquad \text{...(5.8)}$$

$$R_p = \left(1 - \frac{\sinØ}{Ø}\right) \qquad \text{...(5.9)}$$

where:

Ø = angle of flow (radians)

Substitution of Equations (5.8) and (5.9) into Equations (5.6) and (5.7) allows for the calculation of the proportional velocity and discharge for any proportional depth (*d/D*).

Minimum sewer diameter

The minimum diameter used is 100 mm. A 150 mm diameter is, however, recommended as the minimum diameter for domestic, office, retail or commercial use and a 250 mm diameter for industrial use. Areas with lesser gradients could consider using larger diameters.

Minimum sewer gradient

In general, the sewers could follow the slope of the ground provided that the minimum full-bore velocity is maintained and the hydraulic design capacity is sufficient. However, since a minimum full-bore velocity has been specified a minimum gradient can be back-calculated and these are provided in **Table 5.6**.

Table 5.6: Minimum sewer gradients

Number of dwellings	Minimum gradient*
< 10	1:90
10 – 80	1:120
81 – 110	1:150
111 – 130	1:180
> 130	1:200
Internal sewer diameter (mm)	**Minimum gradient**
200	1:260
225	1:300
250	1:340
300	1:440
375	1:600
450	1:760
525	1:940
600	1:1080
675	1:1280
750	1:1500
825	1:1770
900	1:1920
975	1:2150
1 050	1:2350
1 125	1:2600
1 200	1:2800
1 275	1:3050
1 350	1:3300
>1 425	1:3550

* For a 150 mm sewer

Should circumstances require flatter gradients and lower velocities consideration should be given to:

- Increasing the gradient by increasing the depth of excavation downstream
- Using larger-diameter pipes
- Finding an alternative route; and/or
- Providing a pumping station

A costing analysis is usually carried out to determine whether or not it would be beneficial to increase the diameter and thus reducing the excavation required, since the sewer can now be placed at a flatter gradient.

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5.2.2.5 Pipe material

The general description for specifying the pipes is simply a pipe suitable for the conveyance of sewage, under the particular working and installation conditions to which they will be subjected to in accordance with Sections 3.1 and 3.2 of SANS 1200 LD: 1982 SABS (1982a). Each type of sewer pipe, its advantages and limitations, should be evaluated carefully in the selection of the pipe material for a given application.

Numerous factors are involved in the evaluation and selection of materials for sewer construction and are dependent on the anticipated conditions under which the sewer will operate (ASCE, 1982), for example:

- *Type of wastewater*
- *Abrasion or scour conditions*
- *Installation requirements*
- *Corrosion conditions*
- *Flow requirements (pipe size, slope and velocity)*
- *Extraneous flow requirements*
- *Product characteristics (length, fittings and connections)*
- *Cost effectiveness (materials, installation, maintenance and life expectancy)*
- *Physical characteristics (soil condition, pipe stiffness, loading strengths, etc.)*
- *Handling requirements (weight, impact resistance)*

Furthermore, local authorities would also have their own specific preferences in terms of the pipe material.

Table 5.7: Pipe materials

Material	Description	
Concrete	Reinforced and non-reinforced concrete pipes are used for gravity sewers, and generally larger bulk/outfall sewers. Concrete fittings and appurtenances such as wyes, tees and manhole sections are readily available. For detailed design information please read *Design Manual for Concrete Pipe Outfall Sewers* (PIPES, 2009). Relevant standards - SANS 677:2010 (SABS, 2010a)	
VCP	Vitrified clay pipes are for gravity sewers. Manufactured from clay and shales and vitrified at a temperature at which the clay mineral particles become fused. Clay fittings are available to meet most requirements. Relevant standards - SANS 559:2005(SABS, 2005a), SANS 50295-1:1991, SANS 50295-2:1991 and SANS 50295-3:1991 (SABS, 1991).	

Table 5.7: Pipe materials (continued)

Material	Description	
FCP	Fibre-cement pipes are used for both gravity and pressure sewers. The product was always produced from asbestos fibre and cement, bitumen dipped, minimum-series 4, with triplex coupling. Relevant standards - SANS 819:2010 (SABS, 2010b)	
CIP	Cast-iron pipe is used for both gravity and pressure sewers. Cast-iron fittings and appurtenances are readily available. A cement mortar lining with an asphaltic seal may be specified on the interior of the pipe. Relevant standards - SANS 746:2010 (SABS 2010c), SANS 6594:2008 (SABS, 2008a).	
PE	Polyethylene pipe is used for both gravity and pressure sewers. PE fittings are available and jointing is primarily accomplished by butt-fusion or flanged adapters. Usually solid wall HDPE. Relevant standards - SANS 674:2010, (SABS, 2010d), SANS 10112:2003 (SABS, 2003b), SANS 21138-1:2008 (SABS, 2008b), SANS 21138-2:2008 (SABS, 2008c) and SANS 21138-3:2008 (SABS, 2008d)	
Steel	Steel pipe is rarely used for sewers; when used, it usually is specified with interior protective coatings or linings. Appurtenances include tees, wyes, elbows, and manholes are fabricated from steel. Relevant standards - SANS 51124-1:2008 (SABS, 2008e), SANS 51124-2:2008 (SABS, 2008f), SANS 51124-3:2008 (SABS, 2008g) and SANS 51124-4:2008 (SABS, 2008h)	
DIP	Ductile iron pipes are used for both gravity and pressure sewers. DIP is manufactured by adding cerium or magnesium to cast iron just prior to the pipe-casting process. Cast iron or ductile iron fittings are used with DIP. Linings for the interior of the pipe (cement mortar, epoxies, polyethylene may be specified). Relevant standards - SANS 16132:2010 (SABS, 2010e), SANS 50598:1999 (SABS, 1999) and SANS 1835:2009 (SABS, 2009b)	

Table 5.7: Pipe materials (continued)

Material	Description	
GRP	Glass-reinforced polyester pipes are used for both gravity and pressure sewers. GRP fittings are available with jointing primarily accomplished with a bell and spigot connection hydraulically sealing with an elastomeric O-ring. Relevant standards - SANS 1748-1:2004 (SABS, 2004), SANS 1748-2:2005 (SABS, 2005b).	
PVC	Polyvinyl chloride pipes are used for both gravity and pressure sewers. PVC pressure fittings are available with jointing primarily accomplished with an elastomeric seal gasket joint although solvent cement joints for special applications are available. Relevant standards - SANS 791:2010 (SABS, 2010f), SANS 1601:2010 (SABS, 2010g), SANS 21138-1:2008 (SABS, 2008b), SANS 21138-2:2008 (SABS, 2008c) and SANS 21138-3:2008 (SABS, 2008d).	

No single pipe product will provide optimum capability in every characteristic for all sewer design conditions.

5.2.2.6 Alignment of sewers

The alignment of sewers shall be determined by the need for sewer service, environmental constraints and economic feasibility. There are three major elements to a sewer alignment:

- The route selection
- The horizontal alignment
- The vertical alignment

Each element needs to be considered in detail to ensure an economic alignment that provides the service required.

- Route selection

As detailed by the City of Tshwane the following aspects must be considered as far as the routing of sewers is concerned (CTMM, 2007):

- The sewer must follow the natural fall of the ground (consult the contour plan)
- The sewer must be laid in those properties which will benefit most directly from the sewer
- Road crossings must be kept to a minimum
- There must be minimum interference with existing structures

- The centreline of sewers must be 1.2 m from the rear and side boundaries. In the road-reserve boundary, sewers must be positioned in accordance with the applicable general layout drawing indicating the positions of the various services.
- All other services must be taken into account
- Where the sewer and water lines are to be installed in the same trench, sewer manholes must be positioned so as to allow for a minimum clear distance of 500 mm between the outside of any manhole and the water pipeline
- The final finished levels of carriageways, sidewalks and vehicle entrances to properties, and the depth of sewer inverts below finished sidewalk levels, particularly for steep cross-falls, must be considered in the design of sewers

Horizontal alignment

The most economical horizontal alignment will, generally, be the shortest length possible. The alignment may be varied to accommodate utilities, to maintain traffic safety and convenience, to equalize house-connection lengths on either side and to minimize other appurtenant work. The goal should be a horizontal alignment that provides the necessary level of service yet represents the shortest and most economical length possible.

It is preferable to have sewers constructed with a straight alignment between maintenance manholes. Straight sewers are easier to inspect after construction, cleaning is less prone to damage a straight pipe, and it is easier to locate a sewer that is aligned straight between manholes. There are, however, situations where it is practical and economical to construct sewer lines with curves between manholes. These situations might be to avoid other substructures, to avoid an excessive number of manholes in curved and hillside streets or to avoid using short radius in manholes where high-velocity flow may overtop the channel. Whenever possible the sewer horizontal curve should be concentric with the street horizontal curve.

In normal circumstances a straight alignment between manholes should be used, but curvilinear horizontal alignment is acceptable subject to the following limitations:

- The minimum radius of the curvature is 60 m

- Curvilinear alignment is used only when approved flexible joints or pipes are used.

The minimum radius of curvature attainable is governed by the type of joint specified or permitted, by the pipe lengths, by the maximum bevel permitted, and by the maximum separation of the abutting pipe ends permitted on the convex side of the curved sewer.

When any portion of a vertical curve is located within the limits of a horizontal curve, the pipe joint is being pulled in two directions. The resultant joint deflection is greater than the deflection in either the horizontal or vertical plane. Care must be taken during design to ensure that the true joint deflection does not exceed allowable limits. A useful equation in this respect is Equation (5.10):

$$\cos(R) = \cos(V)\cos(H) \quad \text{...(5.10)}$$

where:

R = resultant or true joint deflection
V = vertical joint deflection angle
H = horizontal joint deflection angle

Vertical alignment

The sewer vertical alignment shall account for substructures, basement elevations, low ground and general terrain of the area being served (Bureau of Engineering, 2007). Generally, the shallowest vertical alignment will be the most economical with a depth of 2.5 m commonly used. The minimum HC depth shall be 1.2 m at the property line measured from the top of the curb. The vertical location of the sewer shall accommodate substructures, especially substructures carrying unstable substances and potable water mains.

Grade changes are usually limited to subcritical flows. Where flow is supercritical, especially when the flow changes from supercritical to subcritical a vertical curve (VC) may be preferable to a grade change. Where a hydraulic jump could occur a VC shall be mandatory.

The cost of constructing a sewer on a VC is greater than a grade change and the excess cost should be considered in making the decision to provide a VC. Additionally, a VC should be provided if it will reduce an excess cut and/or fill. VCs may be either a circular curve or a parabolic curve. All VCs, except for a few cases, are a series of short chords. Where possible, a circular VC should be provided. The greatest advantage of a circular curve is that it allows all joints to be pulled the same amount, thereby greatly facilitating the construction procedures.

Parabolic curves should be provided where the flow is supercritical or where a hydraulic jump may occur. As a parabolic curve more closely approximates a water surface profile in a vertical transition, regardless of the invert slope, the utilization of a parabolic curve will allow a smoother transition of the water surface profile, thereby, minimizing turbulence and other hydraulic losses. Additionally, it may eliminate, or at least reduce the effects of a hydraulic jump.

Alignment losses

The frictional losses are accommodated in the calculation by sloping of the pipe but directional change results in an energy loss. Theoretically this loss can be calculated from Equation (5.11):

$$\Delta E = \left(1 - (\cos\theta)^2\right)\left(\frac{V^2}{2g}\right) \quad \ldots(5.11)$$

where:

ΔE	=	the energy loss at bend (m)
θ	=	the angle of deflection (°)
V	=	the velocity (m/s)
g	=	gravitational acceleration (m/s^2)

5.2.2.7 Pipe cover

The connecting sewer should be located deep enough to drain the full areas of the erf portion on which building construction is permitted. On street boundaries the connection should be located either at a distance of 1.15 m or at a distance of 5 m or more from a common boundary with an adjacent erf, unless the local authority already has an accepted standard location (SABS, 1993).

The City of Tshwane Metropolitan Municipality specifies the minimum depth to the invert of the sewer connection as 1.5 m in road reserve and 1.2 m in mid-block (CTMM, 2007).

The recommended minimum values of pipe cover to the outside of the pipe barrel for connecting sewers are:

- In servitudes: 600 mm

- In road reserves: 1 000 mm

Lesser depths may be permitted but integrated design of all services is required and the pipe should be protected by placing a concrete slab over the pipe or using structurally stronger pipes or by placing additional earth filling over the pipe. Only in exceptional circumstances should pipes be encased in concrete and any encasement should be made discontinuous at pipe joints to maintain flexibility.

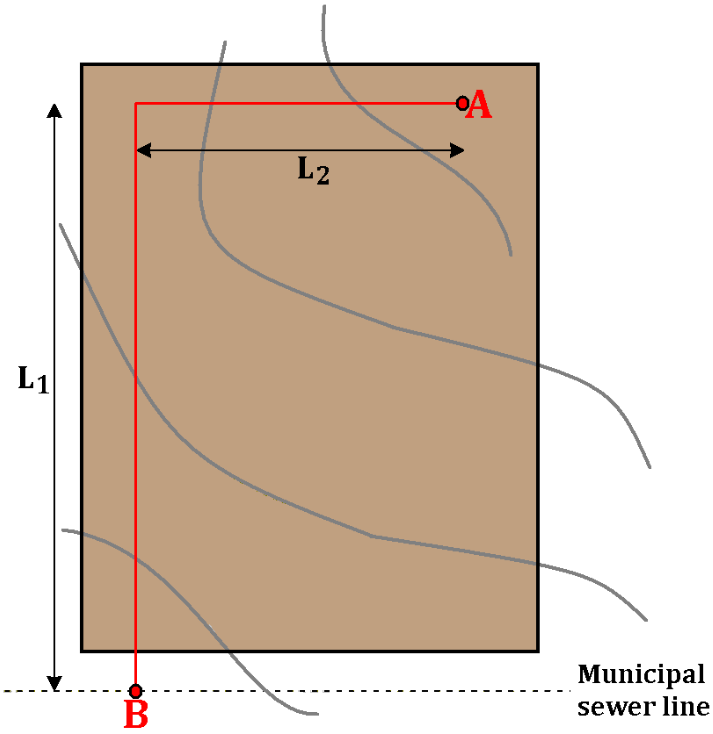

Figure 5.4: Pipe cover at house connection (CTMM, 2007)

To determine the sewer depth at the house connection (see **Figure 5.4**) the following should also be taken into consideration:

- The minimum depth at the extremity of any house drain is 0.45 m (0.3 m cover plus 0.15 m for the diameter and thickness of the drain pipe)
- The minimum gradient for a house drain is 1:60 (SABS, 1982a)
- Allow 0.5 m for the junction at the municipal sewer

$$D_B = \frac{(L_1 + L_2)}{60} + 0{,}45 + 0{,}5 - (Z_A - Z_B) \qquad ...(5.12)$$

where:

D_B = depth of sewer at point B (m)
L_1 & L_2 = maximum length (m) of private sewer as shown in **Figure 5.4**
Z_A = ground level (m) at Point A
Z_B = ground level (m) at Point B (normally the lowest point of the erf)

5.2.2.8 Loading conditions

The structural design of a sewer requires that the supporting strength of the installed pipe, divided by a factor of safety, must equal or exceed the loads imposed on it by the combined weight of the soil and any superimposed loads. The supporting strength of a buried sewer pipe is a function of installation conditions as well as the strength of the sewer pipe itself.

The most widely used method of estimating external loads on a buried pipeline was pioneered by Marston, Spangler and Schlick (ASCE, 1982) at Iowa State University in the US and is generally termed the 'Marston' or 'computed load' method. It is convenient to classify the various types of installation conditions in order to write specialized forms of the general Marston equation as shown diagrammatically in **Figure 5.5**.

The Marston theory states that the load on a buried pipe is equal to the weight of the prism of soil directly over it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prism of soil (ASCE, 1982). The general form of the Marston equation is:

$$W = CwB^2 \qquad \text{...(5.13)}$$

where:

- W = vertical load per unit length acting on the sewer pipe due to gravity soil loads
- W = unit weight of soil
- B = trench width or sewer pipe width depending on installation conditions (m)
- C = dimensionless coefficient that measures the effect of the following variables:
  - i) The ratio of the height of fill to width of trench or sewer pipe
  - ii) Shearing forces between interior and adjacent soil prisms
  - iii) The direction and amount of relative settlement between interior and adjacent soil prisms for embankment conditions

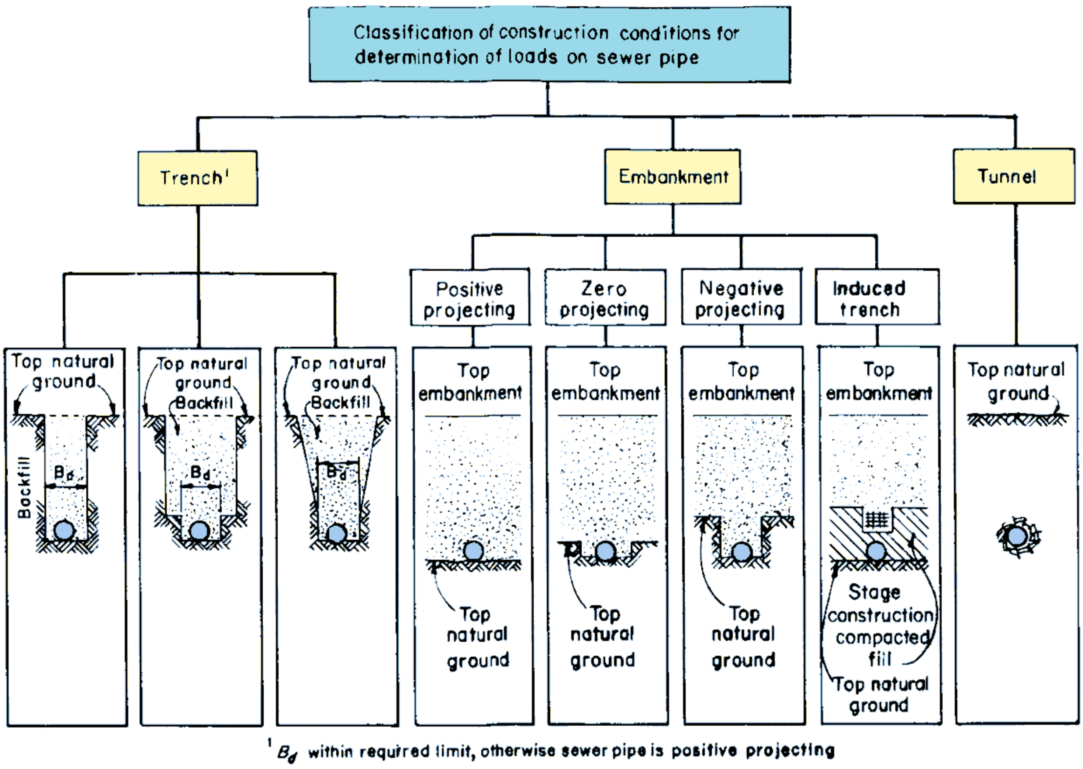

Figure 5.5: Classification of construction conditions (ASCE, 1982)

Detailed structural requirements are discussed in **Section 11** of this guide.

5.2.2.9 Bedding and backfill

The ability of a sewer pipe to safely resist the calculated soil load depends not only on its inherent strength but also on the distribution of the bedding reaction and on the lateral pressure acting against the sides of the sewer pipe (ASCE, 1982). The sewer pipe-soil system focuses attention on the pipe zone which comprises five specific areas, namely foundation, bedding, haunching, initial backfill and final backfill as shown in **Figure 5.6**. Not all these areas are necessarily referred to in all pipe designs.

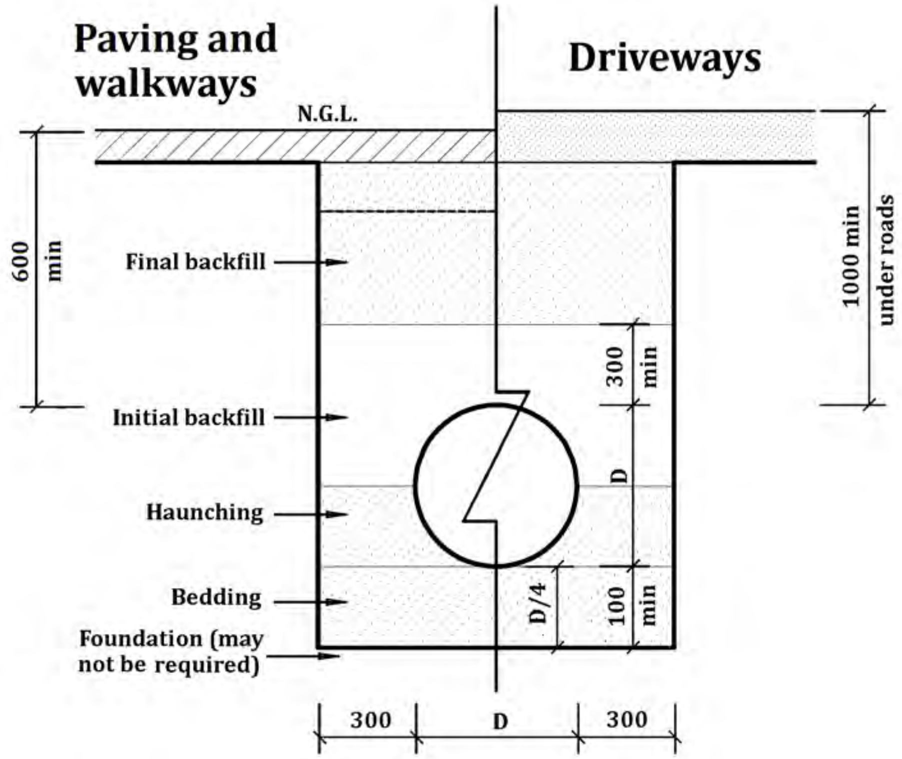

Figure 5.6: Trench cross-section illustrating pipe-soil system terminology

The load-carrying capability of sewer pipes (all materials) is influenced by the pipe-soil system, although the importance of each of these areas may be different for the various materials.

Specifications as set out in SANS 1200LB:1983 (SABS, 1983b) must be followed for the bedding and backfill of sewer pipes. Bedding, backfill and pipe strength shall be sufficient to ensure that pipelines are not overstressed from all superimposed loading.

- Foundation

This provides the base for the sewer pipe-soil system. The designer should be concerned with the possible presence of unsuitable soils such as compressible soils and how a stable trench bottom will be obtained.

Bedding

The contact between a pipe and the foundation on which it rests is called the sewer pipe bedding. The bedding influences how the reaction will be distributed against the bottom of the sewer and thus the supporting strength of the installed pipe.

In general, crushed stone or gravel meeting the requirements of SANS 1200LB:1983 (SABS, 1983b) will provide the most satisfactory sewer pipe bedding. In some areas the natural soils at the level of the bottom of the sewer pipe may be sands of suitable grain size and density to serve as foundation and bedding for the pipes.

In some cases it need not be removed and replaced but simply shaped for the class of bedding that is required.

Haunching

The soil that is placed at the sides of the pipe from the bedding to the centreline is the haunching. The way in which the material in this layer is placed could have a significant influence on the performance of the sewer pipe, especially in the space just above the bedding. For flexible pipes, compaction of the haunching material is essential and in the case of rigid pipes the compaction can ensure a better distribution of the forces on the pipe. The space limitation forces the compaction of this material to be done manually. The material used for haunching may be crushed stone, or a well-graded granular material of intermediate size.

Initial backfill

The initial backfill is material that is placed from the level of the haunching material to about 300 mm above the crown of the pipe, depending on the class of bedding specified. The function of this layer is to anchor the pipe, protect it from damage during subsequent backfilling and to ensure a uniform load distribution on top of the pipe.

Final backfill

The selection of material and placement method of the final backfill is related to the site of the sewer line. Usually the final backfill does not affect the pipe design except under special embankment conditions or induced trench conditions.

In trenches where groundwater is a problem subsoil drains can be installed. Density tests must be carried out on backfill. Refer to standard drawings for bedding and backfill details (included in **Appendix A** and **B**)

5.2.2.10 Corrosion

In general, a surface exposed to sewer gases is always subject to corrosion. Similarly, any surface that is intermittently wet or dry from liquid sewage is also subject to corrosion. Conduits that normally flow full but may be evacuated intermittently (i.e., during maintenance operations) are also subject to corrosion.

Concrete is the most frequently used material for the manufacture of outfall sewers. Under particular conditions concrete sewers may be subject to corrosion from sulphuric acid (H_2SO_4) formed as a result of bacterial action.

As described in the *Design Manual for Concrete Pipe Outfall Sewers* (PIPES, 2009) there are three sets of factors contributing to this phenomenon: those resulting in the generation of hydrogen sulphide gas (H_2S) in the effluent, those resulting in the release of H_2S from the effluent and those resulting in the biogenic formation of H_2SO_4 on the sewer walls as depicted in **Figure 5.7**.

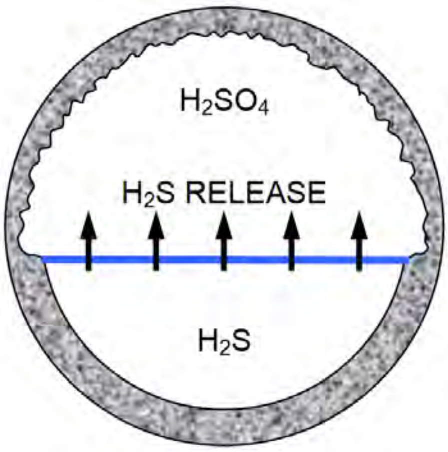

Figure 5.7: Factors contributing to corrosion mechanism (PIPES, 2009)

The most important factors contributing to H_2S generation in the effluent are (PIPES, 2009):

- *retention time in sewer*
- *velocities that are not self-cleansing*
- *silt accumulation*
- *temperature*
- *biochemical oxygen demand (BOD)*
- *dissolved oxygen (DO) in effluent*
- *dissolved sulphides (DS) in effluent*
- *effluent pH*

The most important factors contributing to H_2S release from the effluent are (PIPES, 2009):

- *concentration of H_2S in effluent*
- *high velocities and turbulence*

The most important factors contributing to H_2SO_4 formation on the sewer walls are (PIPES, 2009):

- *concentration of H_2S in sewer atmosphere*
- *the rate of acid formation*
- *the amount of moisture on sewer walls*

The design engineer may specify corrosion-resistant materials or various forms of corrosion protection with consideration of the corrosive conditions anticipated in a specific sewer system. The categories of corrosion control are linings or coatings, composition of materials and/or thickness of sewer pipe materials. Metallic sewer pipes may also be protected from exterior wall corrosion through the use of suitable concrete encasement, insulating wrapping and/or cathodic protection.

To reduce the likelihood of sewers being affected by corrosion, any obstruction in the flow of solids and the creation of turbulence should be avoided.

5.2.3 Maintenance holes and transitions

Certain appurtenances such as manholes, cleanouts, service connections, inverted siphons, and junction chambers are essential for the proper functioning of sewer systems. The aim of a manhole is the following:

- Provide easy access to the sewer for observations and maintenance operations
- Cause as little as possible interference with the hydraulics of the sewer
- Be durable and generally a watertight structure
- Be strong enough to support applied loads

5.2.3.1 Manhole location and spacing

Manholes are located at the junctions of sewers and at changes in grade or alignment. Manholes should also be placed at locations that provide ready access to the sewer for preventative maintenance and emergency service. Care should be taken when manholes are placed in low-lying areas close to a watercourse. In cases where manholes are placed within flood plains the manholes should be raised to a level above the 1:50 year flood level to prevent ingress of flood water.

Manhole spacing varies, reflecting available sewer maintenance methods:

- Manholes are placed a maximum of 150 m apart where the local authority has power rodding machines
- 100 m apart where only hand-operated rodding equipment is available

Additional placing of manholes:

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- All changes in grades or direction
- Positions on steep grades (1:10 or >)
- Higher end of all sections that serve more than three dwellings and that are longer than 50 m
- Where a sewer line crosses a road, at least one in road reserve
- At all junctions
- All changes in grade and/or direction (except in case of curved alignment)
- Decrease the spacing of manholes on steep grades to ensure that the head does not exceed 6.0 m under blockage conditions

Generally, no manholes are to be located on private property, but if required, they are to be placed a maximum of 1.5 m from boundaries, but never on a boundary line.

5.2.3.2 Shape and dimensions

The size of the manhole shafts and chambers should be designed to allow for easy access and working space. Depending on the shape (circular or rectangular) minimum internal dimensions are prescribed, shown in **Table 5.8**. The aim is to provide sufficient space for a person to work in.

Table 5.8: Minimum manhole dimensions (internal)

Shape	Shaft	Working chamber
Circular	750 mm	1 000 mm
Rectangular	610 mm	910 mm

Most manholes are essentially cylindrical in shape, with the inside dimensions sufficient to perform inspecting and cleaning operations without difficulty. A typical manhole for small sewers (≤ 300 mm) is shown in **Figure 5.8**.

In some cases a shallow manhole is required and then the standard design does not provide sufficient space in which a person can work effectively. An extra-large cover with larger access opening helps improve these conditions.

Typical values for H as shown in **Figure 5.8** are:

- H = 50 mm to 75 mm within erven and road reserves
- H = 250 mm in low-cost housing developments and undefined roadways
- H = 500 mm in open veld or road reserves where position of manholes can be concealed by long grass or other growth

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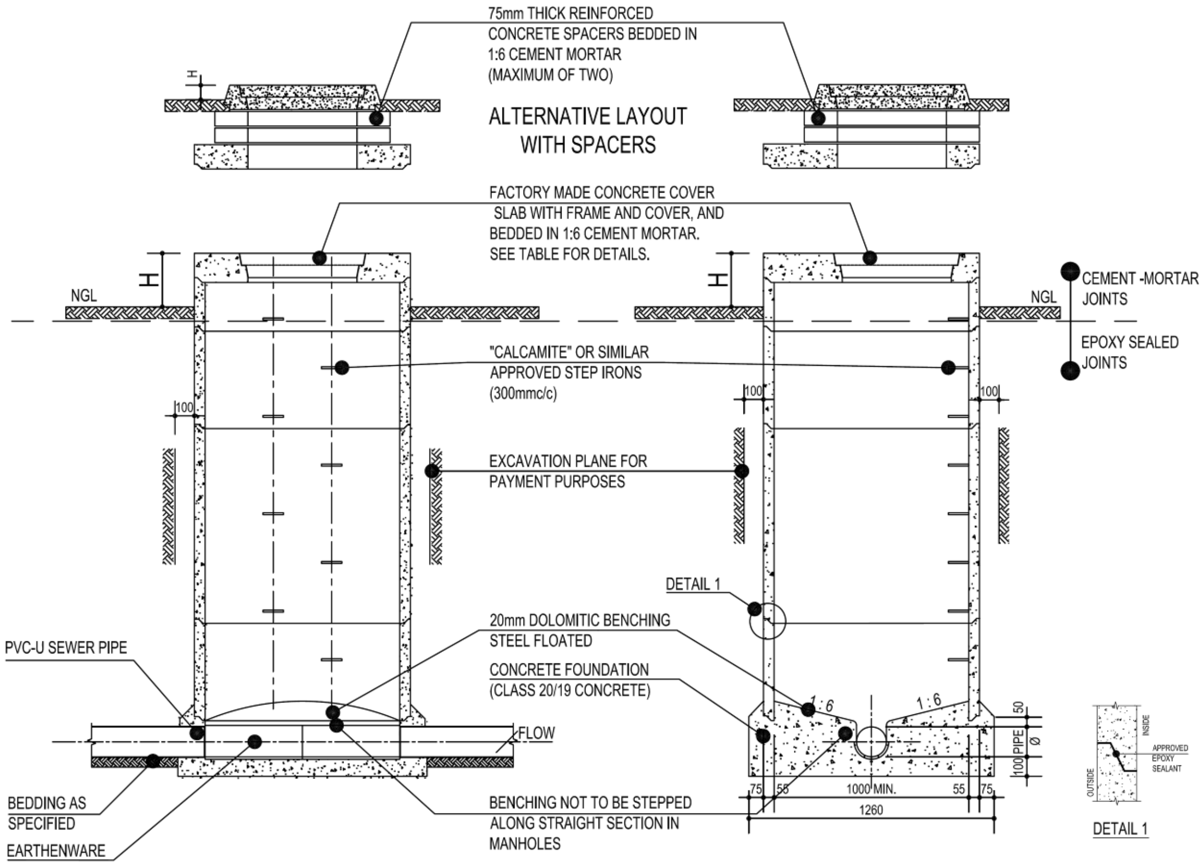

Figure 5.8: Typical circular manhole (CTMM, 2007)

Benching in the manhole is required to allow a maintenance worker to easily stand safely on such benching while working in the manhole and for hydraulic reasons to prevent deposition (self-cleansing). Manhole benching should have a grade not steeper than 1 in 6 nor flatter than 1 in 25, and should be battered back equally from each side of the manhole channels such that the opening at the level of the pipe soffits has a width of 1.2 *d*, where *d* is the nominal pipe diameter.

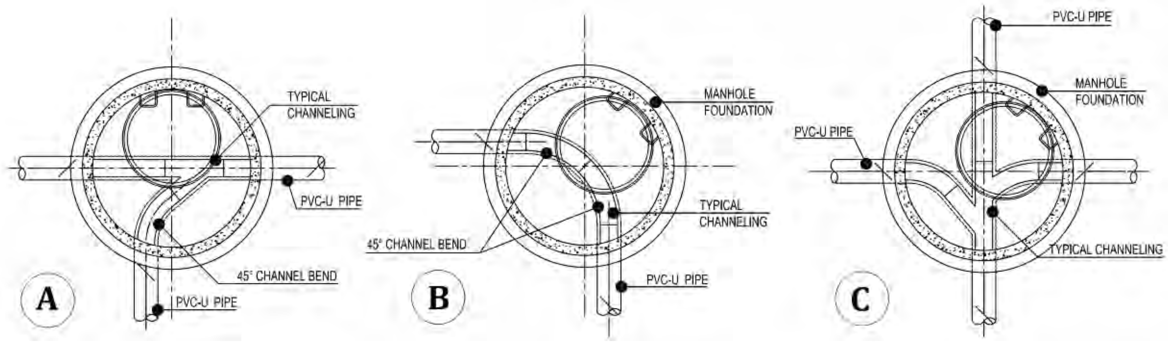

Figure 5.9: Typical sewer channelling (CTMM, 2007)

Figure 5.9 depicts typical sewer channelling: a) 90° connection in manhole; b) 90° bend; c) side connections in manhole.

Manholes deeper than 5.0 m and trunk sewers > 600 mm would usually require specialised design of the manhole.

5.2.3.3 Construction material

The material that is most commonly used for manhole walls includes precast concrete sections and cast-in-place concrete (as per SANS 1294:2006 (SABS (2006)). Manholes are also constructed in brickwork, polyethylene and fibreglass (SABS (2002) and SABS (2003a).

It is difficult to make a brick-built manhole watertight even though it has been plastered with cement mortar, and precast manholes may leak because of imperfect sealing of the joints. The joints of precast sections can be sealed using elastomeric gaskets, joint filler or an epoxy sealer such as Epidermix 344 or Pro-struct 687.

All materials used for manholes should be in accordance with Section 3.5 of SANS 1200 LD:1982 (SABS, 1982a). Precast manholes shall be made with dolomitic aggregate as described in SANS 1294:2006 (SABS, 2006). It is recommended that dolomitic aggregate and low-alkali sulphate-resisting cement shall be used for all concrete, mortar and screeding.

A channel with good hydraulic properties is an important objective to obtain during the construction. Insofar as it is possible the channel should be a smooth continuation of the pipe. Channels can be formed using vitrified clay (earthenware) or even uPVC pipes for the smaller diameters (≤ 300mm). For larger diameters *in situ* cast dolomitic concrete is proposed. The completed channel cross-section should be U-shaped. Some engineers specify a channel constructed as high as the centreline on small pipes; others require that the channel be at least ¾ of the diameter (D).

Buoyancy shall be considered and flotation of the manholes shall be prevented with appropriate construction where high groundwater conditions are anticipated.

Manhole step irons to be cast iron, 'Calcamite', stainless steel or similar.

The manhole frame and cover is normally made of concrete, cast or ductile iron. Alternative materials for manhole covers and frames may be used, provided the cover complies with the strength tests as per SANS 558:2009 (SABS, 2009a). The objectives of the frame and cover are:

- Provision of an adequate opening for access to the sewer
- Providing a good fit between cover and frame
- Preventing earth and gravel from falling into the sewer when the manhole is opened

- Providing sufficient strength to support superimposed loads
- Preventing unauthorized entrance
- Provision for opening of the cover

Due to the potential for differential settlement of the manhole and the sewer a pipe joint just outside the manhole will allow flexibility preventing the sewer from breaking. For very unstable soil conditions a second joint within 1.0 m from the first may be necessary.

Large sewers have other requirements and usually require the manholes to be designed individually. Sometimes a platform is provided on one side, or the manhole is simply a vertical shaft over the centre of the sewer. Large T-sections can also be used. In cases where the small-sewer type manhole is used for large sewers, the diameter of the manhole is increased (see **Figure 5.10**), to maintain an adequate width of bench.

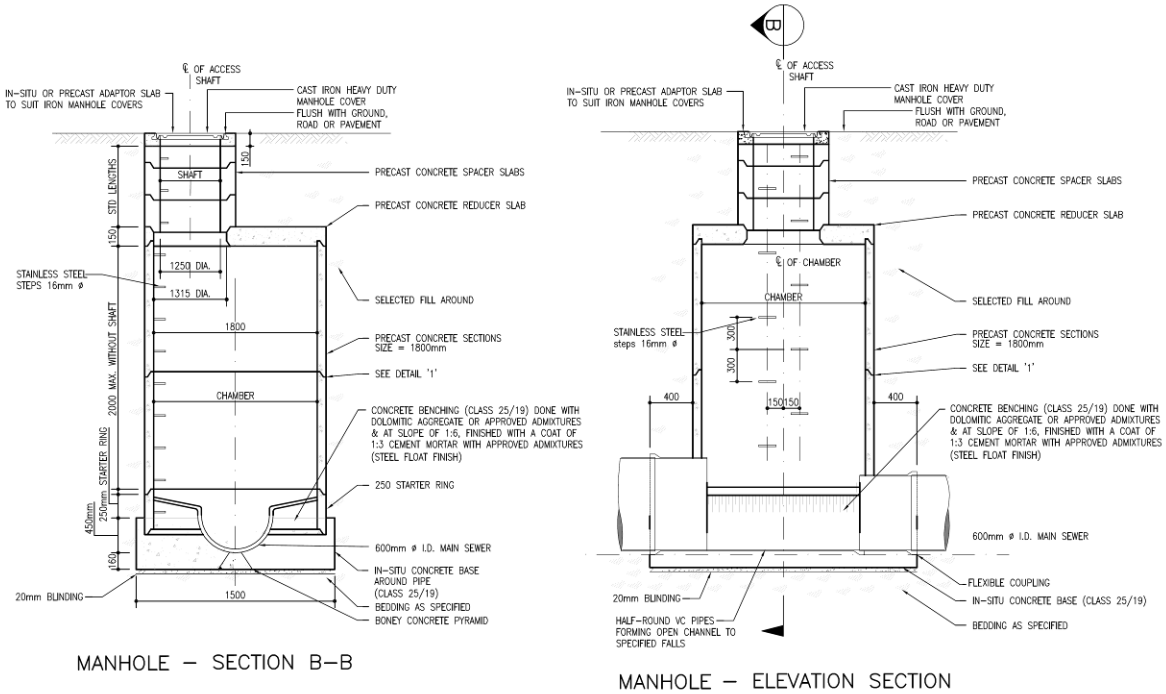

Figure 5.10: Manhole for large sewers

If a sewer enters a manhole at an elevation considerably higher than that of the outgoing pipe, it is generally not acceptable to simply let the sewage pour into the manhole since the structure then does not provide an acceptable working space. Drop manholes are usually provided in these cases, see **Figure 5.11** as an example. If there is a difference of 1.0 m or more between invert of an incoming sewer and an outgoing sewer, a drop manhole should be used. Drop manholes should be constructed with an outside drop connection. Inside drop connections (when necessary) shall be secured to the interior wall of the manhole and access shall be provided for cleaning.

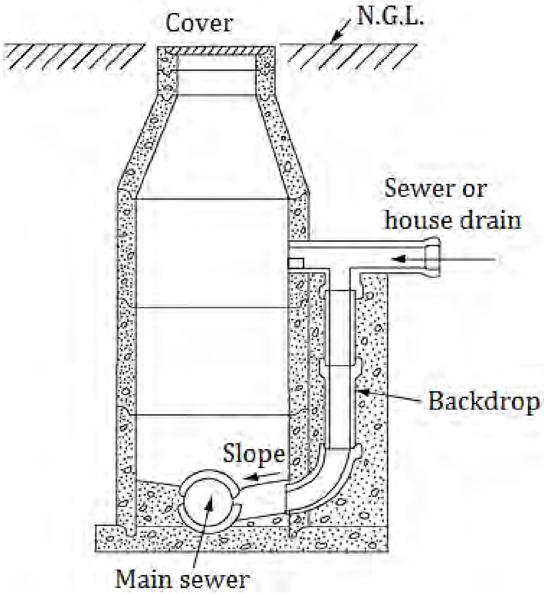

Figure 5.11: Drop manhole

Where corrosive conditions due to septicity or other causes are anticipated, consideration shall be given to providing corrosion protection on the interior of the manholes. Where high flow velocities are anticipated, the manholes shall be protected against displacement by erosion and impact.

5.2.3.4 Fall through manhole

A fall to compensate for energy losses must be made in the channel of all manholes. Some of the design guidelines used in South Africa specify a minimum fall through the manhole which depends on the diameter and the gradient. In some cases it is specified that the actual fall through the manhole should be calculated using the energy equation.

The following is recommended:

- Pipe diameters ≤ 300 mm:
  - Gradients ≤ 1:15 – fall must be 75 mm
  - Gradients > 1:15 – actual fall based on inlet pipe slope or outlet pipe slope whichever is the greatest plus 25 mm

- Diameters > 300 mm:
  - Calculate actual fall using standard energy equation

The energy loss due to change in direction (in metres) can be determined using Equation (5.14).

$$h_b = k_b \frac{V_f^2}{2g} \qquad \text{...(5.14)}$$

where:

h_b	=	energy loss due to bend (m)
k_b	=	energy loss coefficient, which is a function of change in direction
V_f	=	velocity in pipe at full-flow conditions (m/s)

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Values of k_b:

Angle	k_b
0 - 22,5°	0 – 0.1
22.5°- 45°	0.1 – 0.2
45°- 90°	0.2 – 0.4

The standard energy equation is:

$$H_1 + \frac{V_1^2}{2g} = H_2 + \frac{V_2^2}{2g} + h \qquad ...(5.15)$$

where:

h = fall in manhole due to gradient or the minimum fall or S x the diameter of the manhole (m)

S = gradient (m/m)

H and V are design depths of flow and velocities on either side of a manhole, respectively

5.2.3.5 Terminal cleanouts

Terminal cleanouts are also called rodding eyes and are sometimes used at the upstream ends of sewers, although engineers can also specify manholes. The purpose of a rodding eye is to provide a means for inserting cleaning tools, for flushing or for inspection. A rodding eye is a pipe-fitting that is accessible from surface level, and that has a removable, sealed cover, to enable the clearance of obstructions in one direction by rodding along the drain. Rodding eyes should be detailed in accordance with the requirements of SANS 10252-2:1993 (SABS, 1993). A typical rodding eye setup is depicted in **Figure 5.12**.

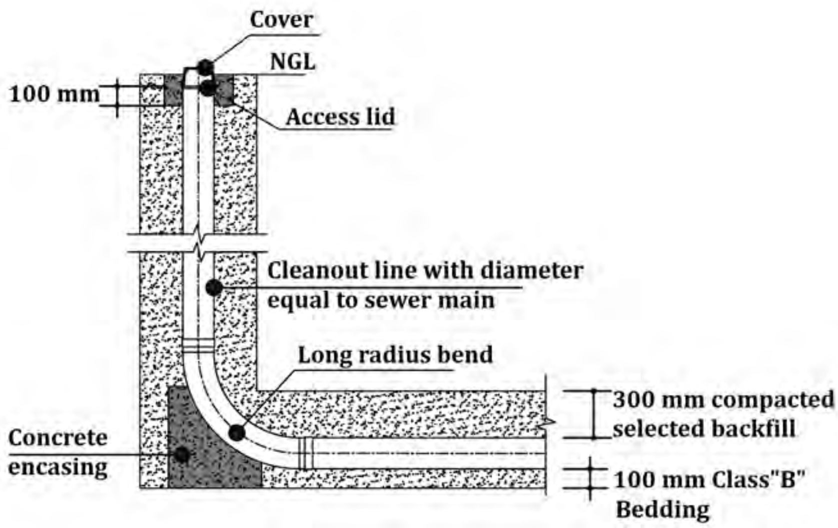

Figure 5.12: Typical rodding eye detail

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5.2.4 Erf connections

Erf connections are also called house connections, service connections or service laterals, and are the branches between the street sewer and the property sewer serving individual properties.

Blockages occurring here are usually due to root penetration, grease or sometimes corrosion (in the case of iron pipes). The material, joints and workmanship should be equal to those of street sewers to minimize infiltration and root penetration.

An example of a house connection for shallow sewers (less than 3 m) which is generally used (**Figure 5.13a**) and for deep sewers (greater than 3 m deep), **Figure 5.13b,** is that provided by the City of Tshwane Metropolitan Municipality (CTMM, 2007).

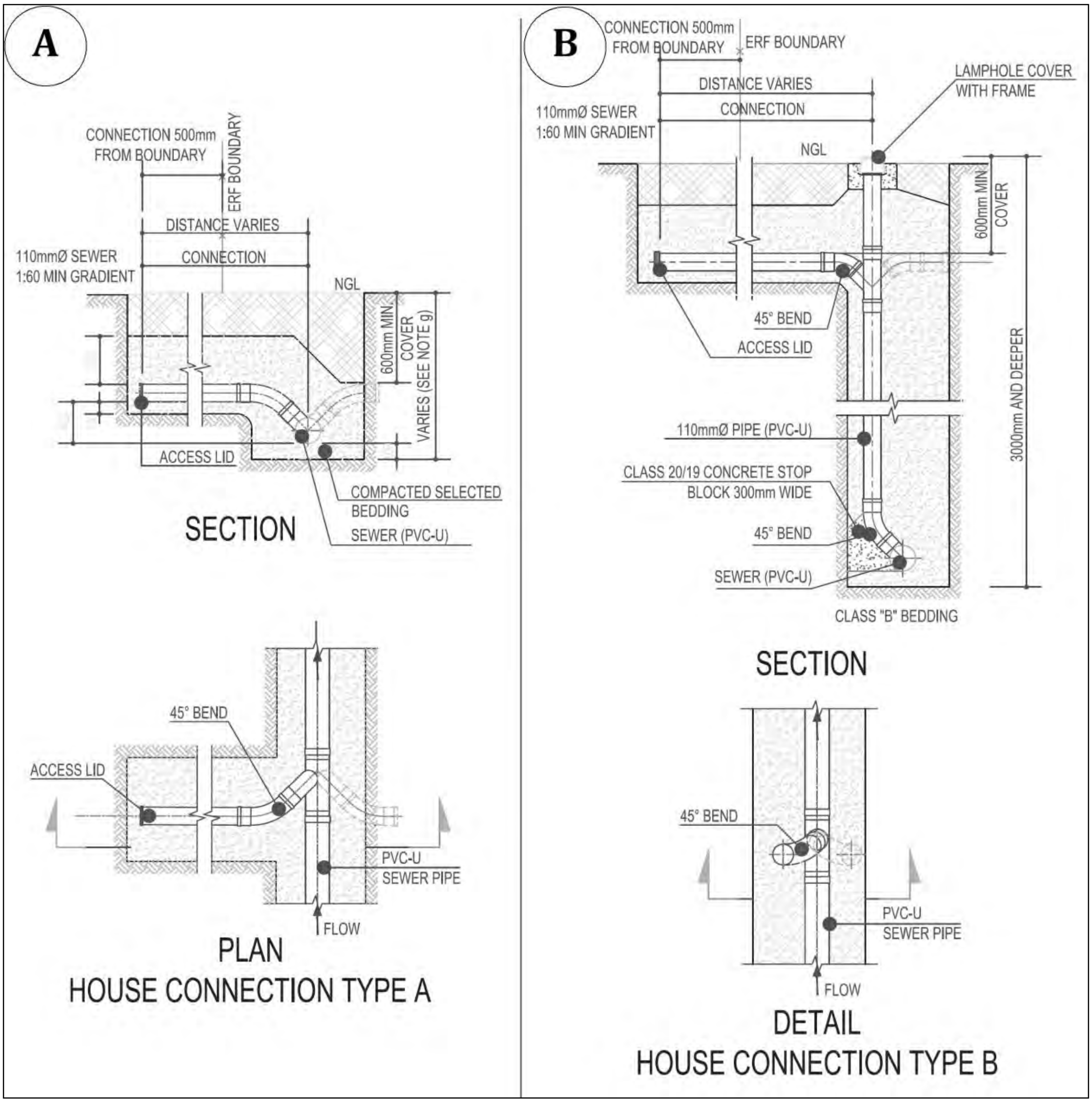

Figure 5.13: House connections (CTMM, 2007)

5.2.5 Field testing and inspection

In some of the guidelines used in South Africa (CTMM (2007); JW (2007)) details are provided of what is required after or during construction of the sewers. Johannesburg Water has the following description of the tests and inspections that should be carried out.

The onus is on the engineer to ensure that the works are properly carried out. Without absolving the engineer of this responsibility, JW shall require the following tests and inspections during the construction phase (JW, 2007):

- *Inspection on completion of the excavation prior to laying the pipe.*
- *Inspection of prescribed bedding and the compaction of surrounding material up to the center line level of the pipe.*
- *Air tests, as prescribed in the standard specification SANS 1200, are to be conducted after the placing and compaction of the backfilling. Contractors are encouraged to pressure test the pipeline prior to backfilling, at their own expense, to avoid excavation of the final backfill.*
- *Density tests must be conducted on backfill. Correlated DCP tests will be acceptable unless otherwise specified.*
- *Inspection of the construction of the manholes and completion of the backfill.*
- *CCTV inspections are to be conducted on all sections of the pipe.*
- *A percolation test must be conducted on sites where septic tanks and French drain systems will be installed.*

The City of Tshwane Metropolitan Municipality specifies that all newly built sewers must be CCTV inspected/surveyed. The requirements are as follows (CTMM, 2007):

- *Colour CCTV cameras equipped with inclinometers must be used, and a pipeline profile must be produced. If ovality as a general fault is present, the City of Tshwane reserves the right to call for laser profiling of the pipelines as well, in addition to the CCTV inspections ('ring of light technology').*
- *For the purpose of CCTV inspection, manholes shall be numbered in accordance with the Division's manhole numbering procedure. This nomenclature requires that each manhole has a unique number consisting of the suburb code followed by the erf number of the stand closest to the manhole followed by A, B, C etc. in the case of more than one manhole in the vicinity of the stand. Pipelines have the same name as the upstream manhole. The latest cadastral data for this purpose is available from the Tshwane Geomatics Section.*

- *The CCTV contractor must produce a status quo report to all concerned containing the normal CCTV report and inter alia recommending which pipes should receive remedial action. The report shall contain maps showing incidents reported on as symbols, according to the Tshwane system of which an example can be supplied if necessary. The layout will be captured in advance by the firms GLS/CEs on GIS, upon being supplied with design drawings by the Consultant.*

- *The CCTV status quo report including CCTV inspection reports form part of the as-built information that must be submitted to the Divisional Head: Water and Sanitation. The inspection reports in electronic format must be submitted in .pdb format as the information has to be uploaded into the consolidated Tshwane CCTV inspection database.*

- *Any attempt to influence the CCTV contractor to inspect only certain lines, to falsify reporting, or threats to withhold payment etc. shall result in the party concerned being blacklisted.*

- *Consultants must recommend to the City of Tshwane the remedial action upon receipt of the report. This recommendation must be based on adherence to the applicable construction specifications.*

- *Any suitable proprietary front end software or camera system may be used but the CCTV inspections must be done according to the Sewer Inspection Manual as used by Tshwane.*

- *A portion of the inspections should be done close to the beginning of the installation and the balance after completion on the 'prevention is better than cure' principle. Consultants are expected to supervise construction in order to ensure that their professional responsibilities are fulfilled. CCTV inspections should only be a confirmation of correct construction.*

- *The costs of the CCTV inspection must be included in the services tender (including possible laser profiling).*

5.3 Sewage pump-station design criteria

This paragraph describes the basic flow capacity, hydraulic design, and equipment/material requirements for new sewage pump-station facilities. The purpose of this section is to establish standard design criteria which set the level of quality for sewage pump-station design. Sewage pumping stations shall be avoided where possible and should only be considered where a gravity system to the existing sewer system is not feasible.

The pump station itself should be as simple as possible but all reasonable measures should be taken in the planning and design to minimise the incidence and consequences of any pollution as a result of wastewater overflows.

Figure 5.14: Aesthetically inspiring sewage pump-station designs

Each pumping station should be designed to match the operation and hydraulic characteristics of the collection system it is serving. Familiarization of the operation of the collection system by the designer prior to detailed design is imperative.

5.3.1 General requirements

The following criteria shall be met when selecting the pump-station site and layout:

- Located to avoid gravity sewers being laid at a depth of greater than 6 m in private property
- Located to avoid associated rising main traversing private property
- Have a minimum impact in the event of failure
- Located within a reserve of the local authority road reserve or on a property which is owned by the local authority for the life of the pumping station
- The site must be accessible by vehicles by means of an all-weather road. The layout of the pumping station and other features must ensure that the available space for maintenance purposes is maximised. A water supply is necessary for wash-down purposes. A hard standing area must also be provided adjacent to the pumping station. Sufficient working space for maintenance personnel and equipment.
- Has metered reticulated water supply

- Top of slab to be above 1 in 100 year flood level and located 150 mm above finished natural surface level. City of Tshwane Metropolitan Municipality specifies 1:50 year flood line (CTMM, 2007) and 1:100 year flood line is the requirement in Ethekwini (Ethekwini Water and Sanitation, 1987).
- Minimise aesthetic issues for neighbouring properties and the general public. Will cause minimum inconvenience to those using it and those operating and maintaining it and those living near it.
- Not encumbered by existing or proposed overhead power lines (e.g. adequate turning radius and overhead clearance)
- Has electricity supply capable of meeting pump-station loading
- Soil conditions are suitable or may be made suitable
- Minimise length of rising main

The pump station shall meet the following design criteria:

- The pump-station structure is to have a 100-year design life
- Be integrated with the local authority's telemetry system
- Be designed with minimisation of long-term maintenance and operation costs in mind
- Meet current environment protection guidelines and regulations
- Have two complete pump sets and associated pipe work with one on duty and one on standby, but capable of operating simultaneously and both capable of pumping raw sewage
- Cleaning and removal of equipment to be achievable without entering the pump station
- Have sufficient storage capacity to prevent frequent pump on-and-off switching
- Minimise noise pollution

5.3.2 Facility capacity and hydraulic design criteria

Each pump-station plant should be designed to deliver the design flow at system head. Wastewater flows from each pumping plant's service area must be carefully investigated. The minimum, average and peak flows corresponding to the dry and wet weather flows (MDWF, MWWF, ADWF, AWWF, PDWF and PWWF) should be estimated in order to determine the size and number of pump units required. When flow from the service area is projected to increase in the future, the pumping station should be designed to accommodate the increased wastewater flows.

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The pump-station structure should be provided with space for future pumps, piping with blank flange, or space for additional hydraulic structures. Present, future and ultimate flows should be calculated. The system head is the total calculated head required to discharge wastewater at a given flow rate through a rising/force main from a given elevation in the wet well. It is the sum of the static lift, the velocity head and the head losses in the rising main. The rising main head losses include friction loss and minor losses caused by valves, fittings, meters and other turbulence- or friction-causing components in the system.

The total discharge head (TDH) of a pump is the total head capable of being developed by a pump at a specific flow rate. The TDH of the pump must be sufficient to overcome the system head at the design-flow rate.

Pump head/capacity (performance) curves are graphical plots of the heads developed by a pump with respect to corresponding capacities. These are normally supplied by the pump manufacturers.

Pump-performance curves also normally show the pump efficiencies, net positive suction heads (NPSH) required and power requirements, all plotted with respect to the corresponding flow rates. Because flows from a given service area vary, the pumping plant must be capable of accommodating a range of flows at the corresponding system heads. A system curve is a graphical plot of the (calculated) system heads with respect to corresponding flow rates.

Fluctuating water levels in the wet well and in the receiving reservoir or pipeline at the downstream end of the rising main must be considered in determining the static lift of a system.

Not only does the flow from a service area and the static lift requirement vary, but the roughness coefficient of the rising main also decreases with ageing piping. The pumping station must be designed to accommodate all these variations expected during the life of the system.

Pumping flow from a service area can be accomplished by three commonly used methods (Bureau of Engineering, 2007):

- *Constant speed pumps with 'fill-and-draw' control*

  This type of pumping station usually requires a larger wet-well storage volume in order to provide enough capacity to limit starting/stopping cycles of the pumps to prevent premature failures of the motors. This method is commonly used for smaller capacity pumping plants with adequate space for wet-well construction.

- *Variable speed pumps with 'matched flow' control*

  Pumping with variable speed pumps is the modern approach, requiring smaller wet wells, and fewer starts and stops of the pump units.

In addition, variable speed pumping plants produce fewer hydraulic surges and smoother flow variation into the treatment plant.

- *Combination constant and variable speed pumps*

In pumping systems with combination constant and variable speed pumps, the variable speed pump is normally used to trim flows in excess of what the constant flow pumps can handle. These plants require a larger wet well than the all variable speed pumping systems.

The pumping configuration should be determined based on analysis of the pump vs. system curves. Various combinations can be used such as multiple pumps, combinations of small and large pump units, all variable speed pumps, and a combination of constant and variable speed pumps. The best choice is the one which provides the optimum overall pump station efficiency, range of operation, and reliability.

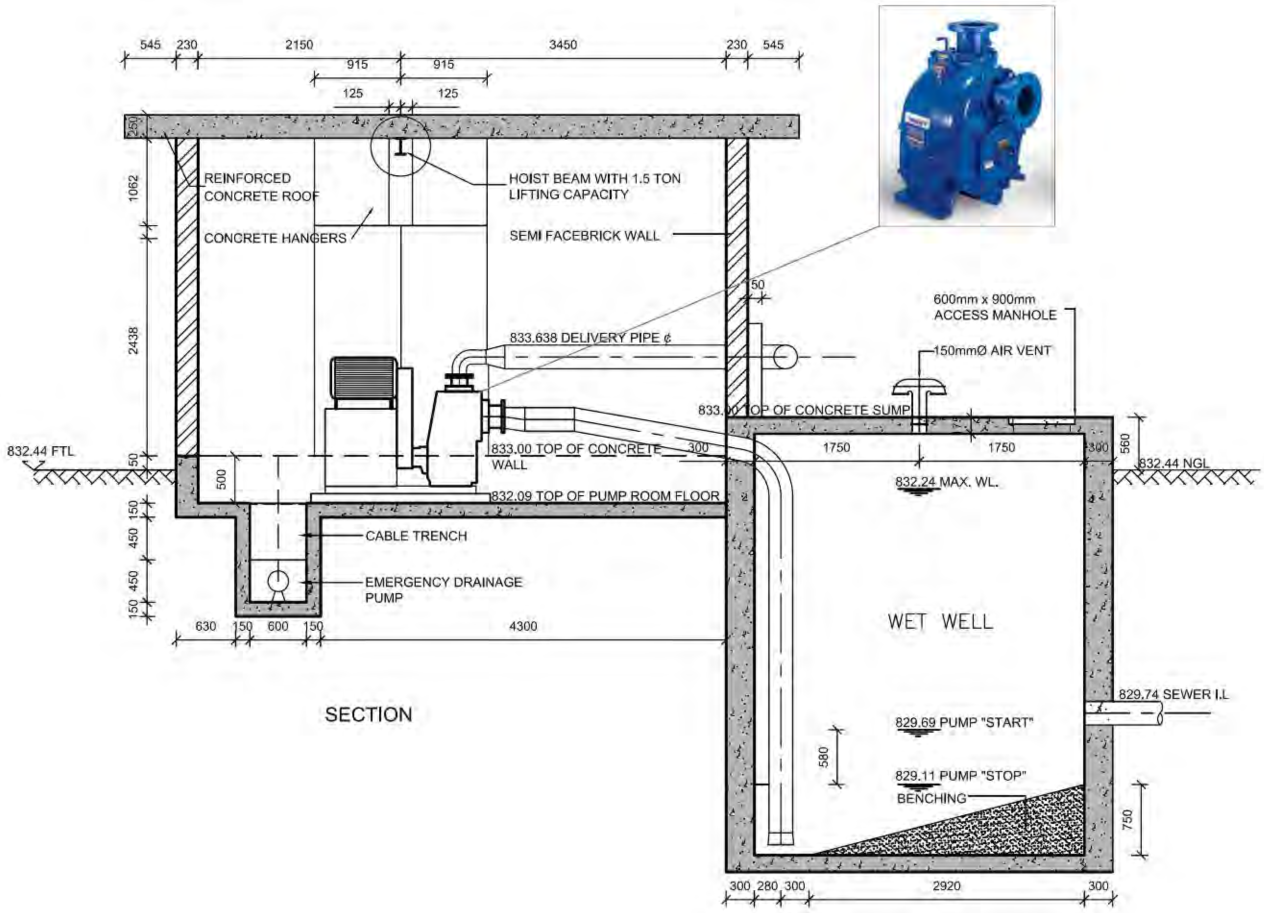

Figure 5.15: Typical dry-well pump station (courtesy CED)

5.3.3 System design and pump selection

From the manufacturer's published performance curves, select the pump with the best efficiency at the design point, or within the operating range where the pumps are likely to operate most of the time, and with the required net positive suction head (NPSH) of at least 1.5 m below the available NPSH at the maximum flow range.

Operating beyond the manufacturer's recommended points in the curve should be avoided because it would shorten the life of the pumps due to cavitation or excessive vibration.

The selected pump curves should be plotted against the system curve. The operating point of the pump is where the pump curve intersects with the system curve. It is important to plot the curves to determine set points of pressure switches, initial settings of pump speeds in the case of variable speed pumps, and setting of pressure-relief valves when required by the system.

For operation and maintenance purposes, it is to the local authorities' advantage to have the same type and manufacturer of equipment in all pumping stations. In this regard, selection of manufacturer and equipment model should be given careful consideration.

The pumping station should be designed to handle the present, future and ultimate flows. Space should be provided to allow for additional pump units to handle future and ultimate flows. The steps to select the most suitable pump are:

- Step 1 - Flow range

  Determine the range of flows which the pumping station should cater for. **Table 5.9** lists the design-flow range.

Table 5.9: Design-flow range

Flows (ℓ/s or m^3/h)	Present	Future	Ultimate
ADWF	Q_{ADWF-P}	Q_{ADWF-F}	Q_{ADWF-U}
PDWF	Q_{PDWF-P}	Q_{PDWF-F}	Q_{PDWF-U}
PWWF	Q_{PWWF-P}	Q_{PWWF-F}	Q_{PWWF-U}

- Step 2 – Determine size of rising main

  Determine the size of the rising main(s) based on velocities and operational criteria. The velocities for the rising main should be kept between 0.7 m/s and 2.5 m/s to prevent settlement of solids during low flows, and the maximum limit to prevent high head loss and other associated problems (surges, etc.).

  The velocity of 0.9 m/s to 1.2 m/s is commonly considered for designing of rising mains. Furthermore, the diameter should be such that solids that are deposited when pumps stop will be scoured out when pumps are working again.

  A matrix of possible diameters vs. flow rates for the rising main can then be set up, as shown in **Table 5.10**, and the associated velocity determined. In some cases a dual rising main may be considered.

Table 5.10: Rising main velocities for various diameter options

Flows (ℓ/s or m^3/h)	Option 1 (D_1)	Option 2 (D_2)	Option 3 (D_3)	Option 4 (2 x D_1)
$Q_{ADWF\text{-}P}$	$V_{1\text{-}ADWF\text{-}P}$	$V_{2\text{-}ADWF\text{-}P}$	$V_{3\text{-}ADWF\text{-}P}$	$V_{4\text{-}ADWF\text{-}P}$
$Q_{PDWF\text{-}P}$	$V_{1\text{-}PDWF\text{-}P}$	$V_{2\text{-}PDWF\text{-}P}$	$V_{3\text{-}PDWF\text{-}P}$	$V_{4\text{-}PDWF\text{-}P}$
$Q_{PWWF\text{-}P}$	$V_{1\text{-}PWWF\text{-}P}$	$V_{2\text{-}PWWF\text{-}P}$	$V_{3\text{-}PWWF\text{-}P}$	$V_{4\text{-}PWWF\text{-}P}$
$Q_{ADWF\text{-}F}$	$V_{1\text{-}ADWF\text{-}F}$	$V_{2\text{-}ADWF\text{-}F}$	$V_{3\text{-}ADWF\text{-}F}$	$V_{4\text{-}ADWF\text{-}F}$
$Q_{PDWF\text{-}F}$	$V_{1\text{-}PDWF\text{-}F}$	$V_{2\text{-}PDWF\text{-}F}$	$V_{3\text{-}PDWF\text{-}F}$	$V_{4\text{-}PDWF\text{-}F}$
$Q_{PWWF\text{-}F}$	$V_{1\text{-}PWWF\text{-}F}$	$V_{2\text{-}PWWF\text{-}F}$	$V_{3\text{-}PWWF\text{-}F}$	$V_{4\text{-}PWWF\text{-}F}$
$Q_{ADWF\text{-}U}$	$V_{1\text{-}ADWF\text{-}U}$	$V_{2\text{-}ADWF\text{-}U}$	$V_{3\text{-}ADWF\text{-}U}$	$V_{4\text{-}ADWF\text{-}U}$
$Q_{PDWF\text{-}U}$	$V_{1\text{-}PDWF\text{-}U}$	$V_{2\text{-}PDWF\text{-}U}$	$V_{3\text{-}PDWF\text{-}U}$	$V_{4\text{-}PDWF\text{-}U}$
$Q_{PWWF\text{-}U}$	$V_{1\text{-}PWWF\text{-}U}$	$V_{2\text{-}PWWF\text{-}U}$	$V_{3\text{-}PWWF\text{-}U}$	$V_{4\text{-}PWWF\text{-}U}$

The minimum diameter for the rising main is 100 mm but some local authorities will allow a reduction in this diameter to 75 mm if a macerator system is used.

The maximum velocities in the suction line are usually limited to 1.5 m/s although a lower velocity is recommended to prevent NPSH-associated problems.

Step 3 – Pump-station system heads

The suction head loss, discharge head loss, friction head losses and static height difference for the selected rising main should be determined (Equation (5.16)).

$$\mathrm{TDH} = h_{l(suction)} + h_{l(discharge)} + h_{f(suction)} + h_{f(discharge)} + (z_2 - z_1)_{static\ height} \qquad ...(5.16)$$

where:

TDH	=	total dynamic head (m)
$h_{l(suction)}$	=	secondary losses on suction side of pump (m)
$h_{l(discharge)}$	=	secondary losses on discharge side of pump (m)
$h_{f(suction)}$	=	friction losses on suction side of pump (m)
$h_{f(discharge)}$	=	friction losses on discharge side of pump (m)
$(z_2 - z_1)_{static\ height}$	=	static height difference between supply and end fluid levels (m)

The secondary losses on the suction and discharges sides can be calculated using Equation (5.17).

$$h_l = K\frac{V^2}{2g} \qquad ...(5.17)$$

where:

h_l	=	secondary loss (m)
K	=	energy loss coefficient (values for pipe fittings are widely published in piping handbooks)
V	=	velocity in pipe (m/s)

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Friction formulae such as the Darcy-Weisbach equation (Equation (5.18)) and Colebrook White (Equation (5.19)) could be used to determine the friction losses in the rising main.

$$h_f = \frac{\lambda L V^2}{2gD} \qquad \text{...(5.18)}$$

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{k_s}{3{,}7D} + \frac{2{,}51}{Re\sqrt{\lambda}}\right) \qquad \text{with } Re = \frac{VD}{\nu} \qquad \text{...(5.19)}$$

where:

h_f	=	friction loss (m)
λ	=	friction factor (dimensionless)
L	=	pipe length (m)
D	=	inner diameter (m)
V	=	flow velocity in pipe (m/s)
ν	=	kinematic viscosity of sewage (m^2/s)
k_s	=	absolute roughness of conduit (m)
g	=	gravitational acceleration (m/s^2)
Re	=	Reynolds number

Fluctuating water levels in the wet well and in the receiving reservoir or pipeline at the downstream end of the rising main must be considered in determining the static lift of a system.

Not only does the flow vary between a service area and the static lift requirement, but the roughness coefficient of the rising main also decreases with ageing piping. Biofilm growth could also have a negative impact on the hydraulic capacity of the pipe and it is better to consider its impact already at the design stage of the system. The pump station must be designed to accommodate all these variations expected during the life of the system.

Step 4 – Set-up system characteristic curves

Plot the system characteristic curves to obtain the design points by utilizing the information and analysis in Steps 1 to 3. Identify possible operational conditions, i.e. ultimate PWWF with Option 2 pipe diameter (D_2) and an increased roughness parameter, etc.

Further economic evaluation can be performed to compare various sizes of rising mains vs. pump-station capital cost, energy cost and O&M costs.

Step 5 – Pump selection

First, determine the type of pump suitable for the application. For example, non-clog type impeller pumps suitable for unscreened municipal type sewage with impeller clear openings capable of passing 75 mm diameter solids. This would be a typical recommendation that the pump have non-clog type impellers able to pass solids up to 75 mm in diameter and have a suction inlet of at least 100 mm.

From the manufacturer's pump head-discharge curve data such as flow on the abscissa, head on the ordinate, a family of curves representing different sizes of impellers plotted diagonally from left to right, impeller efficiencies that intersect the pump curve, and the NPSH required plotted vertically intersecting the pump curves, see **Figure 5.16**. The power curve shows the kW required by the pump corresponding to the size of impeller. Other information such as the pump speed, size of inlet and outlet, and the number of impeller blades is also shown on the manufacturer's pump-performance sheets.

Plot the design point on the pump curve which is the intersection of the flow and head. The design point should be at or near the best efficiency point.

Determine the size of the electric motor drive (remember start-up conditions).

Verify whether the NPSH requirements are met (see **Paragraph 5.3.6**)

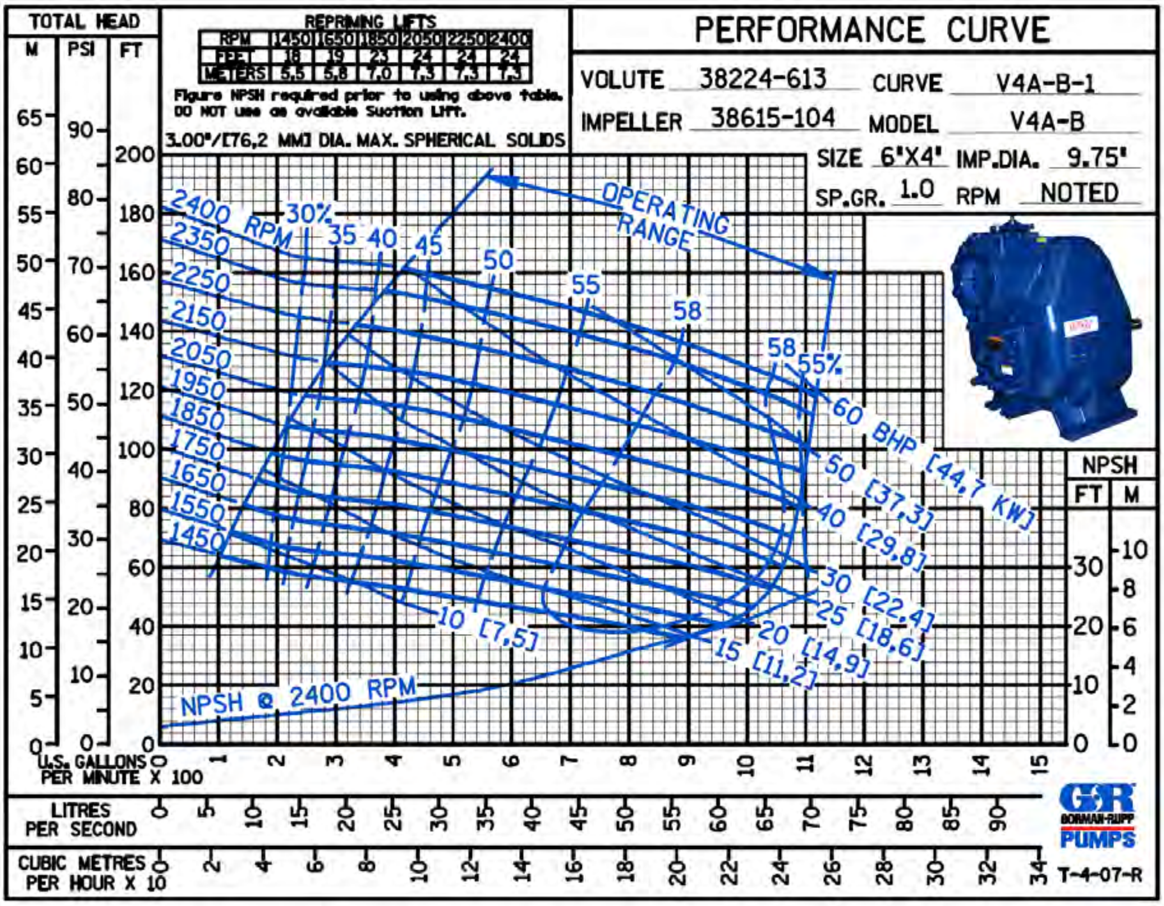

Figure 5.16: Typical pump head-discharge curve (Gorman-Rupp self-priming centrifugal pump)

Determine the number of pumps and configurations to meet the estimated flow range (**Table 5.9**).

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Although each local authority may have its own preference for pumps a favourite choice is to specify the horizontal, self-priming, centrifugal end-suction design pumps (CTMM, 2007). Submersible pumps are only permitted in special cases and usually for 100 erven or less.

The typical design capacity of any pump station is to accommodate PWWF with at least one standby. There should also be a minimum of two pumps permanently installed

5.3.4 Piping and appurtenances

The pump-station piping and support system consists of the gravity sewer, pump suction and discharge piping, plant water or utility water piping, potable water piping, air piping, sanitary drainage piping, fire protection, and sprinkler piping systems. All piping systems, including connections to equipment, should be designed with proper support to prevent undue deflection, vibration, and stresses on piping, equipment, and structures.

The recommended velocities in the piping are:

- Delivery pipes must not exceed 3.0 m/s
- Suction pipes must not exceed 1.5 m/s

The following is recommended for the pipe work in the pump station:

- Ductile iron or mild steel pipe work and fittings are to be used in the pump well, valve pit and between the last manhole and the pump sump
- All pipe work carrying wastewater is to be Epoxy lined
- Polyethylene pipe work could be proposed as an alternative. However, no PE pipe is allowed to be cast through any concrete walls, and as such flanged distance pieces are to be used to provide the transfer through any concrete.
- The discharge pipe work shall be supported at not greater than:
    - 1.5 m intervals for ductile iron or mild steel
    - 1.0 m intervals for PE
    - At all changes in direction
- Where support of pipe work is provided by brackets, the brackets are to be manufactured from at least 50 mm wide by 5 mm thick stainless steel fixed to the pump station structure with stainless steel bolts fastened by either chemical set anchor bolts or ferrules.
- All pipe work connections should be flanged joints within the pump well and valve pit

- A flexible coupling must be provided between the valve on the suction pipe and the pump as well as between the pump and the non-return valve

- The pipe work should allow removal of a pump without affecting other pumps or having to disassemble the pipe work

- The bell-mouth suction inlet should be sized to ensure that the velocity entering the bell mouth does not exceed 1.0 m/s

- A general rule of thumb is that the suction pipe work should be one diameter larger than the discharge pipe work

The following valves can be considered in the pump station and associated works (see **Figure 5.17)**:

- Isolation valve of same diameter as sewer main is to be located on the sewer main either in the manhole immediately prior to the pump station or in the pump station. The valve is to be a sluice valve or where space is insufficient to accommodate a sluice valve, a knife-gate valve shall be installed.

- Non-return valves of the swing-check type with cast-iron casing and bronze disc

- Sluice valves as rising main isolation valves with cast iron casing and bronze wedge

- Air valves will be needed at the pump station and utilizing the ASAP procedure as described by Van Vuuren et al. (2004) for air-valve sizing and positioning will indicate these requirements for the complete system. The type of air valve to be used is, for instance, the Vent-O-Mat RGX or equivalent suitable for sewerage application.

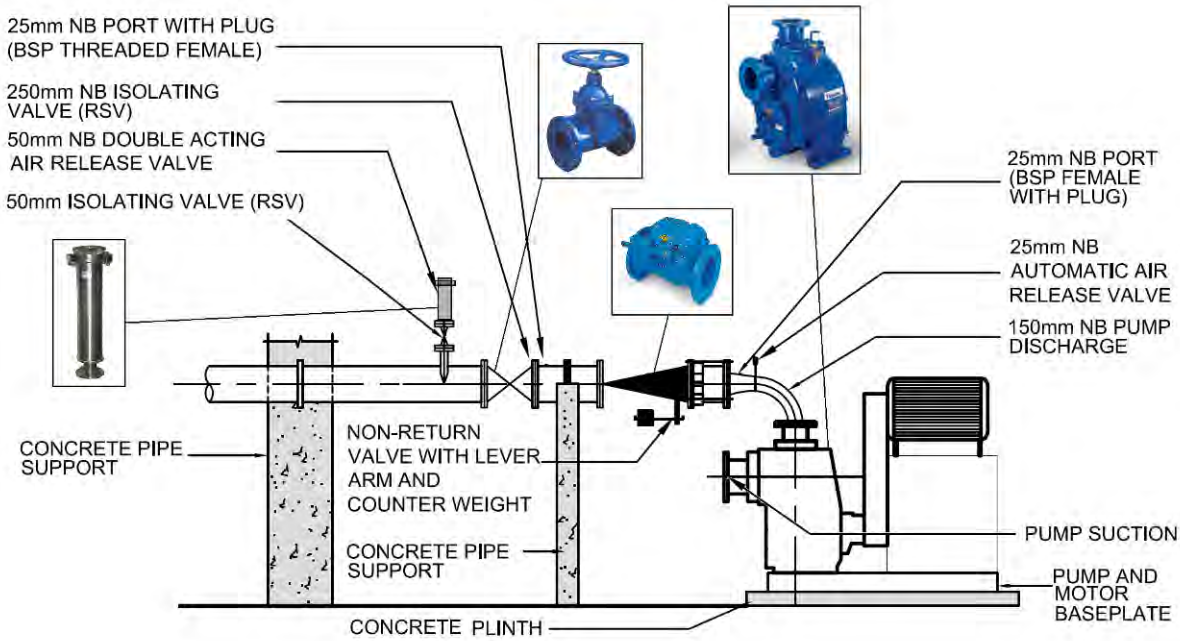

Figure 5.17: Example of discharge pipe work and valves (courtesy CED)

It is also recommended to install a flow meter suitable for sewage, such as magnetic flow meters, on the discharge rising mains for all industrial and large catchment pump stations and to be installed on the pump-station site.

5.3.5 Sumps

Emergency storage

Historical data indicate that most plant overflows are caused by power failures. When an overflow connection into an adjacent sewer system is not available upstream of the pump station, a storage basin may be provided to retain the flow for a predetermined period of time in order to allow the operations personnel adequate time to restore power to the pump station.

The minimum emergency storage capacity required is 6 h. The storage capacity should be based on future ADWF into the pumping station with space for ultimate capacity.

The maximum high-water elevation in the storage facility should be set lower than the top of the lowest manhole in the system, basement, or other plumbing fixture upstream of the pumping station.

For smaller pump stations, for example those draining a building, the emergency storage capacity above the level at which the pump cuts in should be equivalent to the greater of at least 24 h flow at the average flow rate from the building, or at least 1 kℓ (SABS, 1993).

Sump sizing

The required volume of the wet well depends on the way in which the pumping station is to be operated. Pumping stations equipped with constant speed pumps will normally require larger wet wells than those equipped with variable speed pumps.

Each pumping station shall be provided with a sufficiently large wet well to prevent frequent pump starting/stopping (cycling). The wet well should be designed to provide adequate submergence to the pump suction, configured to preclude formation of vortices and flow pre-rotation that could cause pump cavitation. The determination of storage volume of the wet well should be based on the rate of inflow, size of pumps and the type of pump drive.

The following method of computing cycle time and wet-well volume was extracted from the *Sewer Design Manual* (Bureau of Engineering, 2007). The time between starts is a function of the pumping rate and the quantity of flow entering the station. For multiple-speed pumps, the pumping rate is the difference in flow between the two speed steps. The volume of the wet well between start and stop elevations for a single pump or a single-speed control step for multiple-speed operation is given by Equation (5.20):

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$$V = \frac{tQ_p}{4} \quad ...(5.20)$$

where:

V = required wet-well capacity (m^3)

T = minimum time in minutes of one pumping cycle (time between successive starts or changes in speed of a pump operating over the control range)

Q_p = pump capacity (m^3/min) or increment in pumping capacity where one pump is already operating and the second pump is started, or where pump speed is increased

The wet well shall be designed to have adequate storage capacity to sustain the pump operation without exceeding the recommended number of motor starts per hour as shown in **Table 5.11**.

Table 5.11: Recommended number of maximum pump starts (Bureau of Engineering, 2007)

Motor (kW)	Maximum starts per hour	Minimum cycling time (minutes)
Up to 35	6	10
45 to 55	4	15
70 and larger	2	30

It is good practice to also include a maximum retention time in the wet-well design criteria to minimize the potential for the development of septic conditions and the resultant odours. A maximum retention time of 10 min at average design-flow rates is often quoted. Unfortunately, this requirement may conflict with the need for adequate volume to prevent short-cycling of the pumps.

In these cases, multiple pumps or multiple-speed pumps should be considered to reduce the incremental change in the pumping rate and, therefore, the required volume. Also, odours can be minimized if the lowest liquid level in the well is set above the sloping portion of the wet well. This can be accomplished by making this level the stop point for the lead pump in the sequence.

It is recommended that the inlet sewers not be used to provide the wet-well storage.

Johannesburg Water specifies that the inlet to the pump station must be equipped with macerators as well as screens in case of an emergency (Johannesburg Water, 2007).

Self-cleansing

The sump should be designed to minimize solids build-up and be self-cleansing. It can either be trench or hopper style with side slopes ≥ 45° (60° is preferred) towards the inlets of the pumps.

The wet well should be designed to prevent vortexing and/or air binding.

Level controls

The pump station's primary level control should be the ultrasonic level sensing device. The ultrasonic level sensor should be mounted inside the wet well at a distance recommended by the manufacturer above the high water level. The ultrasonic level sensor switch shall be positioned in line with the suction pipe and away from turbulence. The mounting should be designed to allow cleaning of the sensor. Where an ultrasonic device is used, the wet well shall be provided with a float-type level switch to activate the low-low level pump cut-off and the high-high level alarm in the event of failure.

Buoyancy calculations

In cases where there is a high groundwater table and the pump sump is empty there is a risk that the structure may float. The necessary calculations must be done to verify this and if needed anti-buoyancy measures must be specified.

5.3.6 Net-positive suction head

The wet well should be designed to provide adequate submergence of the pump suction to prevent air from being drawn into the pump by a vortex when the system is operating at low wet-well levels. Head losses through the suction piping, fittings and valves shall be calculated for use in the following formula (Equation (5.21)) for determining the net-positive suction head available ($NPSH_{available}$) in metres of liquid absolute in the suction system.

$$NPSH_{available} = \frac{P_a}{\rho g} - \frac{P_v}{\rho g} \pm H_S - h_f - h_l - h_v \qquad ...(5.21)$$

where:

- P_a = atmospheric pressure (Pa)
- P_v = vapour pressure (Pa)
- ρ = fluid density (kg/m^3)
- g = gravitational acceleration (m/s^2)
- H_s = static height (m)
- h_f = friction loss in suction pipe (m)
- h_l = secondary losses in suction pipe (m)
- h_v = velocity loss at pump inlet (m)

The calculated $NPSH_{available}$ should be compared with the $NPSH_{required}$ which is supplied by the pump manufacturer ($NPSH_{available} > NPSH_{required}$).

5.3.7 Electrical, controls, and instrumentation

Electrical systems in the pump station consist of the power supply, transformers, motor control centres, electric motors, electric variable-speed drives, electrical wires and conduits, lighting fixtures, and other associated interfaces with the instrumentation and control systems.

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A general specification for a pump-control panel set-up by City of Tshwane Metropolitan Municipality (CTMM, 2007) reads as follows:

- *All overloads to be rated at motor full load current.*
- *Each drive to have individual stop and start control.*
- *Each drive to have individual run and trip indication.*
- *Panel must have control power "on'' indication.*
- *Panel must have manual and automatic selector switch.*
- *Panel must have duty and standby selector switch.*
- *All motor circuits to be protected by suitable motor protection breaker.*
- *Control circuits to be protected by 2 pole control breaker*
- *Panel must have incoming circuit breaker or isolator protection with door interlock facility.*
- *Panel must accommodate facility to alternate pumps in sequence (flip flop relay) and have the facility to override a pump when a pump is removed for breakdown or maintenance purposes.*
- *Unless otherwise specified, all control panels will provide for high and low level start and stop control.*
- *All wiring will be to SANS specification according to motor size.*
- *All control wiring to be a minimum of 1,5 mm*^{2}*.*
- *All wiring will be terminated into approved terminals with lugs incoming and outgoing.*
- *All terminals and control gear to be dearly marked.*
- *All wiring to be in slotted trunking.*
- *Surge protection must be included where applicable.*
- *Earth leakage protection must be Included where applicable.*
- *Where applicable panels may include "high-high" alarm signals and bath pumps to operate in a high-high condition, and then revert to normal alternating operation under normal conditions.*
- *Level control must be ultrasonic.*
- *All enclosures to be i.p.65*
- *Where applicable, run hour meters must be employed.*
- *Where applicable, kilowatt hour meters must be employed.*
- *Where applicable, phase rotation meters must be employed.*
- *Where applicable, a control philosophy will be submitted with control panel.*

The control system shall typically be programmed to have the following wet-well and other operator adjustable control points:

- *Abnormal low level alarm*
- *Low level pump off*
- *Lead pump start level*
- *Lag pump start level*
- *Abnormal high level*
- *Long on time alarm for either pump*
- *Long off time alarm for either pump*

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Radio-telemetry equipment is to be compatible with the local authorities' existing telemetry systems. The telemetry system is to be capable of transmitting and receiving information from the station's signals and controls.

Provision is to be made to isolate the telemetry system during normal operation of the pump station, including the situation when the pump station is operating from an alternative power supply.

5.3.8 Ancillary equipment

Each pumping station should be designed to provide the necessary ancillary equipment to support the operation and maintenance of the pumping system. This equipment is essential to the operation and maintenance of the system. Ancillary equipment or systems that will be discussed herein are commonly required equipment or systems in larger pump stations.

- Hoisting equipment

  Most pumping plants are located underground to provide adequate submergence for the pump units. Therefore, the substructure and superstructure need to be designed to allow for installation and removal of equipment. The provisions for access hatches, lifting hooks, hoisting systems, roll-up doors and other means to provide ease of maintenance should be carefully investigated and designed as required.

- Odour control

  Careful attention must be given to the design of the necessary odour-control facilities. In the initial design phase a system to control and eliminate potential odour generation at the source can be incorporated. A good hydraulic design and in some cases a phased implementation approach can solve a number of potential odour problems.

  In cases where it is required by the local authority or if the pump station is located near residential areas or other sensitive areas, an odour-control system may be provided. The selection of the odour-control system should be evaluated based on the power and chemical consumption, initial capital cost, maintenance cost, reliability and efficiency of the system.

  For the maintenance of hole-type pumping stations, activated carbon type scrubbers may be used. For larger plants, chemical type scrubbers are commonly used where foul air from the pump station is introduced into the scrubber. A bio-filter system contains typically organic fixed bed materials, which can adsorb the odorous compounds and degrade these compounds biologically and is used in combination with an upstream humidifier (exhaust air-conditioning).

Noise control

The pumping station facility should be designed to meet the minimum noise level requirement of the local authority and the Occupational Safety and Health Act (Act 85 of 1993). All mechanical equipment and enclosures should be acoustically treated to bring the noise level down to an acceptable limit. These attenuation devices may consist of exhaust mufflers, sound isolators or acoustic panels.

There are several ways to design a pump station to meet the noise-level requirement. The location of the pump station is the major contributing factor. If the pump station is located in a residential area, the equipment may have to be located inside an acoustically treated building because architectural sensitivity and sound attenuation can easily be integrated in the building design for optimum noise reduction. When the pump station is located in an industrial or commercial area, acoustic packages may meet the requirements to provide a comfortable workspace for staff.

Ventilation

An automatic pumping station can be expected to have doors and windows closed for at least 23 h in any 24 h period, and as fresh air movement is essential for preservation of the electrical equipment in particular, a system of ventilation must be provided.

The pump-station sump receives and stores wastewater before it is being pumped via the rising main. Corrosive and hazardous gases are normally present in the sump. These gases can become a safety hazard to operating personnel or can cause corrosion of building materials and equipment in the sump. In order to minimize accumulation of gases inside the sump, the sump should be flushed with fresh air by an adequately sized ventilation system.

The pump-station plant dry well is normally located adjacent to the sump to house the pumps, valves, meters and other ancillary equipment. The dry well and equipment rooms should be designed for a ventilation rate of at least 15 air changes per hour or a ventilation rate equivalent to cool internal heat load from the equipment whichever is greater or not greater than 60 air changes per hour (Bureau of Engineering, 2007).

Backup power

Due to the current uncertainty in receiving a reliable supply of energy it is recommended that all pump stations be designed with the facility for emergency power. The larger pump stations should have permanent diesel-oil fuelled, engine driven generator units with automatic transfer switch. The smaller pump stations should be supplied with portable generators.

5.3.9 Wet- and dry-well installations

Pumping stations in sewage collection systems are designed to handle raw sewage that is fed from underground gravity pipelines. Sewage is fed into and stored in an underground pit, commonly known as a wet well. The well/sump is equipped with electrical instrumentation to detect the level of sewage present. When the sewage level rises to a predetermined point, a pump will be started to lift the sewage upward through a pressurized pipe system called a sewer rising main from where the sewage is discharged into a gravity manhole or WWTW. In the case of high sewage flows into the well (for example during peak-flow periods and wet weather) additional pumps will be used. If this is insufficient, or in the case of failure of the pumping station, a backup in the sewer system can occur, leading to a sanitary sewer overflow or the flooding of the pump station.

Traditional sewage pumping stations incorporate both a 'wet well' and a 'dry well'. Often these are the same structures separated by an internal divide as shown in **Figure 5.18** (SABS, 1993). In this configuration pumps are installed below ground level on the base of the dry well so that their inlets are below water level on pump start, priming the pump and also maximising the available NPSH. Normally isolated from the sewage in the wet well, dry wells are underground, confined spaces and require appropriate precautions for entry.

Further, any failure or leakage of the pumps or pipe work can discharge sewage directly into the dry well with complete flooding not an uncommon occurrence. As a result, the electric motors are normally mounted above the overflow, top water level of the wet well, usually above ground level, and drive the sewage pumps through an extended vertical shaft as shown in **Figure 5.15** and **Figure 5.19**. To protect the above-ground motors from weather, small pump houses are normally built, which also incorporate the electrical switchgear and control electronics.

These are the visible parts of a traditional sewage pumping station although they are typically smaller than those of the underground wet and dry wells.

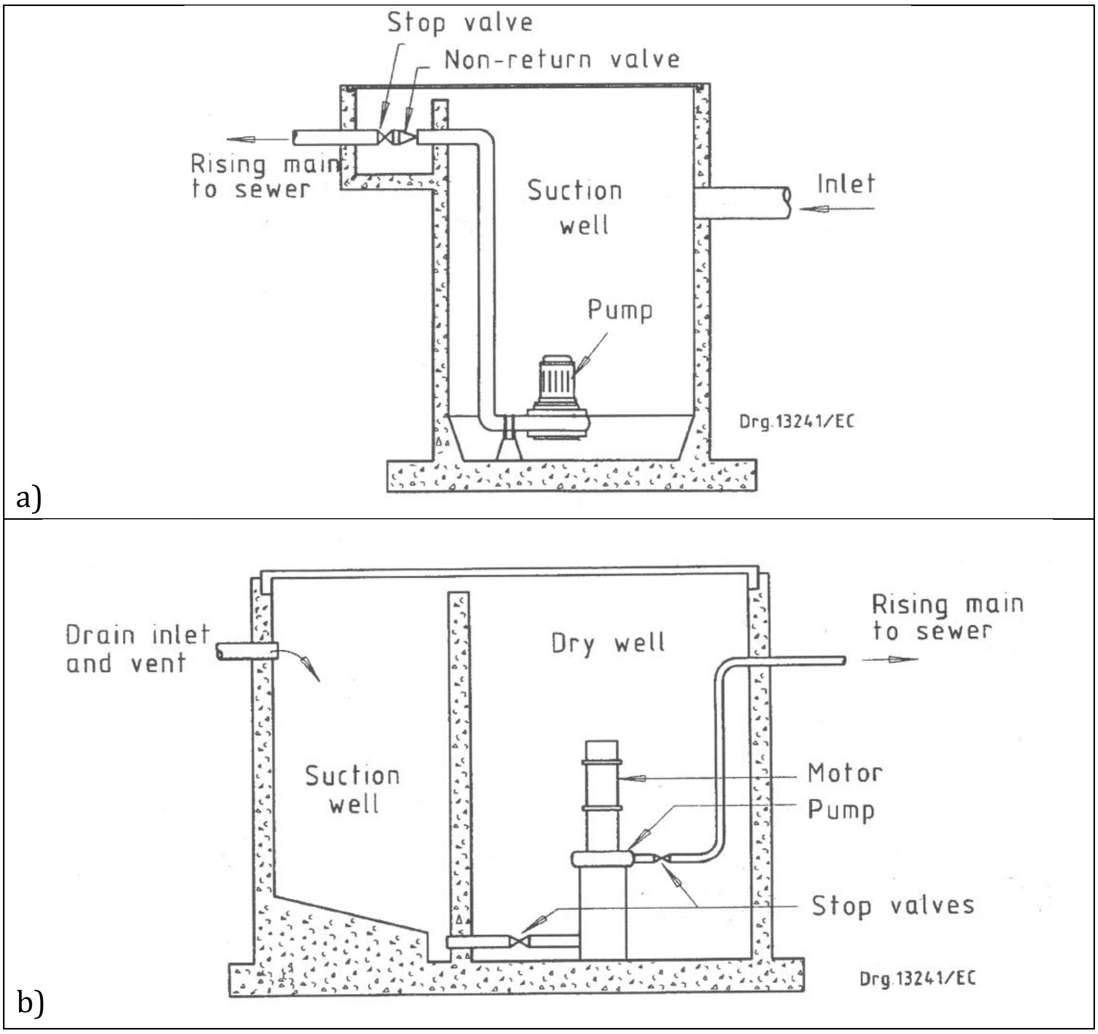

Figure 5.18: Typical a) wet-well and b) dry-well installations (SABS, 1993)

Figure 5.19: Typical dry-well installation (above ground level)

Advantages and disadvantages of wet-well and dry-well pumping stations are summarized in **Table 5.12**.

Table 5.12: Comparisons of wet-well and dry-well pumping stations

	Advantages	Disadvantages
Wet well pump installation	- Lower construction cost since it is not necessary to build a dry well - Requires less area, no super-structure required except for engine-generator or cabinet for motor control - Immersion of pump in the wet well makes pump priming unnecessary and reduces NPSH requirement - No sealed water system, no long shafts with steady bearings required - Quick removal and replacement in emergencies - Neither daily nor weekly maintenance (but overhaul needed every 1 or 2 years) - Quiet operation and safe from flooding	- Valves and headers must be accessible (a) in an adjacent vault, (b) in a small above grade superstructure, or (c) by exposing the header above grade - Pump must be removed and disassembled for inspection and maintenance. Requires a hoist or crane and specially trained mechanics. - Hazard of pumps jamming on guide rails or not seating properly causes ball-bearing damage and shortens pump life - Often more difficult to remove pumps than manufacturers admit - Special motor, seals, and moisture monitoring required (but moisture probes are useless for leaks if power cable is defective) - Vibration has occurred with some makes of pumps larger than 22 kW. Inaccessibility of the pump and motor for routine preventive maintenance and the need for expensive overhaul every 1 or 2 years lead some engineers to refuse to specify their use.
Dry well pump installation	- Easily accessible to maintain the pump - Flexible to use with a wide range of flow rate - Can select from a variety of driving equipment	- The construction cost of dry well pump-station infrastructure compared to the wet-well station of the same pumping capacity is relatively higher - Risk of flooding, especially if the pump house is lower than the minimum water level in the wet well In the case where the pump house is higher than the minimum water level in the wet well then this set-up requires additional pump priming system and associated lower reliability.

5.3.10 Rising/force main

The rising main is the pump stations' discharge piping which conveys wastewater under pressure and discharges from the pumping station to another rising main, another pumping station wet well, a gravity sewer, or into a WWTW.

In order to provide the wastewater system with the reliability and capacity needed, some of the existing rising mains may require rehabilitation or replacement, or additional pumping stations may be required in the future.

The design engineer should make a determination of the size, type of piping material, joints, alignment and method of installation most economically feasible for the project. The rising mains shall be provided with thrust blocks, pig launching and recovery stations for internal cleaning, rising main taps to bypass the pump station, and whenever possible, dual rising mains with isolation valves.

The rising main shall be designed for a minimum velocity of 0.7 m/s to maintain solids in suspension. The recommended velocity of the rising mains would normally be between 0.9 m/s and 1.2 m/s, with the maximum velocity not exceeding 2.5 m/s during intermittent flow conditions. The design engineer should consider the most economical pipe size, material and piping alignment.

The piping material should be designed and selected considering the environment normally encountered in wastewater system applications and buried conditions. The recommended rising main piping materials are reinforced concrete pressure pipe (RCPP) and ductile iron pipe (DIP), although for high-pressure and large-force mains, cement mortar-lined and coated steel pipe may be used. Polyvinyl chloride (PVC) piping material may be used for smaller-diameter force mains subject to approval by the local authority. In selecting the type of materials to use for force main construction, the following factors should be considered (Bureau of Engineering, 2007):

- *External corrosion caused by aggressive soils, groundwater, and stray currents.*
- *Internal corrosion caused by sulphides and other chemical constituents.*
- *Internal erosion from abrasive solids.*
- *Ground movements, such as those caused by subsidence, landslides, earthquakes and differential settlement.*
- *External loading.*
- *Normal operating internal pressures and surge pressures.*
- *Construction methods suitable and most economical for the selected alignment.*

Rising main piping accessories are as follows:

- Sewage type air- and vacuum-release valves will be needed on the rising main and utilizing the ASAP procedure as described by Van Vuuren et al. (2004) will assist in the air-valve sizing and positioning for the system. The type of air valve that can be used is, for example, the Vent-O-Mat RGX or equivalent suitable for sewerage system application.
- Blow-off valves at low points

- Access maintenance holes and bypass valves
- Pig launchers and retrievers for internal cleaning
- Thrust blocks to provide a reaction force for the unbalanced thrust

The most commonly found problems in the rising main are related to internal and external corrosion. Internal corrosion is normally caused by H_2S accumulation inside the pipeline, especially at high points in the piping system. When possible, high points in the piping system shall be avoided. Where high points are unavoidable, sewage type air- and vacuum-release valves should be installed. External corrosion is normally found where the pipe is in contact with seawater, corrosive liquid or corrosive soils. Ground movements such as differential settlement, subsidence or earthquakes are considered secondary causes of piping system failures.

Most of the pipe materials, except plastic pipes, are susceptible to corrosion by acids forming in the sewer gas space when the pipe is not flowing full. The results of H_2S corrosion are similar for both metallic and cementitious pipes. Internal corrosion will result in reduction of the pipe-wall thickness and ultimately holes will form in the top of the pipe. Progressive corrosion will lead to weakening of the pipe wall which could result in pipe collapse. The pipe-wall thickness shall be calculated based on the internal pressure and the external load (trench load) plus corrosion allowance. In addition, the wall thickness shall be based on the predetermined bedding and backfill conditions.

5.3.11 Pump-station building and site

The pump-station facility includes the pumping-plant structure, buildings, electrical substation, access roads and other appurtenant equipment inside the pump-station property. The facility design should incorporate access road and security. The architectural design should blend with the surrounding area.

5.4 Siphon design

Within the sanitary sewer system there are numerous special structures serving particular needs. These special structures include inverted siphons crossing rivers, streams, depressed highways and other obstructions. Inverted siphons and airlines (sometimes called an 'air jumper') are constructed to convey sewage flows (liquid and gas) across obstructions where such crossings cannot be attained by a sewer placed on a continuous grade.

Inverted siphons and airlines are designed to meet criteria to ensure proper functioning during the design period of the system, to be fail-safe and to minimize maintenance and odours.

Paragraph 10.1 provides a detailed description of the analyses required for an inverted siphon.

5.5 Silt trap

Silt traps are designed to trap sand and grit. This is usually achieved by reducing the flow velocity and allowing enough time/distance for the particles to settle and remain in the trap.

Paragraph 10.2 provides additional information on the design criteria for silt traps.

5.6 Standard drawings and schedule of quantities

Included in **Appendix B** are standard drawings for a conventional gravity sewer system. These drawings are available from City of Tshwane Metropolitan Municipality (CTMM, 2007) and have also been included on the IMT.

Based on SANS 1200 LD:1982 (SABS, 1982a) a typical schedule of quantities is included as **Appendix C** for conventional gravity sewer systems.

5.7 House connection pipe

The need for a hands-on step-by-step design procedure evolved during the development of design procedures for local municipality structures involved with the decision making and design of waterborne sanitation systems. This section provides a step-by-step set of procedures to be followed to initiate and complete a design for house connection pipes in conventional gravity sewer systems.

5.7.1 Step-by-step design procedure (house connection pipe)

The house connection pipe connects the main sewer line with a house or building(s) on a single erf. **Figure 5.20** presents a flow diagram of the steps required in designing a house connection pipe.

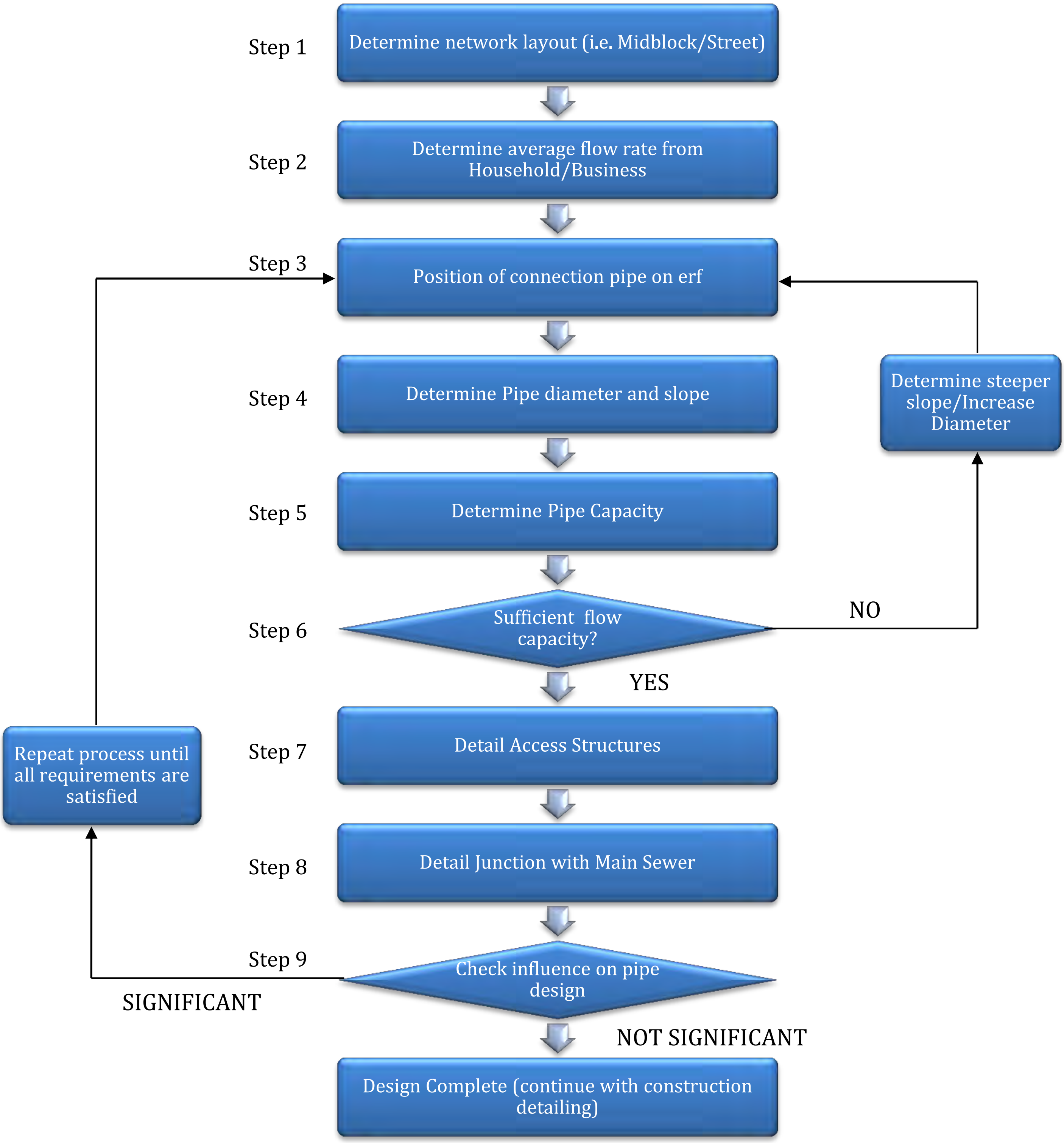

Figure 5.20: Flow diagram – House connection pipe design

<u>Step 1: Determine network layout (i.e. midblock/street):</u>

The first step is to determine the gravity sewer network layout of the system to be analysed. This process is discussed in more detail under **Paragraph 5.8**.

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The position of the distribution sewer line, i.e. whether it is located in a street reserve or midblock servitude, affects the layout and positioning of the house connection pipe. If the house is located closer to the street, but the main sewer line is in the midblock servitude, then the pipe length and therefore by implication the depth of excavation will be more than if the main distribution line was located in the street. Therefore, determining the sewer layout requires a holistic overview of the total cost implication with excavation depth and pipe length for all the house connections vs. the advantage of laying the distribution pipe in a position further away from the house/business.

Step 2: Determine average flow rate from household

Municipalities and local authorities normally provide guidelines or data on the expected average sewage flows from households. If no such data are available, average daily sewage flow-rate data provided by the SANS 10252-2:1993 (SABS, 1993) may be used.

The average daily sewage flow from dwelling houses or dwelling units with and without full in-house water reticulation is given in **Table 5.1.**

Step 3: Position of connection pipe on erf

It is not always possible to prescribe definite rules on the position of the connection pipe on the erf, because so many factors affect its positioning, such as:

- Connection point at building
- Connection point at main sewer
- The number of branch drains
- The access (such as rodding eyes, inspection chambers, etc.) positions
- Drain bends radii
- Site topography
- Position of house on erf

Figure 5.21 provides some recommendations for access to installations at a domestic dwelling.

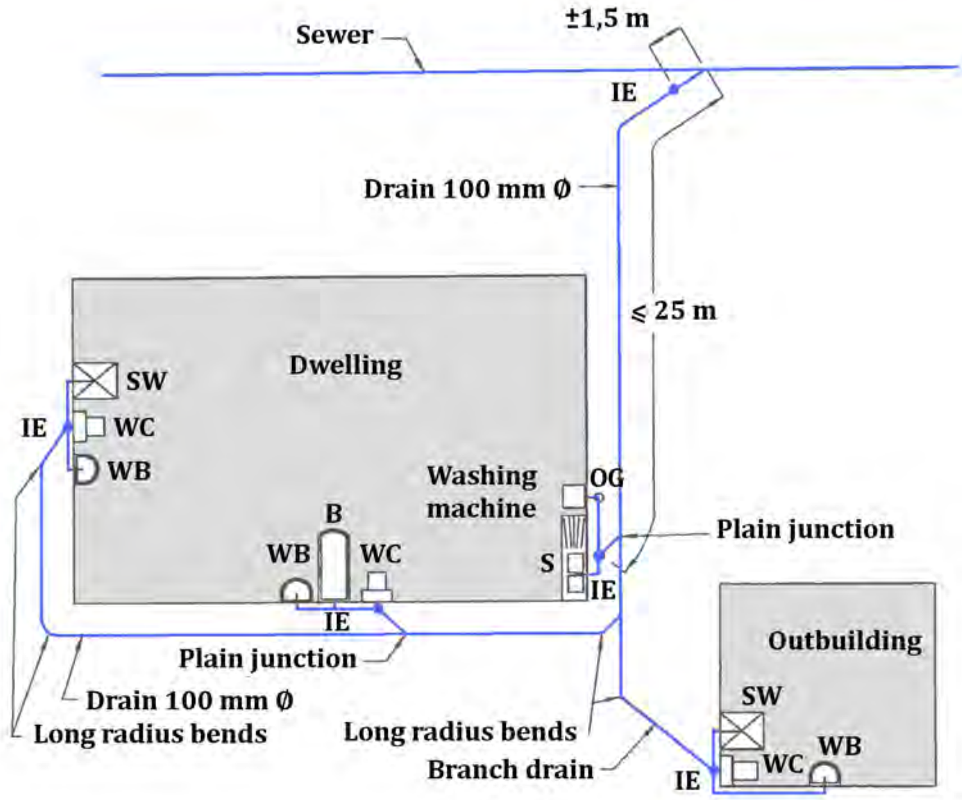

Figure 5.21: **Connection pipe typical layout (SABS, 1993)**

As shown in **Figure 5.21**, the sanitary fixtures collect at an inspection eye on the outside of the house. From each inspection eye, a collector drain (100 mm diameter) runs parallel around the house and collects all the discharges from the other sanitary fixtures. All bends are long radius bends with a radius of at least 600 mm. Branch drains connect to the main drain with an inspection eye. From the last connection fixture, the house connection pipe drains towards the main sewer line. Rodding/cleaning eyes shall be located every 25 m along the house connection pipe. The pipe connects into the main sewer line at a 45° angle. The connection can either be through a manhole, or, if a manhole is not located at that position, an inspection/rodding eye can be located approximately 1.5 m upstream of the connection point in the connection pipe.

The following criteria must be adhered to when designing the layout position of the connection pipe (SABS, 1993):

- *Branch or house drains should be joined to the main connection pipe at a 45° angle.*
- *At changes in direction, long radius bends (600 mm radius) should be used as far as possible. When a short radius 90° bend is required, inspection chambers/ manholes should be provided.*
- *Rodding eyes should be provided at the highest point on the drain and along the drains at a distance not more than 25 m apart.*

Step 4: Determine pipe diameter and slope

The connection pipe must have an inside diameter of at least 100 mm. The 100 mm diameter is an over-design for typical household sewage flows, but the standard size of WC pans necessitates a minimum diameter of 100 mm for drains at outlets, because of the provision that no branch pipe or drain may be narrower than the outlet of any sanitary fixture discharging into it (SABS, 1993).

The average slope of the pipe is calculated between the nearest access structure (i.e. rodding eye or manhole) to the house connection point and the connection point at the main line sewer pipe. The following formula calculates the average slope of the pipe:

$$\Delta S = \frac{H_1 - H_2}{L_{1-2}} \quad \text{...(5.22)}$$

where:

ΔS = average slope of pipe (m/m)

H_1 = elevation of pipe invert at house connection manhole/rodding eye (m)

H_2 = elevation of pipe invert at main sewer line connection point (m)

L_{1-2} = length of pipe between Point 1 and Point 2 (m)

Recommended slopes for 100 mm diameter house connection pipes are 1 in 60 (one vertical unit for every 60 horizontal units) (CSIR, 2003). View **Figure 3.4** for diagrammatic illustration of elements in design of house connections' slope and depth to link up with the main sewer (SABS, 1982a).

Steeper slopes, however, are acceptable provided that anchor blocks are provided at bends with slopes of 1:10 and steeper.

If, however, a large level terrain necessitates a flatter slope, then the pipe diameter should be increased.

Step 5: Determine pipe capacity

To determine the maximum capacity of the pipe, Manning's flow for open-channel gravity flow may be used as provided in **Table 5.4**.

A 100 mm diameter PVC pipe laid at the minimum slope of 1:60 and assuming the Manning's roughness coefficient of n = 0.013 s/m$^{1/3}$ results in a flow of 6.67 ℓ/s, i.e. maximum capacity.

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Step 6: Check sufficient flow capacity

In determining the flow capacity of the connection pipe, the following requirements should be adhered to:

- The minimum design velocity of flow needed to obtain self-cleansing of drains is 0.7 m/s and the optimum flow velocity for the prevention of sewer corrosion under average conditions is from 0.8 m/s to 1.4 m/s.

- The drain should be designed to have a maximum design depth of flow of 75% of the pipe diameter, to ensure adequate air movement.

Flow velocities of less than 0.6 m/s could result in sediment build-up and would require more maintenance over the lifetime of the pipe (SABS, 1993).

If the average sewage flow from the house exceeds 75% of the capacity of the pipe, then the design process from Step 3 onwards has to be repeated with steeper slopes or larger pipe diameters until the capacity is sufficient.

However, in certain circumstances minimum velocities of 0.6 m/s cannot always be obtained, especially where new connections are made to existing sewer lines of which the elevation cannot be altered to obtain the desired slopes.

Step 7: Details of access structures (i.e. rodding eyes, cleaning eyes, inspection chambers, manholes, etc.)

The positioning and detailing of access structures are important in that the position of these structures as well as certain inlet and outlet level requirements could in certain applications alter the average slope of the connection pipe. **Figure 5.22** shows a typical arrangement of access structures along the connection pipe.

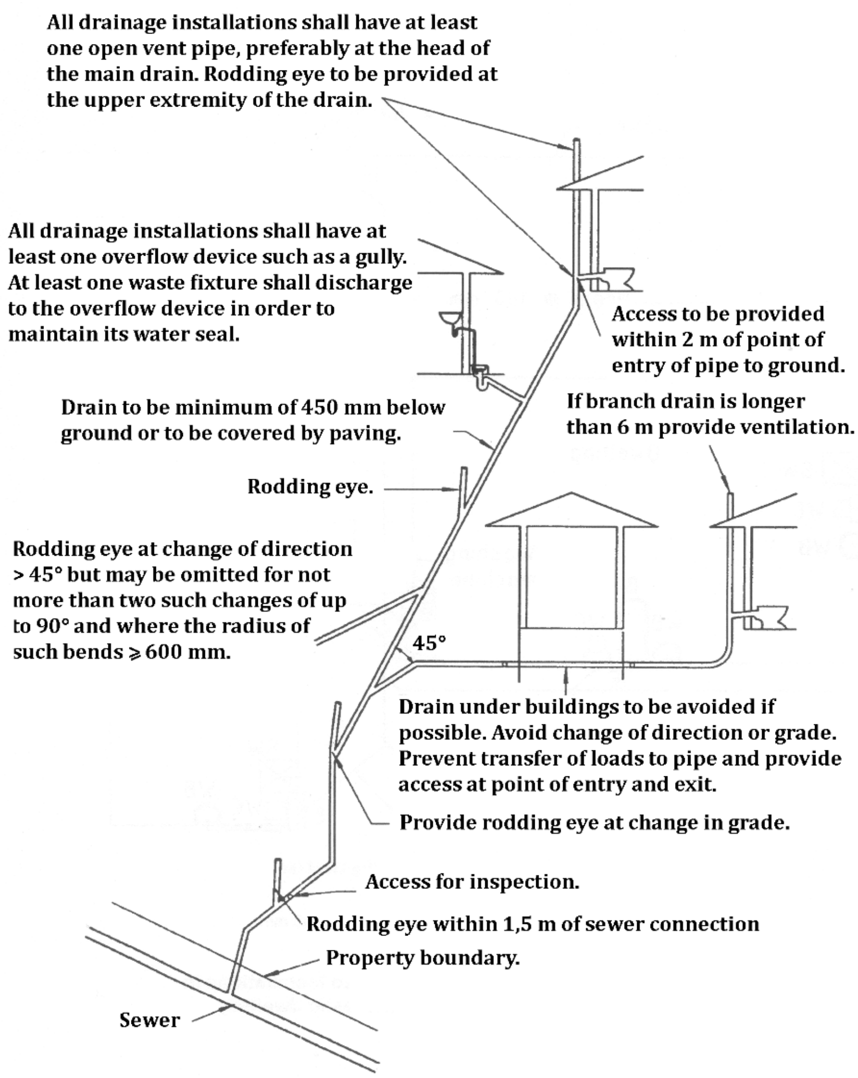

Figure 5.22: General access requirements for connection pipe (SABS, 1993)

Rodding/cleaning eyes

A rodding eye is a pipe fitting that is accessible from the surface level, and that has a removable, sealed cover, to enable the clearance of obstructions in one direction by rodding along the drain. Rodding eyes are essentially extensions of the pipe work system but they are used only for the insertion of cleaning equipment.

Rodding eyes should be detailed in accordance with the requirements of SANS 10252-2:1993 (SABS, 1993).

Inspection chambers

Inspection chambers on a drain or sewer are normally constructed to provide access for inspection, testing, maintenance and clearance of obstructions, as well as for the removal of debris, in all instances operating from the surface level. A typical inspection chamber is shown in **Figure 5.23**.

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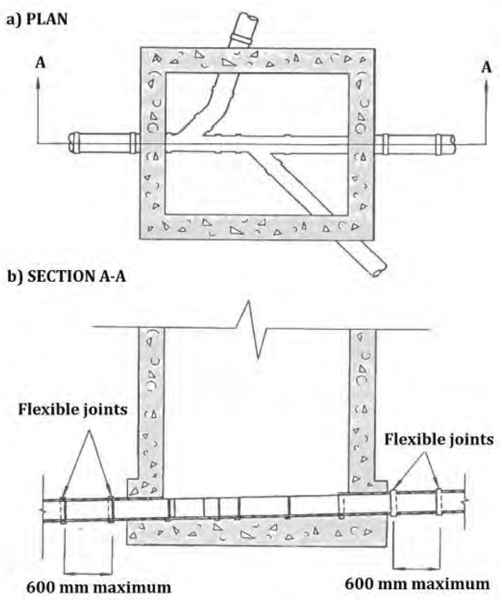

Figure 5.23: Typical inspection chamber detail (SABS, 1993)

- Manholes

Manholes that have removable covers permit the entry of a person for inspection, testing and maintenance, as well as the clearance of obstructions and removal of debris from drains. The dimensional requirements are indicated in **Table 5.8** with other relevant specifications and design criteria in **Paragraph 5.2.3**.

The invert levels indicated at a manhole location should be the levels projected at the theoretical centre, under the access opening of the manhole, by the invert grade lines of the pipes entering and leaving such manhole.

Step 8: Details of junction with main sewer line

The connecting sewer should be located deep enough to drain the full areas of the erf portion on which building construction is permitted. On street boundaries the connection should be located either at a distance of 1.5m or at a distance of 5 m or more from a common boundary with an adjacent erf, unless the local authority already has an accepted standard location (SABS, 1993).

A plain 45° junction should be used at the point where the connecting sewer joins the main sewer. Details of the connecting sewer should be in accordance with one of the types shown in Figures LD-7 and LD-8 of SANS 1200 LD:1982 (SABS, 1982a). Standard drawings are included in **Appendix B**.

To prevent the occurrence of backwater flow from the main sewer into the connection pipe, the designer needs to calculate the average hydraulic gradient of the main sewer (following section) during peak flow and check whether the energy gradient at the junction is not much higher than the connection pipe energy slope. If the latter is critical, then the junction elevation needs to be lowered to obtain a higher energy slope in the connection pipe (TAG, 1985).

Step 9: Check influence on pipe design

If the addition of access structures or the alteration of the junction elevation affects the hydraulic properties of the connection pipe, then the calculation process has to be repeated from Step 3 onwards until all requirements are met.

5.7.2 Worked Example #1: House connection pipe

Design a house connection pipe between a house and an existing main sewer line. The low-income house (6 persons) is fitted with a full in-house water reticulation system. **Figure 5.24** illustrates the position of the house on the erf relative to the main sewer line. The distance between the house and the main sewer is 25 m. Elevations are indicated on the layout plan.

Step 1: Determine network layout

The main sewer line is situated in the street.

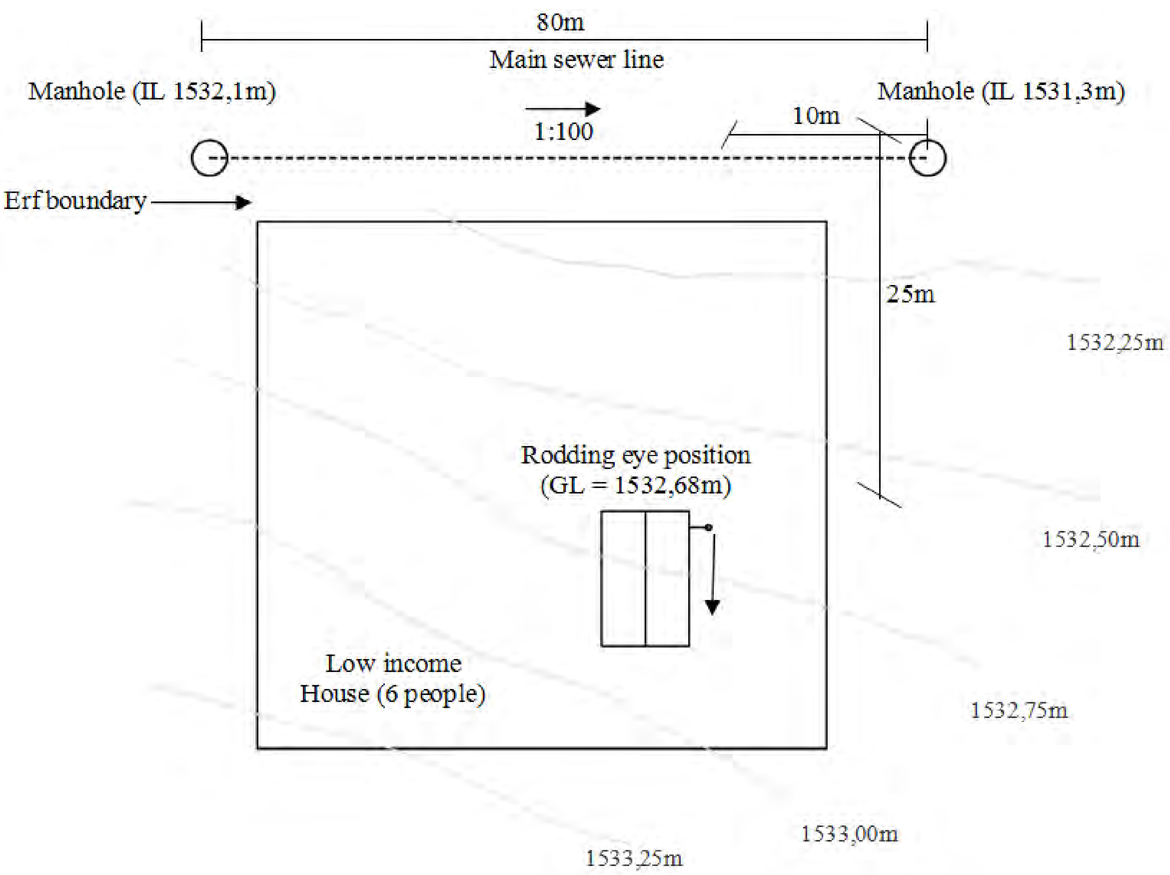

Figure 5.24: Example #1: Layout of house on erf with main sewer in front (not to scale)

Step 2: Determine average daily sewage flow rate

From **Table 5.1**, the average daily sewage flow rate per person is 70 ℓ/d. Therefore, the household flow is 70 x 6 = 420 ℓ/d.

Step 3: Determine position of pipe on erf

The connection pipe will be laid in a straight line from the rodding eye to the main sewer. The length of the pipe is 25 m. The pipe will connect to the main sewer line at a position 10 m from the downstream manhole.

Step 4: Determine pipe diameter and pipe slope

First, determine the invert level at the connection point in the main sewer:

$$\begin{aligned} H_1 &= H_2 + S \times L \\ &= 1\,531.3 + (1/100) \times 10 \\ &= 1\,531.4 \text{ m} \end{aligned}$$

Assuming a minimum pipe diameter at 100 mm and a minimum slope of 1:60, then the rodding eye invert can be calculated:

$$\begin{aligned} H_3 &= H_1 + S \times L \\ &= 1\,531.4 + (1/60) \times 25 \\ &= 1\,531.82 \text{ m} \end{aligned}$$

The ground level at the rodding eye is given as 1 532.68 m, therefore, the depth to rodding eye invert is 1 532.68 – 1 531.82 = 0.86 m.

Step 5: Calculate pipe capacity

First, calculate the maximum capacity of the pipe at full flow:

$$Q = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} \cdot A$$

where:

n = 0.013 s/m$^{1/3}$ (for PVC drain pipe)
R = D/4 = 0.1/4 = 0.025 m
S = 1/60 = 0.0167 m/m

$$A = \pi \cdot \frac{0.1^2}{4} = 0.00785 \text{ m}^2$$

$$Q_{max} = \left(\frac{1}{0{,}013}\right) \cdot (0{,}025)^{\frac{2}{3}} (0{,}0167)^{\frac{1}{2}} (0{,}00785)$$

= 0.00667 m^3/s
= 6.67 ℓ/s

Step 6: Check if capacity is sufficient

The average daily flow rate is:

$$\begin{aligned} Q_{actual} &= \frac{420}{24 \times 60 \times 60} \\ &= 0.00486 \text{ ℓ/s} \end{aligned}$$

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Add 15% for infiltration (refer **Table 5.1**)

$= \quad 0.00590\ \ell/s < 6.67\ \ell/s$

Therefore, pipe capacity is sufficient to cater for the required flow.

<u>Step 7: Details of access structures</u>

The invert level of the rodding eye at the house has to be calculated. First, determine the invert level at the connection point in the main sewer:

Using Equation (5.22), then,

$$H_1 = H_2 + S \times L$$
$$= 1\,531.3 + (1/100) \times 10$$
$$= 1\,531.4\text{m}$$

Using a minimum pipe slope of 1:60, the rodding eye invert is:

$$H_3 = H_1 + S \times L$$
$$= 1\,531.4 + (1/60) \times 25$$
$$= 1\,531.82\text{ m}$$

The ground level at the rodding eye is given as 1 532.68 m, therefore, the depth to rodding eye invert is 1 532.68 – 1 531.82 = 0.86 m (which is satisfactory). A typical detail of a rodding-eye structure is shown in **Figure 5.12**.

<u>Step 8: Details of junction to main sewer</u>

Each erf should be provided with a 100 mm (minimum) diameter connecting sewer pipe, terminating with a suitable watertight stopper on the boundary of the erf or the boundary of the sewer servitude, whichever is applicable. The connecting sewer should be located deep enough to drain the full area on the erf portion on which the building construction is permitted.

The pipe will connect at a 45° angle to the main sewer line using a normal branch connection tee; see an example in **Figure 5.13**. A manhole will not always be required in the main sewer line as an inspection eye, constructed 1.5 m upstream of the connection point on the connection pipe, will be sufficient for cleaning and maintenance purposes. **Figure 5.25** reflects the final design layout.

5.7.3 Design hints and tips (house connection pipe design)

A few hints and tips are given herewith for the designer.

When designing a house connection pipe that connects to an existing system:

- Start with the invert elevation of the junction first when determining the pipe position. The lower the junction along the sewer line, the more likely the house connection pipe slope requirements will be met.

- Try to connect to the nearest existing manhole in the main sewer pipe. Cleaning a pipe from a manhole is more efficient than through a rodding eye. Also, this will reduce the cost of an additional rodding eye at the junction.

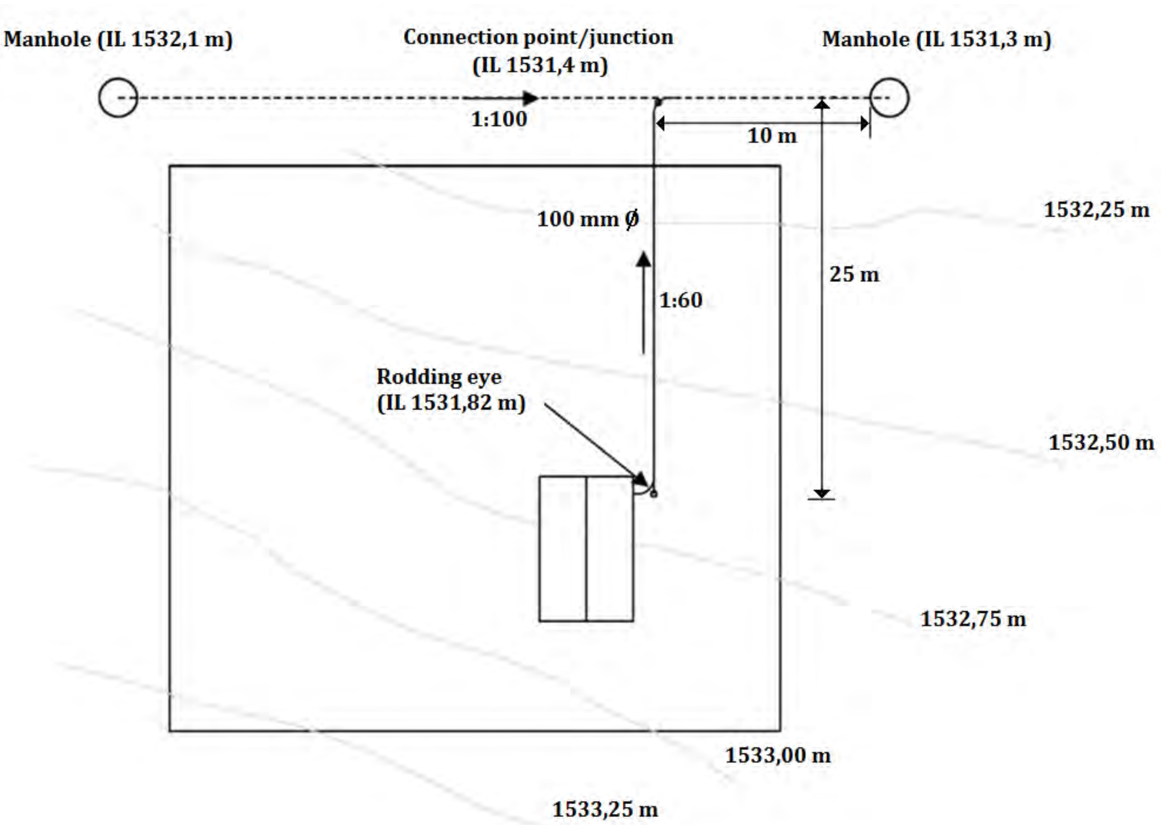

Figure 5.25: Example #1: Final layout: House connection example

When designing a house connection pipe that connects to a system yet to be installed:

- Start with the invert elevation of the house, and maintain a minimum slope to the proposed pipeline position. The shortest route will be most economical.

5.8 Main sewer line

This section provides a flow diagram of the procedures to be followed to initiate and complete a design for the main sewer line in conventional gravity sewer systems.

5.8.1 Step-by-step design procedure (conventional main sewer design)

A conventional gravity sewer system is a method of conveying wastewater from all users to a selected outlet utilizing the non-pressured gravity flow from the upstream end of the system to the downstream end. **Figure 5.26** presents a flow diagram of the steps required in designing a main sewer line.

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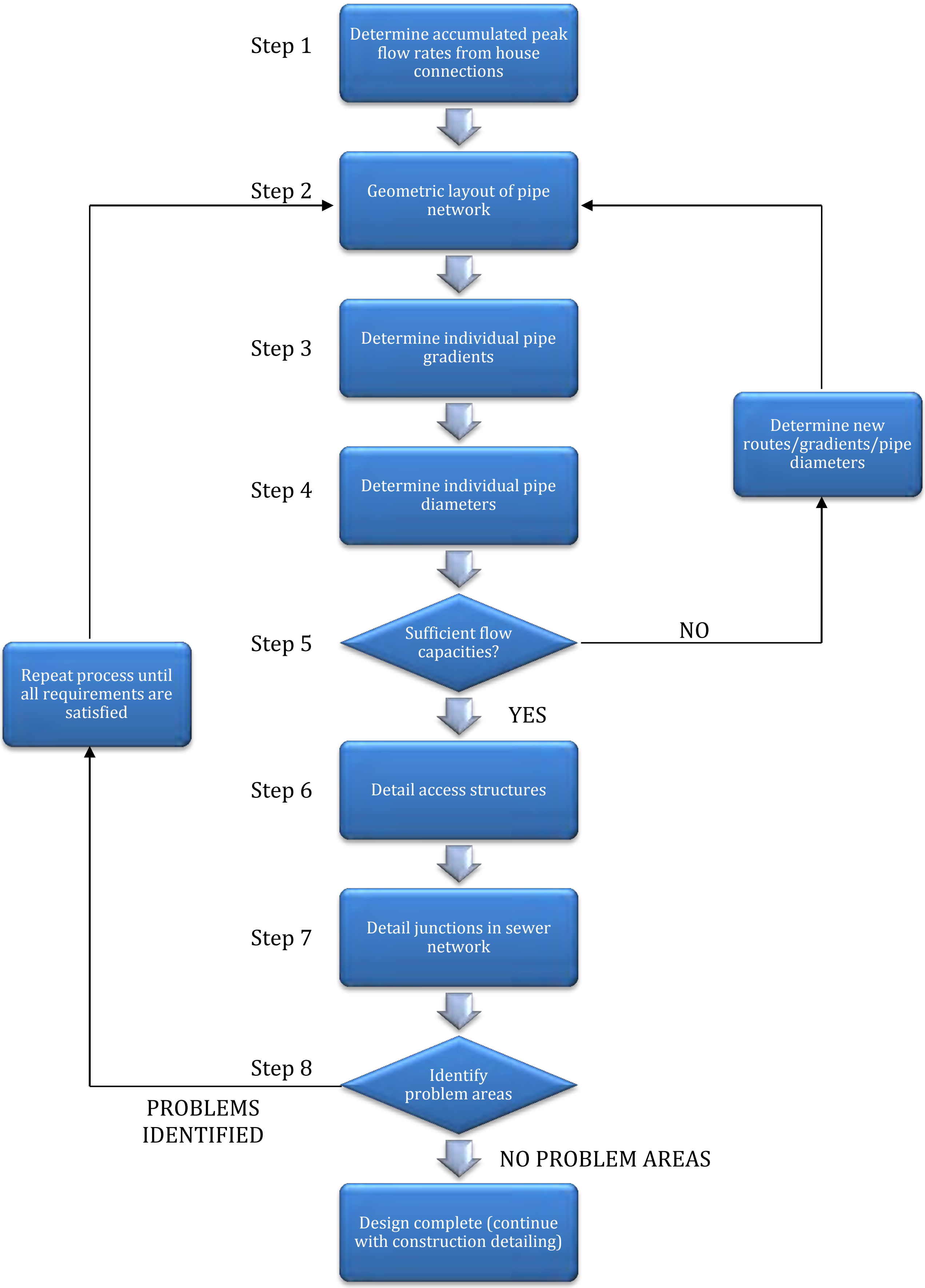

Figure 5.26: Flow diagram – conventional gravity main sewer design

Step 1: Determine accumulated peak-flow rates from house connections

The peak-flow rate from house connections is calculated as follows:

Determine the average daily sewage flow (URE) from **Table 5.1** for dwelling units with full in-house water reticulation.

Calculate the peak-flow factor, PFF, using Equation (5.2) (Harmon, 1918) and determine the anticipated infiltration (INF) rate, groundwater and leakage from **Tables 5.2** and **5.3**.

Determine the accumulated peak-flow rate using Equation (5.23):

$$Q_{design} = (PFF)(URE) + INF + IED \qquad \text{...(5.23)}$$

where:

Q_{design} = design flow (ℓ/d)

PFF = peak-flow factor (between 1.3 and 2.5) depending on the land type and use

URE = average contribution from urban residential erf (ℓ/d·EDU or ℓ/d·SFD or specific by-laws unit)

IED = industrial effluent discharge (ℓ/d·erven)

INF = infiltration of groundwater and leakage from plumbing devices (ℓ/d) - see **Paragraph 5.2.2.3**

Convert design-flow rate to a flow rate in ℓ/s:

$$Q = \frac{Q_{design}}{(24)(60)(60)} \qquad \text{...(5.24)}$$

where:

Q = design flow (ℓ/s)

Q_{design} = design flow (ℓ/d)

For a gravity main line sewer, the accumulated flow along the sewer line is calculated by summing the individual house connection peak-flow rates.

Step 2: Geometrical layout of pipe network

The geometrical layout of the pipe network can be made mainly from contour maps or layout plans of the area to be served – if these show elevations, existing roads, buildings, property boundaries and other pertinent information. The layout commences by selecting an outlet and service district and sub-district boundaries. The district and sub-district boundaries are usually made to conform to natural drainage basins. These are normally provided by the local authority or municipality. Within these boundaries, the branch and main sewer routes are selected. Selection of sewer routes must consider the following (TAG, 1985):

- Vertical and horizontal alignments
  The location and outlet elevation of the house connection pipes together with the local topography will establish the routes and necessary depths of the sewers in most cases.

- Rights-of-ways and servitudes
  Use of existing rights-of-ways and servitudes should be assessed but if excavation costs can be reduced significantly by some other route special servitudes may be necessary.

- Lift stations (where applicable)
  While it is desirable to serve every connection by gravity, the local terrain or cost of excavation may require that lift stations be used.

- Future development
  The additional flow volumes that will be generated by future developments have to be incorporated in the design of the sewer main lines, to prevent major capital expenses of removing and re-installing a larger pipe diameter in the future.

- Site restoration
  The cost of reinstating pavements, curbs and gutters and other structures which may be torn up during construction will be an important consideration in locating routes. Curvilinear alignment will allow some structures to be avoided but this must be carefully planned so that joint deflections do not exceed those permitted by the pipe manufacturer.

- Resident and traffic disruption
  If homes are on either side of a roadway, consideration should be given to laying sewers on both sides to avoid expensive road crossings and tedious traffic disruptions.

- Determine connectivity of erf

Step 3: Determine individual pipe gradients:

The individual pipe section gradients are calculated between manholes or junctions and/or vertical inflection points. The calculation procedure of Equation (5.22) can be used to determine the average slope of each pipe section. The minimum required sewer gradients for various pipe diameters are shown in **Table 5.6**.

Step 4: Determine pipe diameter

The minimum recommended pipe diameter is 150 mm. The minimum gradients shown in **Table 5.6** provide guidance in the selection of pipe diameters. Determine the minimum pipe diameters for each pipe section from the above table.

Step 5: Sufficient flow capacities?

The next step would be to compare the accumulated flow from households along the sewer line with the maximum capacity of each pipe section. To determine the maximum capacity of the pipe, Manning's flow for open-channel gravity flow may be used as provided in **Table 5.4**. A 150 mm diameter PVC pipe laid at the minimum slope of 1:90 for the first number (< 10) of houses drained and assuming the Manning's roughness coefficient of n = 0.013 s/m$^{1/3}$ results in a flow of 16.05 ℓ/s. For sections where the number of houses linking into the system is between 10 and 80 the maximum capacity for the pipe laid at 1:120 slope would be 13.90 ℓ/s.

In determining whether the flow capacity of a sewer pipe section is sufficient, the following requirements should be adhered to (SABS, 1993):

- The minimum design velocity of flow needed to obtain self-cleansing of the main sewer pipes is 0.7 m/s
- Sewers should be designed not to exceed 80% of full-flow capacity at the peak design flow. In special circumstances this rule may be relaxed when the slopes are limited due to very flat topographies.

If the peak sewage flow rate exceeds the full capacity of the pipe, or when the minimum flow velocity is not achieved, then the design process from Step 2 onwards has to be repeated with steeper slopes or larger pipe diameters until the capacity and velocity are sufficient.

By using computer applications or spread sheets the design effort for the above process can be made less burdensome. Refer to the worked example in **Paragraph 5.8.2**.

Step 6: Details of access structures

The positioning and detailing of access structures (manholes) in the main sewer line are important in that the position of these structures as well as certain inlet and outlet level requirements could in particular applications alter the average slope of the sewer pipe.

Manholes should be placed at all sewer junctions and, except in the case of curved alignment and at the top of shallow drops, at all changes of grade and/or direction.

The maximum distance between manholes on either straight or curved alignment should be (CSIR, 2003):

- 150 m where the local authority concerned has power rodding machines and other equipment capable of cleaning the longer lengths between manholes
- 100 m where the local authority concerned has only hand-operated rodding equipment

The dimensional requirements are indicated in **Table 5.8** with other relevant specification and design criteria in **Paragraph 5.2.3**.

Step 7: Details of junctions in sewer network

The connecting sewer should be located deep enough to drain the full areas of the erf portion on which building construction is permitted. On street boundaries the connection should be located either at a distance of 1.15 m or at a distance of 5 m or more from a common boundary with an adjacent erf, unless the local authority already has an accepted standard location (SABS, 1993).

Examples of the typical connections are shown in **Figure 5.13a** and **b** as well as on the standard drawings included in **Appendix B.**

The recommended minimum values of cover to the outside of the pipe barrel for connecting sewers are:

- In servitudes: 600 mm
- In road reserves: 1 000 mm

A plain 45° junction should be used at the point where the connecting sewer joins the main sewer. Details of the connecting sewer should be in accordance with one of the types shown in Figures LD-7 and LD-8 of SANS 1200 LD:1982 (SABS, 1982a) .

To prevent the occurrence of backwater flow from the main sewer into the connection pipe, the designer needs to calculate the average hydraulic gradient of the main sewer (following section) during peak flow and check whether the energy gradient at the junction is not much higher than the connection pipe energy slope. If the latter is critical, then the junction elevation needs to be lowered to obtain a higher energy slope in the connection pipe (TAG, 1985).

Step 8: Check for problem areas

During and after completing the design process, the designer must ensure that the system will work. The designer is responsible to identify potential problem areas to ensure the successful operation of the system.

Typical potential problem areas are:

- Location of house connection points along the sewer line. Special attention should be given to manholes located above the lowest point in each erf. The connecting sewer should be located deep enough to drain the full area on the erf portion on which the building construction is permitted. Designers sometimes feel compelled to save costs by aligning the house connection pipe with the nearest manhole in an existing sewer system, which in some cases, could be higher than the erf portion on which building construction is permitted, instead of installing a new connection adjacent to the lowest point along the erf boundary, therefore compromising on pipe drainage slope. Cost considerations must never override the ability of the system to operate correctly.

- Distances between manholes. The maximum distances between manholes referred to in Step 6, should be considered in conjunction with house connection and junction location requirements. Designers sometimes limit the placing of manholes to the maximum distances; however, many other considerations should also be taken into account as referred to in Step 6, which may necessitate a higher frequency of manholes in the sewer pipeline.

- Low points along the longitudinal slope of the sewer line. A sewer pipeline must always have a positive slope in the direction of flow, i.e. a fall in the flow direction. Low-lying points must always be avoided in a gravity sewer system.

- Multiple pipe connections at sewer junctions. Not more than three pipes may enter a manhole structure although exceptions can be made. Care should be taken with the selection and design of the downstream pipe in terms of backwater effects. If the downstream pipe's capacity is close to the combined incoming peak flows from the other pipes, the designer should be aware of potential backwater effects at the manhole, especially with the occurrence of blockages or downstream backflows. The designer should consider increasing the outgoing pipe diameter and/or the pipe slope to increase capacity and prevent backwater effects.

- Crossing of services. This entails an integrated design process with other services, i.e. water pipelines, stormwater pipelines, electrical cables, communication cables, road crossings, etc. The designer must ensure that sufficient/required clearances shall be maintained while fulfilling the pipe's required operating drainage capacity. To implement a system with multiple services requires careful consideration of all services. In practice, construction of these services becomes a problem if the designer did not take the crossing of other services into account, increasing the likelihood of an incorrectly installed sewer pipe.

5.8.2 Worked Example #2: Conventional gravity sewer line design

A residential erven block requires a new gravity sewer, connecting to an existing main outfall sewer line. The population in this sub-network system is 60 persons. Assume the total population to be served for the whole sewer network in future to be 1 500 persons. Design the midblock sewer system (i.e. diameter, slope, capacity of each pipe, etc.) for the network indicated which will discharge into the main outfall sewer indicated. Allowance should be made for the inclusion of future expansions of the system, incorporating the flow from the remaining population into the main outfall sewer line. An illustration of the proposed sewer network layout is shown in **Figure 5.27**.

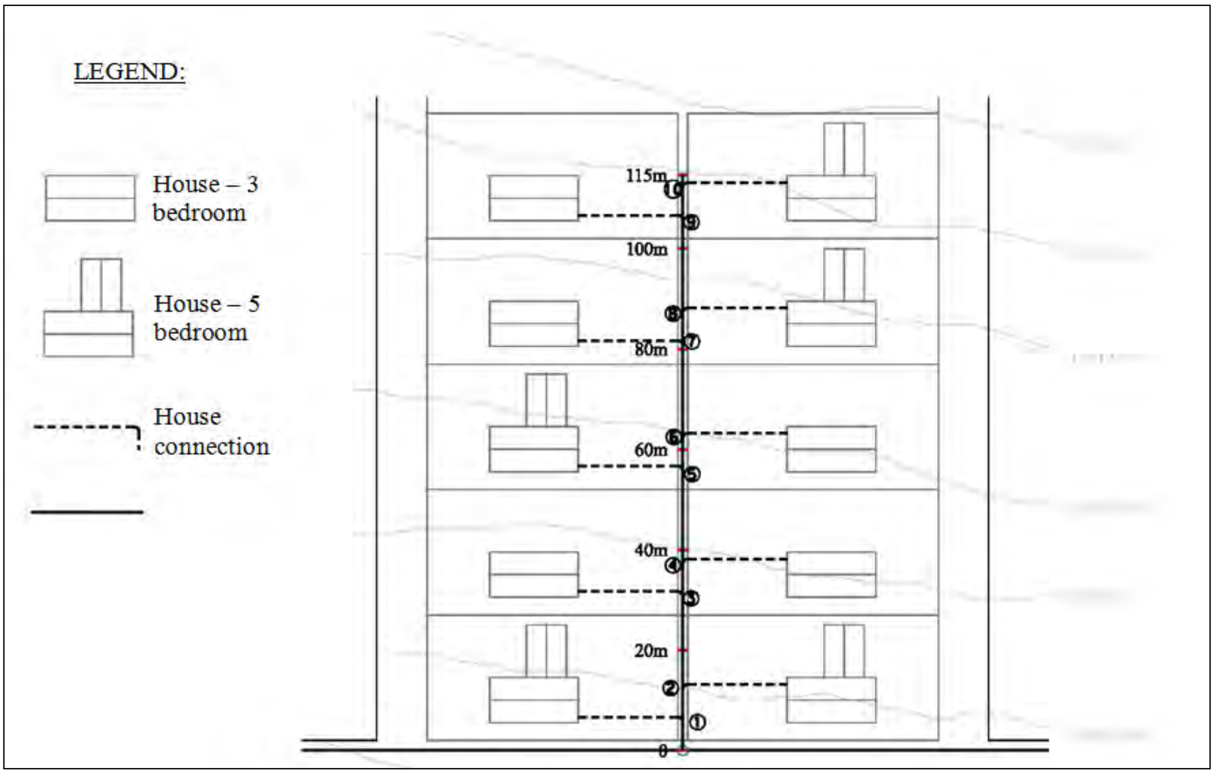

Figure 5.27: Worked Example #2 – Layout of midblock sewer line

Step 1: Determine accumulated flow rate

Two types of houses are shown in this example. The estimated design flows are obtained from **Table 5.1**.

- A 3-bedroom house (900 ℓ/d each) at Junctions 3, 4, 6, 7 and 9.
- A 5-bedroom house (1 400 ℓ/d each) at Junctions 1, 2, 5, 8 and 10

The average daily flow rate per junction using Equation (5.24) is calculated as:

$$Q_{3\text{-bed}} = \frac{900}{(24)(60)(60)} = 0{,}0104 \text{ l/s}$$

$$Q_{5\text{-bed}} = \frac{1400}{(24)(60)(60)} = 0{,}0162 \text{ l/s}$$

The current population figure in this sub-network is 60 persons. Utilizing the Harmon formulae (Equation (5.2)) results in a peak-flow factor (PFF) of 4.3.

Calculate the peak-flow rate per junction by multiplying the average daily flow rate per junction with the PFF:

$Q_{\text{peak-3-bed}}$ = 0.0104 x 4.3 = 0.0448 ℓ/s

$Q_{\text{peak-5-bed}}$ = 0.0162 x 4.3 = 0.0697 ℓ/s

The infiltration rate can be calculated with the data supplied in **Tables 5.2** and **5.3**. The typical groundwater infiltration rates are assumed to be 0.04 ℓ per metre diameter per metre pipe and the water leakage is assumed to be 0.15 ℓ/min per urban erf.

The pipe linking Junctions 10 and 9 will thus have the inflow of the 5-bedroom house at Junction 10 ($Q_{peak\text{-}5\text{-}bed}$ = 0.0697 ℓ/s) plus the anticipated infiltration (INF). The INF is the 0.15 ℓ/min leakage plus the groundwater infiltration which is 0.04 ℓ/min per metre diameter per metre pipe. The assumed diameter of this pipe is 150 mm and the length is 6.49 m resulting in a groundwater infiltration rate for this specific pipe of 0.03894 ℓ/min.

The infiltration for pipe section linking Junction 10 to Junction 9 is:
$INF_{10\text{-}9}$ = 0.15 + 0.03894 = 0.1889 ℓ/min = 0.00315 ℓ/s

The design-flow rate of the pipe section from Junction 10 to Junction 9 is:
$Q_{design\text{-}10\text{-}9}$ = 0.0697 + 0.00315 = 0.0728 ℓ/s

Details of each of the design-flow rates for this sewer network, calculated as described above, are listed in **Table 5.13**.

The total accumulated design-flow rate at Junction 0 is the inflow of 10 houses plus household leakage plus the groundwater infiltration:
$Q_{total\ design}$ = 5 x 0.0448 + 5 x 0.0697 + 0.025 + 0.0112 = 0.609 ℓ/s

Step 2: Geometrical layout of network

Refer to **Figure 5.27** for a layout of the network. A long section along the midblock sewer is drawn to establish the topographic elevation and slope along the midblock sewer pipeline (**Figure 5.28**).

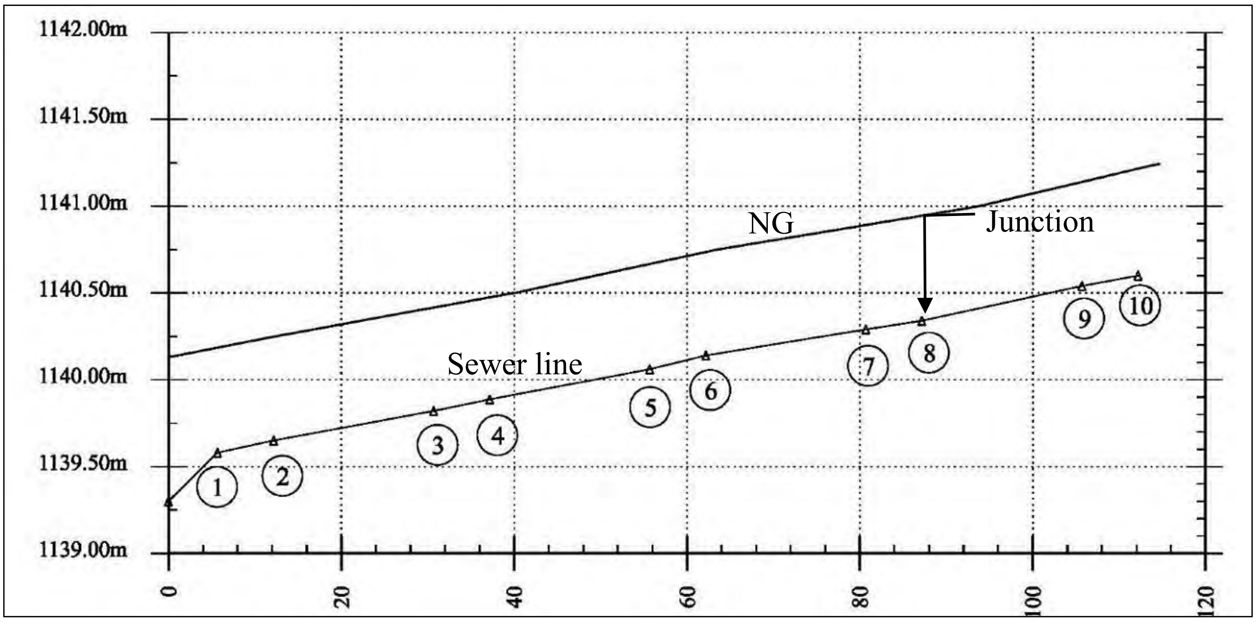

Figure 5.28: Worked Example #2– Long section along midblock sewer line

Steps 3 – 5

For a start, the pipe is lowered to 600 mm below the ground topography.

Calculating the slopes between pipe sections and determining the pipe capacity involves an iterative process which would best be solved through a spread sheet application (or specialized software) as shown in **Table 5.13.** A description for each column follows:

a) **Column 1** – Junction No. : The junction number to which a house is connected.
b) **Column 2** – Junction elevation: The invert elevation of the intersection between the midblock sewer and junction.
c) **Column 3** – Distance: The distance along the midblock sewer line to the junction position, starting at 0 m at the manhole.
d) **Column 4** – Elevation difference over section: The difference in elevation between a specific junction and the junction downstream of it. The values in this column will have to be altered to suit the slope, capacity and velocity requirements stated in Columns 12, 13 and 14.
e) **Column 5** – Length of section: The length of the pipe section between a specific junction and the junction downstream of it.
f) **Column 6** – Average slope of section: The average slope using Equation (5.22) is calculated of the pipe section between a specific junction and the junction downstream of it. The elevation (Column 2) and pipe length (Column 5) are the values used in the equation.
g) **Column 7** – Peak discharge per connection: The design-flow rate as calculated in Step 1 per junction.
h) **Column 8** – Accumulated design flow: Starting from the highest point, calculating the accumulated design flow along the midblock sewer line.
i) **Column 9** – Pipe diameter: The internal pipe diameter of the pipe section between two junctions. The diameter will be altered to suit the requirements of capacity and velocity in Columns 12, 13 and 14.
j) **Column 10** – Full-flow capacity: The capacity is calculated using formulae in **Table 5.4** for each pipe section based on the slope and diameter defined in Columns 6 and 9.
k) **Column 11** – Full-flow velocity: The velocity of the flow in the pipe under full-flow conditions, i.e. sediment blockage or flow at full capacity.
l) **Column 12** – Slope sufficient? This column checks whether the actual slope meets the minimum requirements as set out in **Table 5.6**. If not, then the downstream junction elevation (Column 2) needs to be adjusted.
m) **Column 13** – Capacity sufficient? This column checks whether the pipe capacity is sufficient for the design flow (i.e., comparing values in Column 8 with those in Column 10).
n) **Column 14** – Full-flow velocity? The velocity under full-flow conditions is compared with the minimum flow velocities discussed in **Paragraph 5.2.2.4.**

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Table 5.13: Worked example: Gravity midblock conventional design table

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Junction nr	Junction elevation (m)	Distance (m)	Elevation difference over section (m)	Length of section (m)	Average slope of section (1 in X)	Peak discharge per connection (l/s)	Accumulated design flow (l/s)	Pipe diameter (mm)	Full flow capacity (l/s)	Full flow velocity (m/s)	Slope sufficient?	Capacity sufficient?	Full flow velocity sufficient?
0	1139.30	0	0.00				0.609						
1	1139.58	5.68	0.28	5.68	20.29	0.0727	0.609	150	33.81	1.91	☑	☑	☑
2	1139.65	12.17	0.07	6.49	92.71	0.0728	0.536	150	15.82	0.90	☑	☑	☑
3	1139.81	30.68	0.16	18.51	115.69	0.0491	0.463	150	14.16	0.80	☑	☑	☑
4	1139.87	37.17	0.06	6.49	108.17	0.0479	0.414	150	14.64	0.83	☑	☑	☑
5	1140.06	55.68	0.19	18.51	97.42	0.0740	0.366	150	15.43	0.87	☑	☑	☑
6	1140.12	62.17	0.06	6.49	108.17	0.0479	0.292	150	14.64	0.83	☑	☑	☑
7	1140.28	80.68	0.16	18.51	115.69	0.0491	0.244	150	14.16	0.80	☑	☑	☑
8	1140.34	87.17	0.06	6.49	108.17	0.0728	0.195	150	14.64	0.83	☑	☑	☑
9	1140.54	105.68	0.20	18.51	92.55	0.0491	0.122	150	15.83	0.90	☑	☑	☑
10	1140.60	112.17	0.06	6.49	108.17	0.0728	0.073	150	14.64	0.83	☑	☑	☑

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Comments on Worked Example #2:

In the above example, all the pipe sections are steeper than the minimum required slope of 1:90 as described in **Table 5.6**. The flow capacities of the pipes are sufficient to carry the required design flows. The flow velocities under full-flow conditions are sufficient. The invert depths below ground provide sufficient cover above the pipes.

5.8.3 Design hints and tips (conventional sewer design)

A few hints and tips are given herewith for the designer.

When designing a sewer pipe in a new network:

- First design the house connections (refer to previous section). This will give the minimum invert levels the pipe has to connect to.
- In the main sewer line, commence the analyses starting at the bottom junction first, and then working upstream connecting to all the house-connection junctions
- Try to position manholes to the nearest house-connection junction, thereby reducing the cost of fittings for rodding eyes at normal junctions
- To minimize calculation effort, develop a spreadsheet application similar to the worked example. To obtain the minimum required slopes, changing the elevations between junctions will change the average slopes of pipe sections.
- Try to keep gradients similar for easy installation

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6. DESIGN GUIDELINES FOR VACUUM SEWERS

In some parts of the world, there are established standards for vacuum sewers. According to WEF (2008) Australia has a 2004 standard called the *Vacuum Sewerage Code of Australia* (WSA 06-2004), and Europe has the *1997 European Norm Vacuum Sewerage Systems Outside Buildings* (EN-1091). In the United States there is no national vacuum sewer system standard and in South Africa due to the limited use of this technology there is also no such a document. This guide attempts to address this shortcoming.

The major components of the vacuum-collection system are defined below and are depicted in **Figure 6.1**.

- Main line – larger-diameter trunk lines that enter the vacuum station
- Branch line – smaller-diameter lines that connect to the vacuum main
- Service lateral – 75 mm vacuum line that connects the valve pit to the vacuum main
- Valve pit – point of connection for customer
- Vacuum station – heart of the system, where the vacuum is produced and wastewater is collected

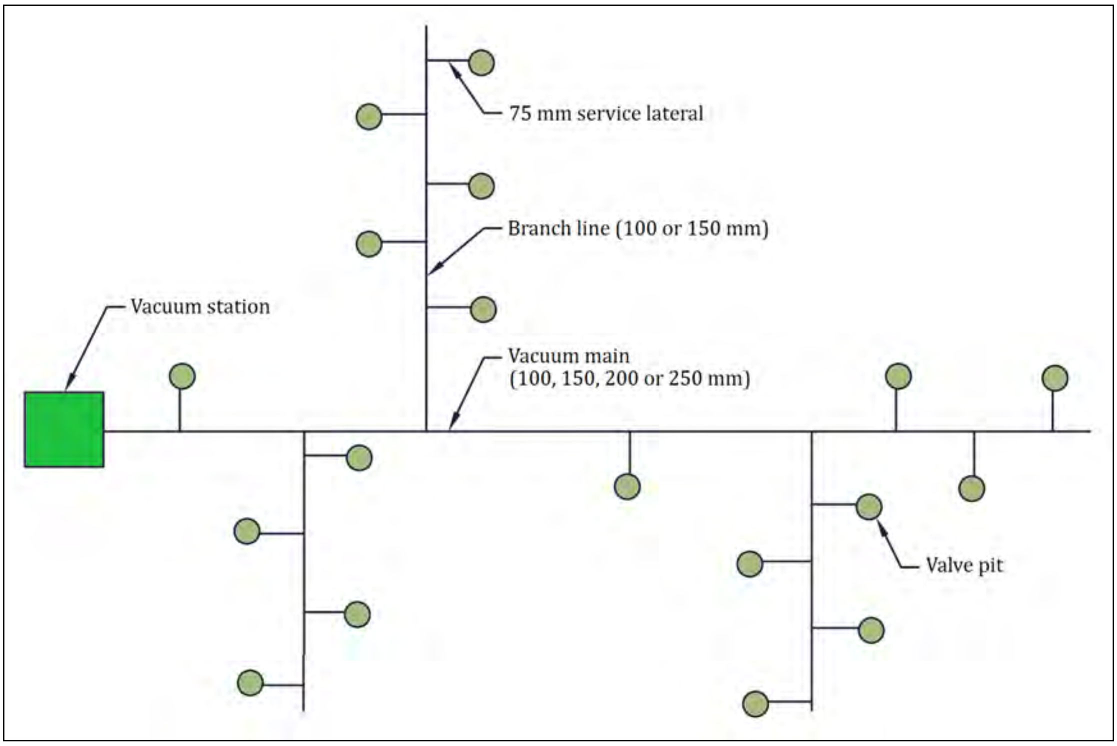

Figure 6.1: Major components of vacuum system

The house/valve pit relationship is shown in **Figure 6.2**.

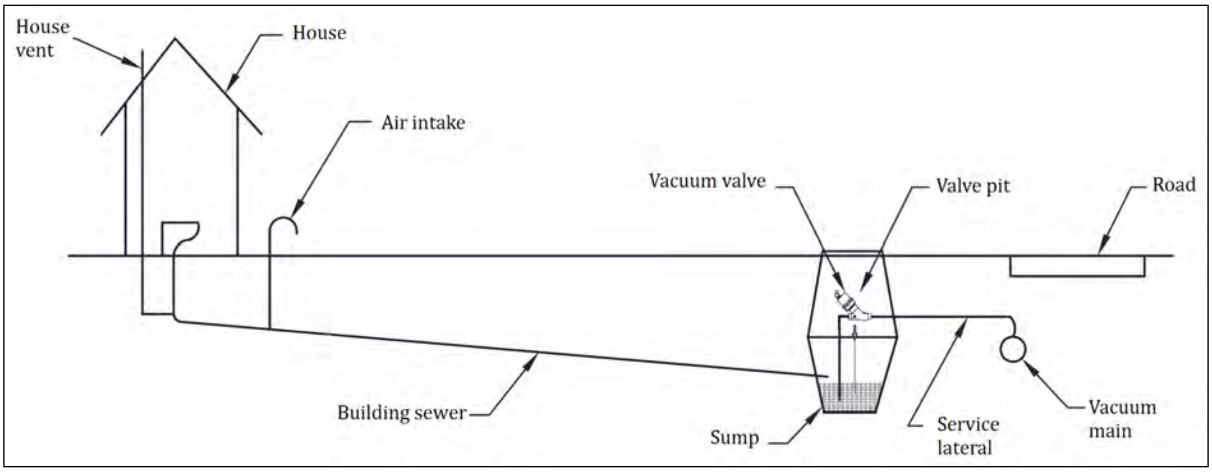

Figure 6.2: House/valve pit relationship (WEF, 2008)

6.1 Vacuum sewer system planning

There are three major items for the designer to consider when laying out a vacuum collection system.

- *Multiple service zones*: By locating the vacuum station centrally, it is possible for multiple vacuum mains to enter the station, which effectively divides the service area into zones. This results in operational flexibility and service reliability. With multiple service zones, the operator can respond to system problems, such as low station vacuum, by analysing the collection system on a zone-by-zone basis, and identifying the problem zone. The problem zone can then be isolated from the rest of the system, so that normal service is possible in the unaffected zones, while the problem is identified and solved.

- *Minimize pipe sizes:* By dividing the service area into zones, the total peak flow to the station is also dispersed among the various zones, making it possible to minimize the pipe sizes.

- *Minimize static loss:* Static loss is generally limited to 4 m. Factors that result in static loss are increased line length, elevation differences, utility conflicts, and the relationship of the valve pit location to the vacuum main.

6.2 Design criteria

6.2.1 General requirements

There are some general requirements for the design of vacuum sewer collection systems as presented in *Alternative Sewer Systems* (WEF, 2008):

- It is recommended that the entire vacuum sewer system, including the individual valve pits, shall be owned, operated, and maintained by a single operating entity

- The vacuum piping network should be designed with the intent to provide an open bore of the entire pipeline. Designs where sections of the pipeline are purposely sealed are not allowed.

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- The vacuum sewer system must be designed to remain functioning even during the loss of vacuum

- The air-to-liquid ratio in a vacuum sewer system shall be a minimum of 2:1

- For routine and emergency operation and maintenance of a vacuum sewer system, the local authority responsible for the sewer system shall have access to an adequate supply of spare valves, pumps and parts

- The maximum static loss in the vacuum sewer system shall be 4 m. These are the losses calculated from the furthest boundaries of the vacuum sewer system to the vacuum pump station:

    - Vacuum loss from each lift is calculated by subtracting the pipe internal diameter from the lift height (i.e., for a 0.3 m lift in a 100 mm pipe, vacuum loss is 0.3 m – 0.1 m = 0.2 m)

    - When lifts are required in specific valve service lines or when concrete buffer tank valve pits have suction lifts in excess of 1.7 m, the static losses shall be added to the losses for that sewer main and shall not exceed 4 m. The suction lift from the bottom of the holding sump to the valve centreline should never exceed 2.4 m

- The maximum friction loss in the vacuum sewer system shall be 1.5 m

    - Friction losses for vacuum sewers installed at slopes of between 0.2% and 2% are cumulative for each 'flow path' from the last valve on a line to the vacuum station. Friction losses for sewers installed in excess of 2% may be ignored. Friction losses are to be calculated using Equation (6.1):

$$h_f = \frac{10{,}69LQ^{1{,}852}}{C^{1{,}852}D^{4{,}87}} \qquad \text{...(6.1)}$$

where:

h_f	=	friction losses (m)
L	=	pipe length (m)
C	=	pipe roughness coefficient (C = 150 for PVC pipe)
Q	=	flow rate (m^3/s)
D	=	pipe internal diameter (m)

    - Flow for a given pipe section equals the mean flow for the section of pipe plus the incoming flow. The mean flow for a pipe section is the total flow directly connected to the section of pipe, divided by two. Friction losses must be calculated using the peak flows.

6.2.2 Vacuum collection system

According to WEF (2008) the collection system should meet the following design criteria:

- The lengths of collection lines are governed by two factors. These are static lift and friction losses. The summation of these two amounts generally cannot exceed 4 m. Due to restraints placed upon each design by topography and wastewater flows, it is impossible to give a definite maximum line length. Vacuum sewer design rules have been developed largely as a result of studying operating systems. Important design parameters are shown in **Table 6.1**.

Table 6.1: Line design parameters (USEPA, 1991)

Main line design parameters	
Minimum distance between lifts	6 m
Minimum distance of 0.2 % slope prior to a series of lifts	15 m
Minimum distance between top of lift and any service lateral	1.8 m
Minimum slope	0.2%

Line slopes*	
Diameter (mm)	**Use largest of**
100	- 0.20% slope - Ground slope - 80% of pipe diameter (between lifts only)
150	- 0.20% slope - Ground slope - 40% of pipe diameter (between lifts only)

Governing distances for slopes between lifts		
Diameter (mm)	**Distance (m)**	**Governing factor**
100	< 40	80% of pipe diameter
100	> 40	0.2 % of slope
150 & larger	< 30	40 % of pipe diameter
150 & larger	> 30	0.2 % of slope

Maximum flow and number of homes served for various pipe sizes			
Pipe diameter (mm)	**Recommended maximum flow rate (ℓ/s)**	**Absolute maximum flow rate(ℓ/s)**	**No. of homes served##**
100	2.5	3.47	70#
150	6.62	9.46	260
200	13.2	19.24	570
250	23.7	34.38	1 050

Notes: * Assuming minimum cover at top of slope
The recommended maximum length of any 100 mm run is 610 m which may limit the amount of homes served to a value of less than 70.
Substituting the values of absolute maximum flow (Q) from **Table 6.1** into the simplified equation (Q= 0,5N + 20) and solving for N, will give the maximum number of homes served for each line size.

- The general design configuration shall be based on the sawtooth pipeline profile concept (see **Figure 6.3**). Lifts or vertical profile changes are used to maintain shallow trench depths and for uphill liquid transport. A single lift consists of two 45° fittings connected with a short length of pipe.

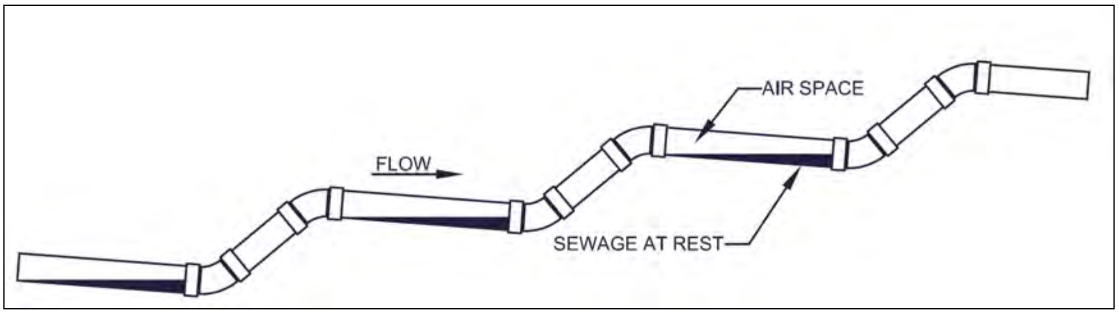

Figure 6.3: Sawtooth pipeline profile (WEF, 2008)

- The minimum pipe diameter for vacuum sewer mains and gravity service laterals is 100 mm

- The maximum length of 100 mm diameter pipeline, for any single run, is 600 m

- Lift heights should be made according to the following guidelines. In no case should a single lift exceed 0.9 m in height, see **Table 6.2**. A series of lifts should be placed with consistent lift heights.

Table 6.2: Recommended lift heights (USEPA, 1991)

Pipe diameter (mm)	Lift heights (m)
100	0.3
150	0.5
200	0.5
250	0.7

- It is preferred for changes in horizontal alignment to utilize two 45° bends connected by a short section of pipe instead of one 90° bend

- Isolation valves are required at every branch connection and at intervals of not greater than 450 m on main lines. Resilient coated wedge-gate valves shall be used. The valves shall be installed with a valve box or other approved apparatus, to facilitate proper use of the valve. Division valves are used to isolate various sections of vacuum mains, thereby allowing operations personnel to troubleshoot maintenance problems as shown in **Figure 6.4**. Some designs have included pressure-gauge tapings installed just downstream of the division valve.

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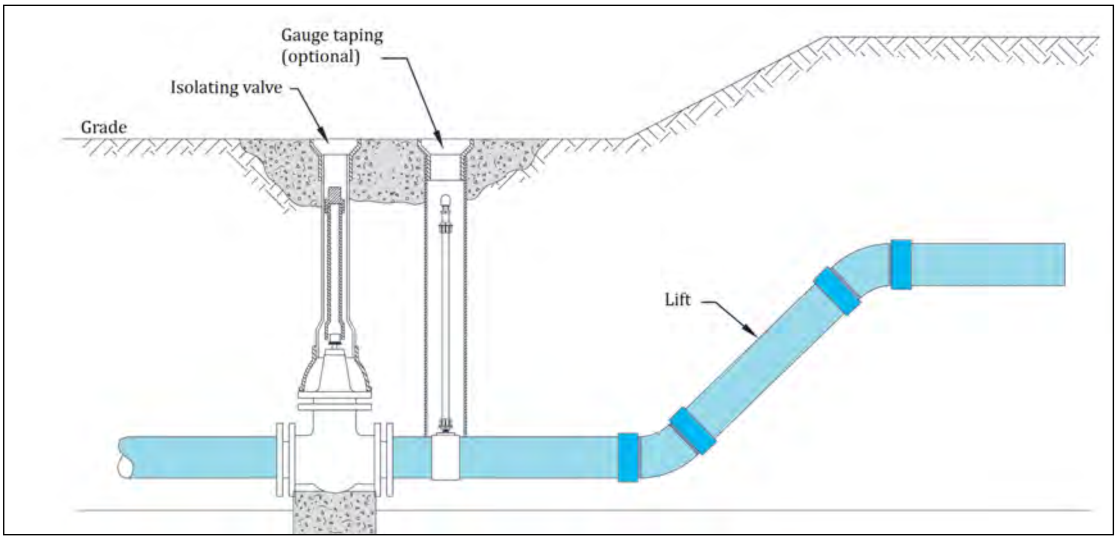

Figure 6.4: Division valve with optional gauge tap (WEF, 2008)

- Where a lift or profile change is required in a branch sewer before entering the main, it should be made 6 m or more from the main

- When vacuum mains or branches must ascend a hill, multiple lifts are placed at a minimum distance of 6 m apart. Between each lift, vacuum lines are installed with a uniform slope, so that the minimum fall (0.08 m) is achieved between these lifts.

- For a series of lifts following a downward slope in excess of 0.2%, the vacuum sewer should be installed at a slope of 0.2%, for a minimum distance of 15 m

- Recommended piping and fittings for the vacuum collection system is PVC with push-on gasketed joints for piping 100 mm in diameter and larger. Recommended piping and fittings for diameters of less than 100 mm is PVC with push-on gasketed joints or with socket-type fittings (solvent-welded).

- The preferred configuration of branch connections to the main line is made above the pipe using a vertical wye and, where required, a 45° bend

6.2.3 Vacuum-valve pit

In the design of the valve pit, shown in **Figure 3.11**, the following is recommended (WEF, 2008):

- A single-valve pit should serve a maximum of four EDUs, but no more than a maximum EDU equivalent to 0.2 ℓ/s. On a system-wide basis, the overall EDU to pit ratio shall not exceed 2.5:1.

- In no case shall a single property be served by more than one valve pit, unless proper justification is provided to support multi-valve pits

- The peak flow to any valve pit should be limited to a maximum of 0.2 ℓ/s

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- In cases where vacuum-valve pits are to be installed within a road right-of-way or other area which will be subject to vehicular traffic the pit needs to be designed accordingly to withstand the traffic loads

- The vacuum-valve pit shall be designed to prevent entrance of stormwater into the sump. The vacuum valve should also remain fully operational if submerged.

- The valve-pit arrangement shall have a receiving sump, with a minimum of 190 ℓ of storage. It is not recommended to have a system designed without a storage sump.

- Care should be taken in positioning of the vacuum-valve pit making them easily accessible to allow valves to be effortlessly removed and replaced

- Air-intakes with a minimum diameter of 100 mm shall be provided for each individual gravity line and shall protrude a minimum of 0.45 m above the ground level, as shown in **Figure 6.5**. It should be protected against physical damage and flooding and it requires a screen to prevent the entry of rodents, insects, and debris.

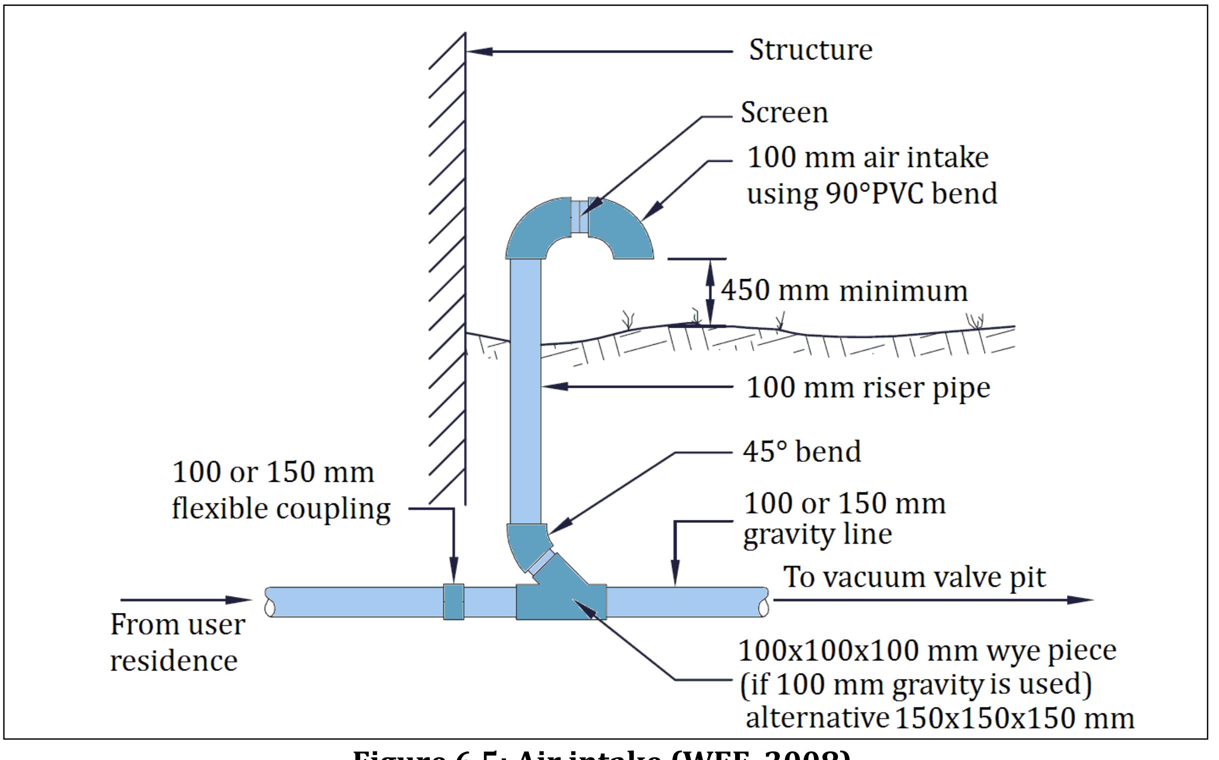

Figure 6.5: Air intake (WEF, 2008)

- Valve pits shall include gravity service connection stub-out piping, to which the sewer customer will ultimately connect. These are typically 100 mm in diameter, schedule 40, PVC, or SDR 21 pipe. Length shall usually be from the pit location to the customer's property line.

6.2.4 Vacuum valve

Most vacuum valves operate without the use of electricity. The valve is vacuum-operated on opening and spring-assisted on closing. System vacuum ensures positive valve seating. The controller/sensor is the key component of the valve. The device relies on three forces for its operation: pressure, vacuum and atmosphere. As the wastewater level rises in the sump, it compresses air in the sensor tube. This pressure initiates the opening of the valve by overcoming the spring tension in the controller and activating a three-way valve. Once opened, the three-way valve allows the controller/sensor to take vacuum pressure from the downstream side of the valve and apply it to the actuator chamber to fully open the valve. The controller/sensor is capable of maintaining the valve fully open for a fixed period of time. After the time period has elapsed, atmospheric air is admitted to the actuator chamber permitting spring-assisted closing of the valve. All materials of the controller sensor are fabricated from a plastic or elastomer that is chemically resistant to normal domestic wastewater constituents and gases. Other design requirements include:

- Vacuum valves shall have the ability to pass a 75 mm spherical solid, while matching the outside diameter of 75 mm PVC pipe. **Figure 6.6** is an example of an AIRVAC vacuum valve with wye body (AIRVAC, 2005b).

- The valves are to be vacuum-operated on opening and spring-assisted when closing

- With the exception of the gravity lateral line air-intake, there shall be no other external sources of air necessary or permitted as part of the valve assembly

Figure 6.6: AIRVAC vacuum valve

- Valve configuration shall be such that the collection system vacuum ensures positive valve seating. Valve plunger and shaft shall be arranged to be completely out of the flow path when the valve is in the open position.

- The valve shall be equipped with a sensor-controller that shall rely on atmospheric air and vacuum pressure from the downstream side of the valve for its operation, thus requiring no other external power source. The controller should be capable of maintaining the valve fully open for a fixed period of time. This should be adjustable ranging from 3 s to 10 s.

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- An internal sump breather unit arrangement shall connect the valve controller to its air source and provide a means of ensuring that no liquid can enter the controller during system shutdowns and restarts. It shall also be arranged to prevent sump pressure from forcing the valve open during low-vacuum conditions and provide positive sump venting, regardless of traps in the home gravity service line.

6.2.5 Buffer tank

For large flows that require attenuation, a buffer tank should be used. Buffer tanks are typically used to accept higher wastewater flows from schools, apartments, nursing homes, and other large users as well as flows from lift stations, or to accept flows from a number of grinder pumps. The buffer tanks are designed with a small operating sump in the lower portion, with additional emergency storage available in the tank. Other design requirements include (WEF, 2008; USEPA, 1991):

- A buffer tank is preferred instead of a single valve pit under the following conditions:
    - If there are non-residential/commercial or high-flow inputs greater than 4 EDUs or 0.2 ℓ/s peak flow
    - Cases where there are higher wastewater flows might also necessitate the use of a buffer tank
    - If there is no other practical method of serving the property by additional vacuum mains and valve pits
- A buffer tank should typically be constructed using 1.2 m diameter precast manhole sections, with the bottom section creating a sump, see **Figure 6.7**. All joints and connections on the buffer tank must be watertight. The buffer tank vacuum valve should be vented aboveground, to ensure proper venting in the event that the tank becomes filled with wastewater.
- Buffer tanks must have an operating sump of at least 38 ℓ at a wastewater depth of between 250 mm to 360 mm
- The total peak design flow that may enter through the buffer tanks on a system-wide basis should be less than 25%.
- The total peak design flow that may enter a single vacuum main through buffer tanks should be less than 50%
- One 75 mm vacuum valve shall be used for every 1 ℓ/s at peak wastewater flow. For higher flows, the wastewater shall be directed to a splitter manhole, which will evenly split and divert the flow to multiple-valve buffer tank units.

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A dual buffer tank is similar to a buffer tank, with the exception that it is larger to accommodate two vacuum valves. These tanks typically utilize 1.5 m diameter precast manhole sections. Dual buffer tanks must be connected to a 150 mm or larger vacuum main; where three or more valves are used a 200 mm vacuum main or larger is required.

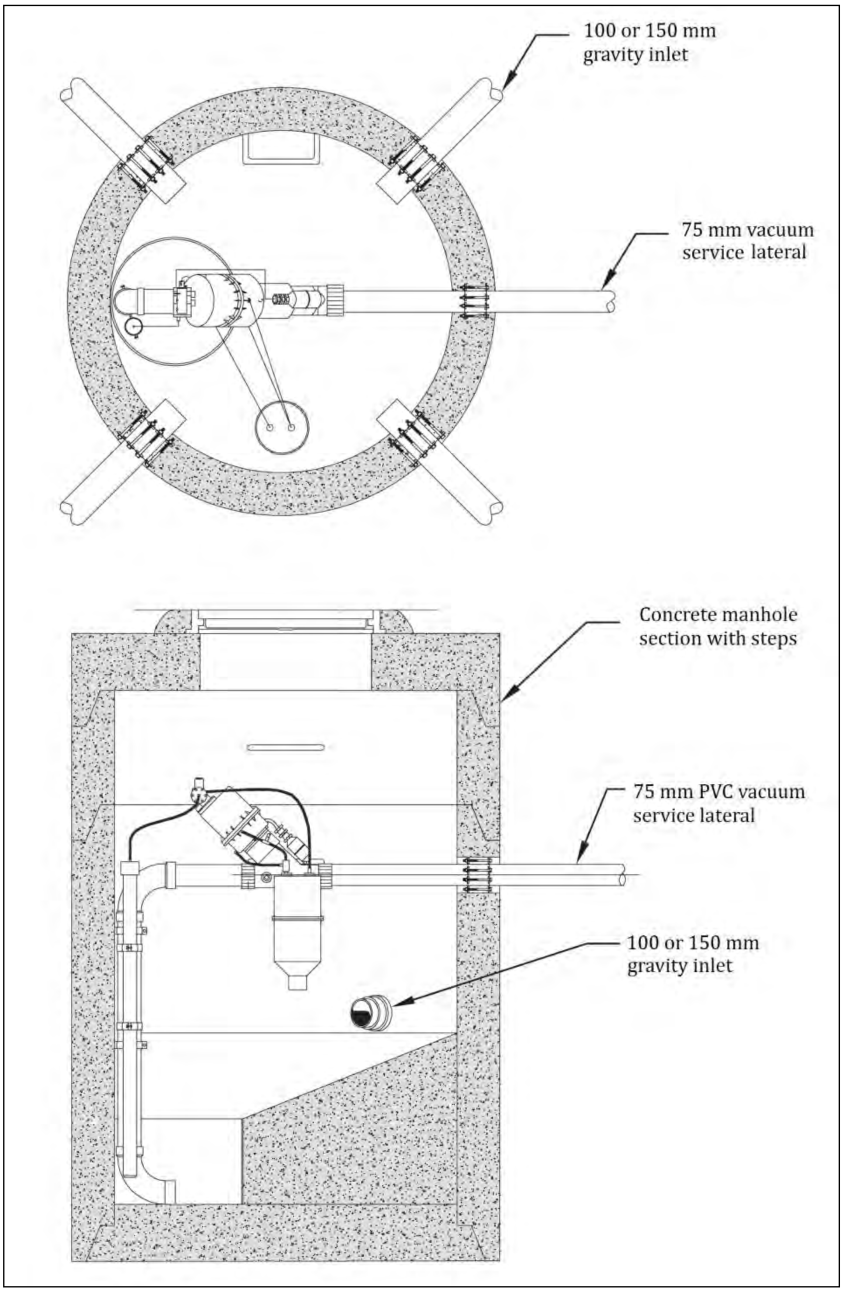

Figure 6.7: Concrete buffer tank (WEF, 2008)

The buffer tank design should make provision for separation of the valve access area (to provide access for maintenance personnel) from the sanitary wastewater storage area

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6.2.6 Vacuum station

The vacuum station is the heart of vacuum sewage collection system. Vacuum stations function as transfer facilities between a central collection point for all vacuum sewer lines and a pressurized line leading directly or indirectly to a treatment facility. The following components are included in the vacuum station, **Figure 6.8:**

- Vacuum pumps
- Wastewater pumps
- Generator
- Collection tank
- Reservoir tank
- Controls
- Motor control centre
- Data recorder
- Fault monitoring system

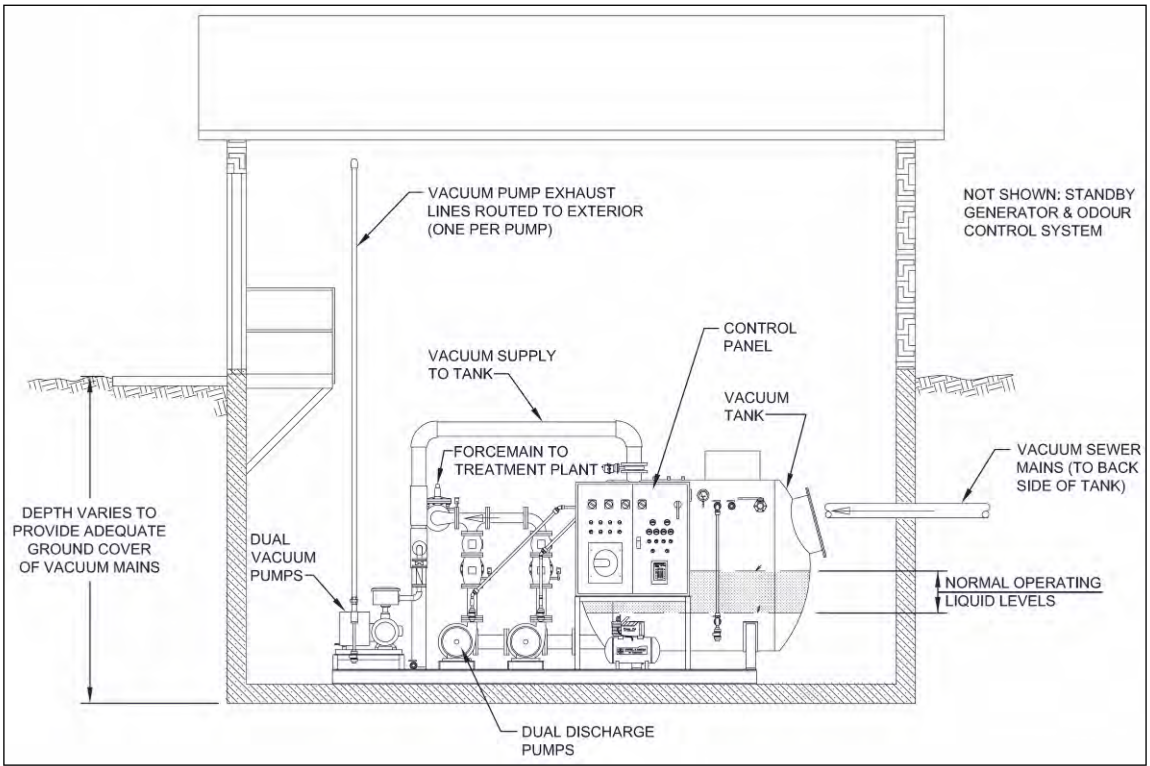

Figure 6.8: Vacuum station (WEF, 2008)

The requirements for designing a vacuum station include (WEF, 2008):

- *A minimum peak-flow-to-average-flow ratio of 3.5:1 is recommended for the component sizing in a vacuum pump station*
- *The standby power required should be capable of handling 110% peak loading. It is recommended to provide a standby generator for all vacuum stations.*
- *A minimum of one pumping unit and one standby unit should be provided for both the vacuum pumps and the wastewater pumps. Both these units should be capable of handling the peak flows.*
- *An alarm system, notifying staff operators remotely (i.e., telemetry system), should be provided. The monitoring system should have 24-hour standby operation (charged batteries), in the event of a power outage.*
- *Certification from the pump manufacturer which states that the wastewater pumps are suitable for use in a vacuum sewer installation is necessary*
- *A check valve is required on each wastewater pump discharge line*
- *Isolation valves should also be provided between the vacuum collection tank, vacuum pump(s), influent line, and wastewater discharge pipe*
- *A shutoff valve (plug or resilient coated wedge gate) is required on both the wastewater pump suction and discharge piping. It is not recommended to use butterfly valves.*
- *The necessary pipes, fittings, and valves shall be provided, to allow the vacuum collection tank to be emptied (pumped out) during an emergency*
- *Any piping or fittings larger than 100 mm in a vacuum station should be flanged ductile iron. PVC with solvent-welded joints can be used for piping and fittings less than 100 mm diameter.*
- *The ferrous metal components of the vacuum pump station should be protected against corrosion using a protective coating*
- *It is recommended that the vacuum station equipment shall be skid-mounted, fully assembled, and tested before transporting to the project site. The testing requirements should be according to the vacuum system manufacturer's standard.*

The design requirements of the components of the vacuum station as listed above are listed below. The vacuum station design nomenclature is as shown in **Table 6.3**.

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Table 6.3: Vacuum station design nomenclature

Term	Definition
Q_{max}	Station peak flow (ℓ/s)
Q_a	Station average flow (ℓ/s)
Q_{min}	Station minimum flow (ℓ/s)
Q_{dp}	Discharge pump capacity (ℓ/s)
Q_{vp}	Vacuum pump capacity (m^3/min)
V_o	Collection tank operating volume (m^3)
V_{ct}	Collection tank volume (m^3)
V_{rt}	Reservoir tank volume (m^3)
t	System pump-down time (min)
V_p	Piping system volume (m^3)
V_t	Total system volume (m^3)
TDH	Total dynamic head (m)
H_s	Static head (m)
H_f	Friction head (m)
H_v	Vacuum head (m)

Pump sizing

- The wastewater discharge pumps should be sized according to Equation (6.2):

$$Q_{dp} = Q_{max} = Q_a \text{ x peak factor} \qquad \text{...(6.2)}$$

- The total dynamic head is calculated using Equation (6.3):

$$TDH = H_s + H_f + H_v \qquad \text{...(6.3)}$$

  TDH is calculated using standard procedures for rising mains. However, head attributed to overcoming the vacuum in the collection tank (H_v) must also be considered. This value is usually 7 m, which is roughly equivalent to 500 mm Hg (typical upper operating value). Since H_v will vary depending on the tank vacuum level (400 mm Hg to 500 mm Hg, with possible operation at much lower and higher levels during problem periods), it is prudent to avoid a pump with a flat capacity/head curve.

- The minimum recommended vacuum pump size is 4.3 m^3/min (7.46 kW). A two-step process shall be used to size the vacuum pumps. Vacuum pumps should first be sized according to peak flow. The suitability of this initial sizing shall then be checked to see that the system pump-down time (t) is between 1 min and 3 min.

  - Preliminary sizing (based on peak flow). Vacuum pumps shall be sized to handle the flow from the vacuum valves adjusted to a 2:1 air–liquid inlet time ratio, by using Equation (6.4):

$$Q_{vp} = 0{,}06AQ_{max} \qquad \text{...(6.4)}$$

    where A varies empirically with mainline length, as shown in **Table 6.4**.

Table 6.4: The *A* factor for use in vacuum pump sizing (WEF, 2008)

Longest line length (m)	*A* factor
0 to 1 524	6
1 524 to 2 134	7
2 134 to 3 048	8
3 048 to 3 658	9
> 3 658	11

- Based on system volume. The adequacy of the selected vacuum pumps shall be checked to see that system pump-down time (t) is between 1 min and 3 min, according to Equation (6.5):

$$t = \frac{0{,}3375\left[\left(\frac{2}{3}\right)V_p + (V_{ct} - V_o) + V_{rt}\right]}{Q_{vp}} \qquad \text{...(6.5)}$$

If t is less than 1 min, smaller vacuum pumps must be used. If t is greater than 3 min, either larger vacuum pumps or additional vacuum pumps are required.

For Q_{vp}, use the combined rated capacity of all of the selected vacuum pumps less 1 pump. For example, if 2 vacuum pumps are used, use the rated capacity of 1 pump. If 3 pumps are used, use the combined rated capacity of 2 of the pumps.

Tank sizing

- The total volume of the vacuum collection tank shall be three times the collection tank operating volume, plus 1 514 ℓ, with a minimum size of 3 785 ℓ.

$$V_{ct} = 3V_o + 1514 \qquad \text{...(6.6)}$$

- The operating volume of the collection tank is the wastewater accumulation required to restart the discharge pump. It is usually sized so that at minimum design flow the pump will operate once every 15 min. This is represented by Equation (6.7):

$$V_o = 0{,}9\frac{Q_{min}}{Q_{dp}}(Q_{dp} - Q_{min}) \qquad \text{...(6.7)}$$

$$Q_{min} = \frac{Q_a}{2} \qquad \text{...(6.8)}$$

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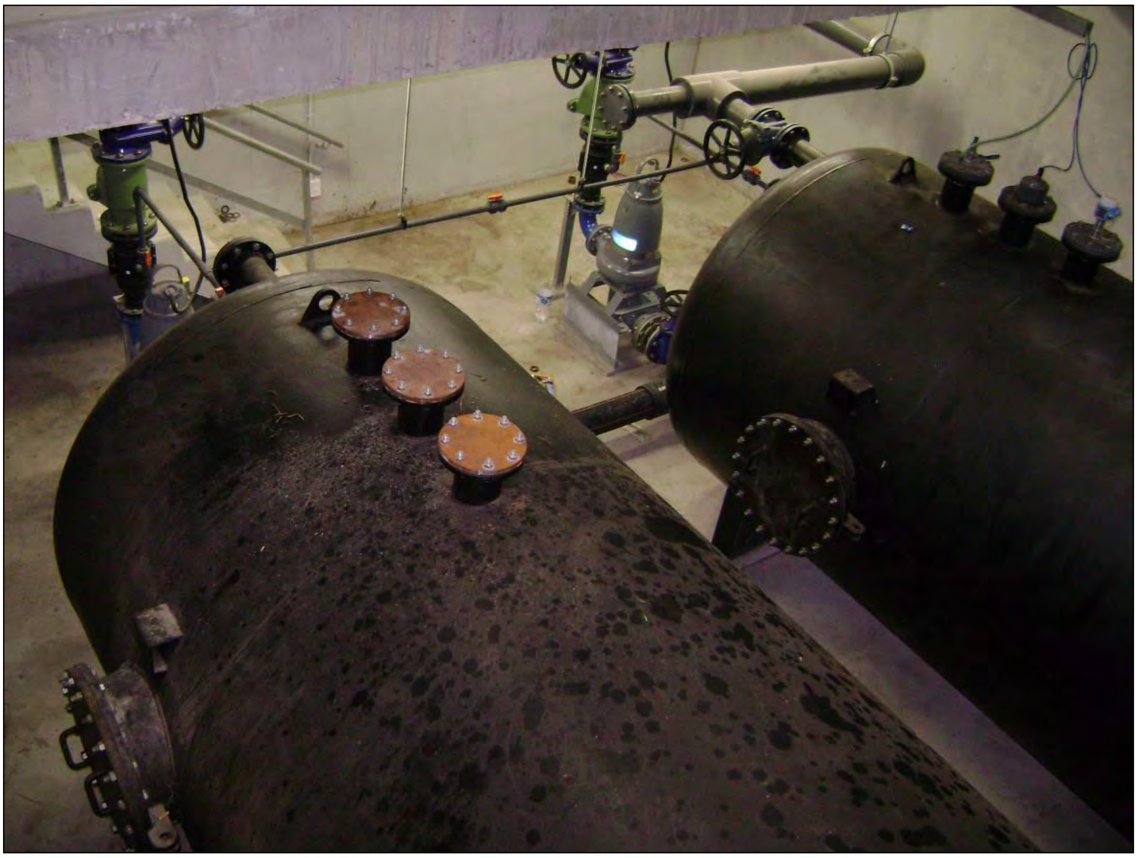

Figure 6.9: Kosovo vacuum station in the City of Cape Town (catering for 353 full-flush toilets)

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Instrumentation and control systems such as telemetry, the motor control centre and the level controllers should also be designed. Instrumentation and control systems should provide operational functionality and be incorporated into the vacuum station equipment skid(s). Instrumentation systems shall be the manufacturer's standard, as a base, with additional requirements as requested by the project owner. Provisions for automatic pump alternation must be included in the instrumentation and control system.

Other requirements to ensure a properly designed vacuum collection system include (WEF, 2008):

- *General access to the facility and for equipment maintenance should meet the local building code requirements*
- *Electrical equipment and installation shall meet all the regulatory requirements and adequate surge protection should be provided*
- *Every vacuum station should be provided with an emergency power supply. This can either be a portable emergency generator or a site-dedicated standby power generator system.*
- *Satisfactory temperature control should be provided for the electrical equipment and primary power distribution*

- *The vacuum pump station should be provided with potable water, power and telecommunication*
- *An acceptable security system is required to prevent unauthorized access with outdoor lighting*
- *Provision should be made for lightning protection*
- *The building shall meet the local authorities' requirements with regard to noise pollution*
- *An overhead crane should be provided to facilitate removal and replacement of equipment and components*
- *The structural design should cater for buoyancy forces on the vacuum pump station in areas where high groundwater conditions can be expected*
- *Users/customers should be educated*

The requirements of the customer connections are listed below:

- Customers should be connected to the vacuum system via gravity flow to the vacuum pit location
- The gravity lateral shall be in accordance with the following:
    - Lateral piping 150 mm in diameter and smaller shall be installed meeting the requirements of the plumbing sections of the local building code
    - Lateral piping shall be schedule 40 PVC or pressure-rated PVC piping
- Individual gravity laterals shall be provided from the vacuum pit to each customer
- An air intake shall be provided on each gravity lateral. The intake shall protrude above ground, with a gooseneck and a screen. The diameter of the air-intake piping and fittings shall be the same diameter as the gravity lateral. Care should be taken in placement of the air-intake fittings to prevent damage to the piping.
- The customer connection lateral must be inspected and approved by the vacuum system owner or operating entity before final connection is allowed. Only then should the vacuum valve be installed.

6.3 Vacuum collection system and vacuum station design

6.3.1 Step-by-step design procedure (vacuum collection system)

This section provides a flow diagram of the procedure to be followed for the design of the vacuum collection system (**Figure 6.10**).

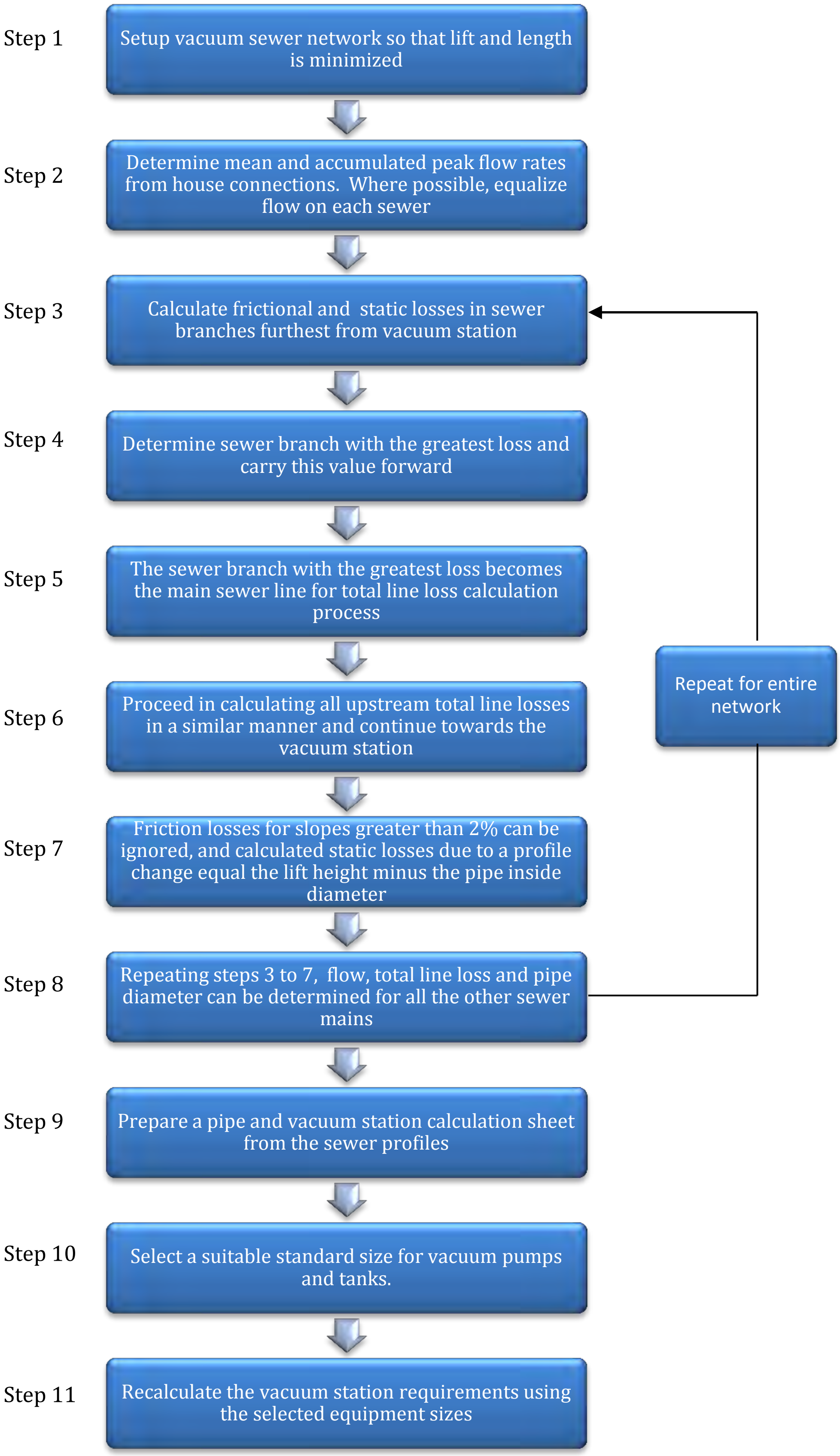

Figure 6.10: Flow diagram – vacuum collection system design

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6.3.2 Worked Example #1

This design example is a metric version of that supplied by USEPA in their document entitled: *Alternative Wastewater Collection Systems* (USEPA, 1991).

Consider the vacuum sewer layout in **Figure 6.11**. The location of the vacuum station, sewers, and valves has been selected in accordance with the requirements of **Paragraph 6.2**, which are restated below:

- Minimize lift
- Minimize length
- Where possible, equalize flows on each sewer

Each valve is assumed to serve two homes and peak flow per home is 0.04 ℓ/s or 0.081 ℓ/s per valve installation.

Three main sewers will be required to effectively serve the area depicted in **Figure 6.11**. Each of these main sewers is connected directly to the vacuum tank at the vacuum station. Sewers are not joined together into a manifold outside the station. Division valves must be located to isolate areas of the sewer network for troubleshooting purposes.

Profiles (**Figures 6.12** to **6.14**) have been prepared for a portion of Main #2. The profiles for Mains #1 and #3 would be similar.

Profiles for Main #2 follow principles stated in **Paragraph 6.2**:

- Maximum length of 100 mm sewer is 610 m
- 80% pipe diameter drop, or 0.2% fall between lifts, whichever is greater on 100 mm and smaller mains
- 40% pipe diameter drop, or 0.2% fall between lifts, whichever is greater on 150 mm and larger mains
- Where the fall in the ground profile is greater than 0.2% in the flow direction, the sewer profile follows the ground profile

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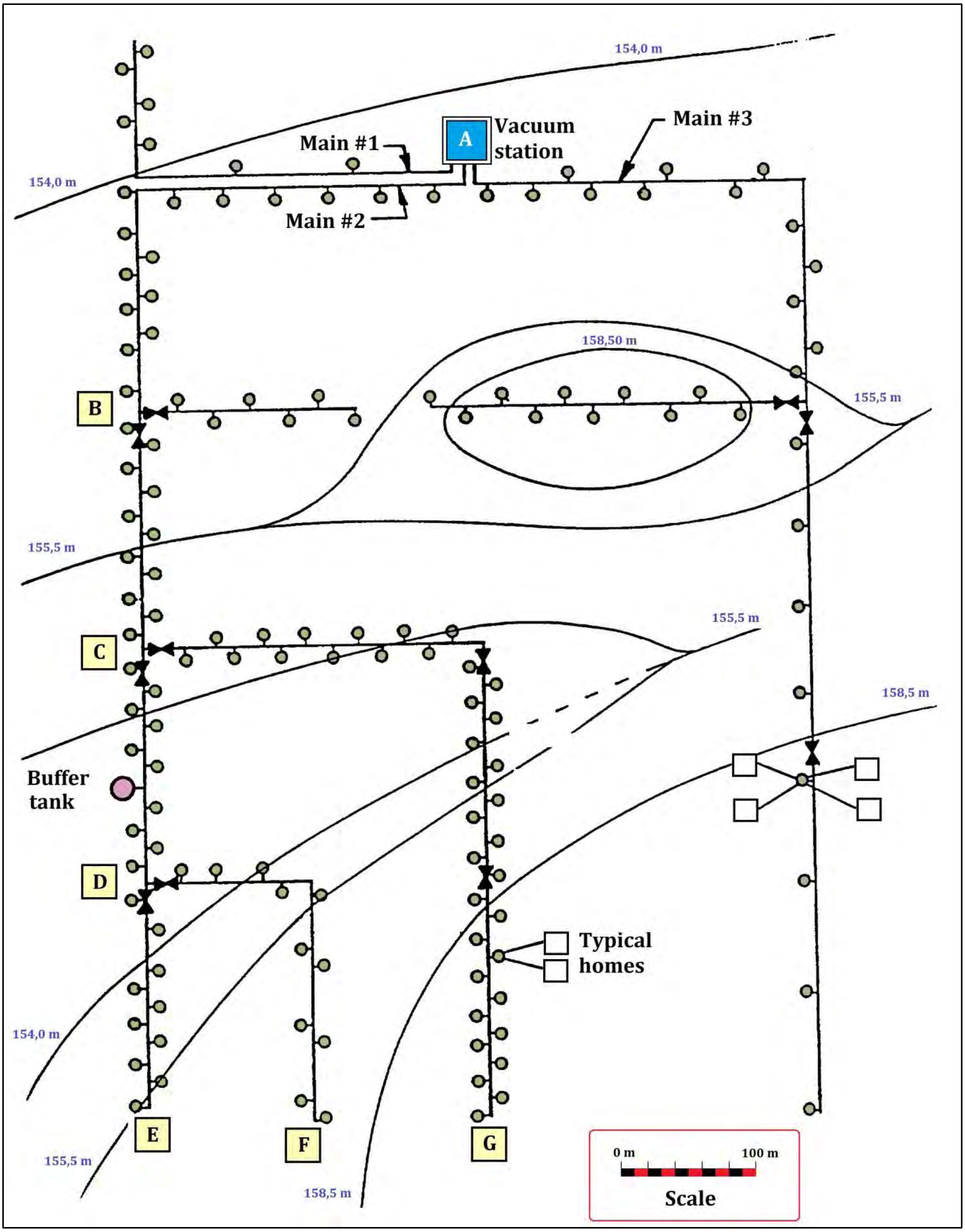

Figure 6.11: Example of vacuum sewer layout

The location of vacuum valves, division valves, and branch sewer connection points follows the principles stated in **Paragraph 6.2**.

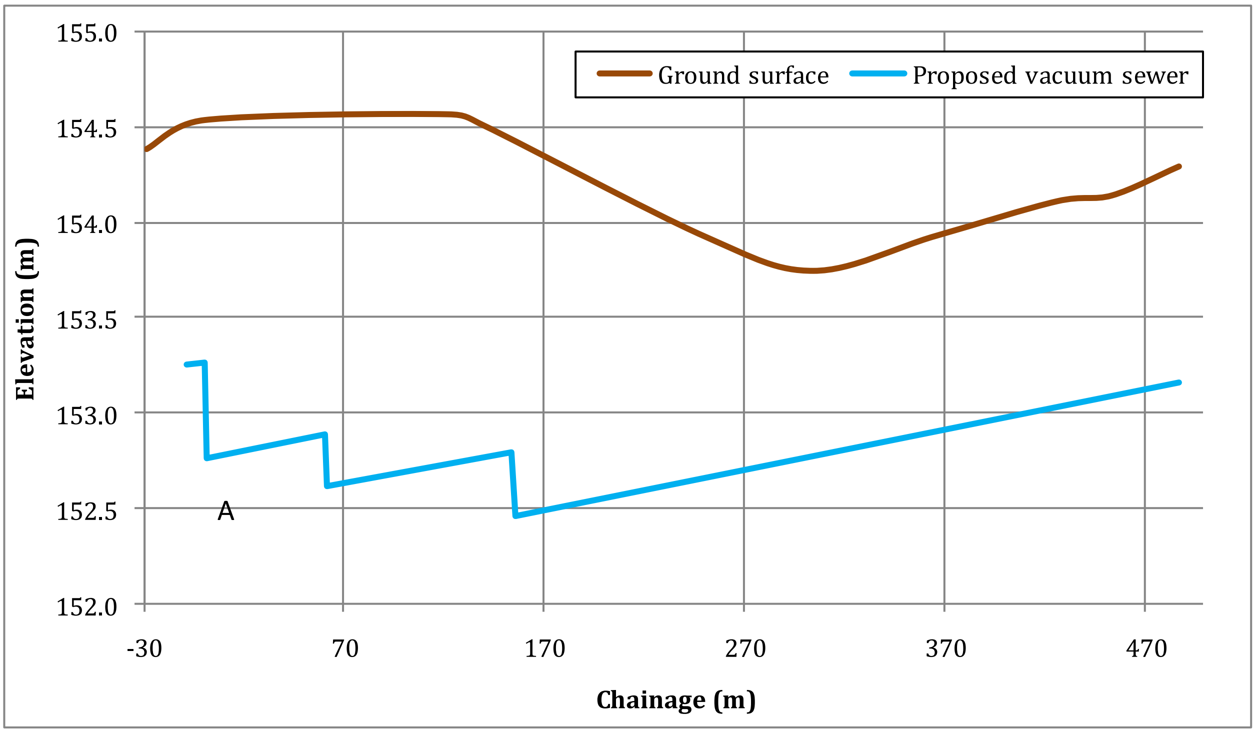

Figure 6.12: Design example profiles (Section 1)

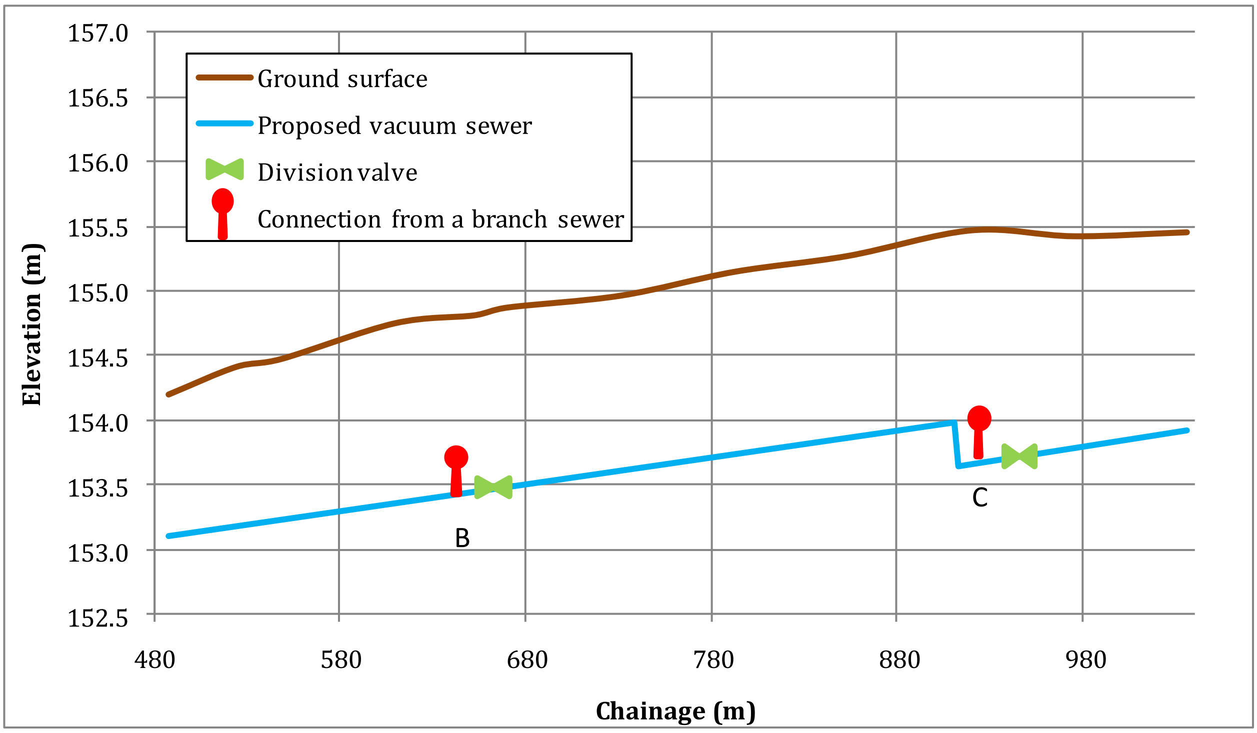

Figure 6.13: Design example profiles (Section 2)

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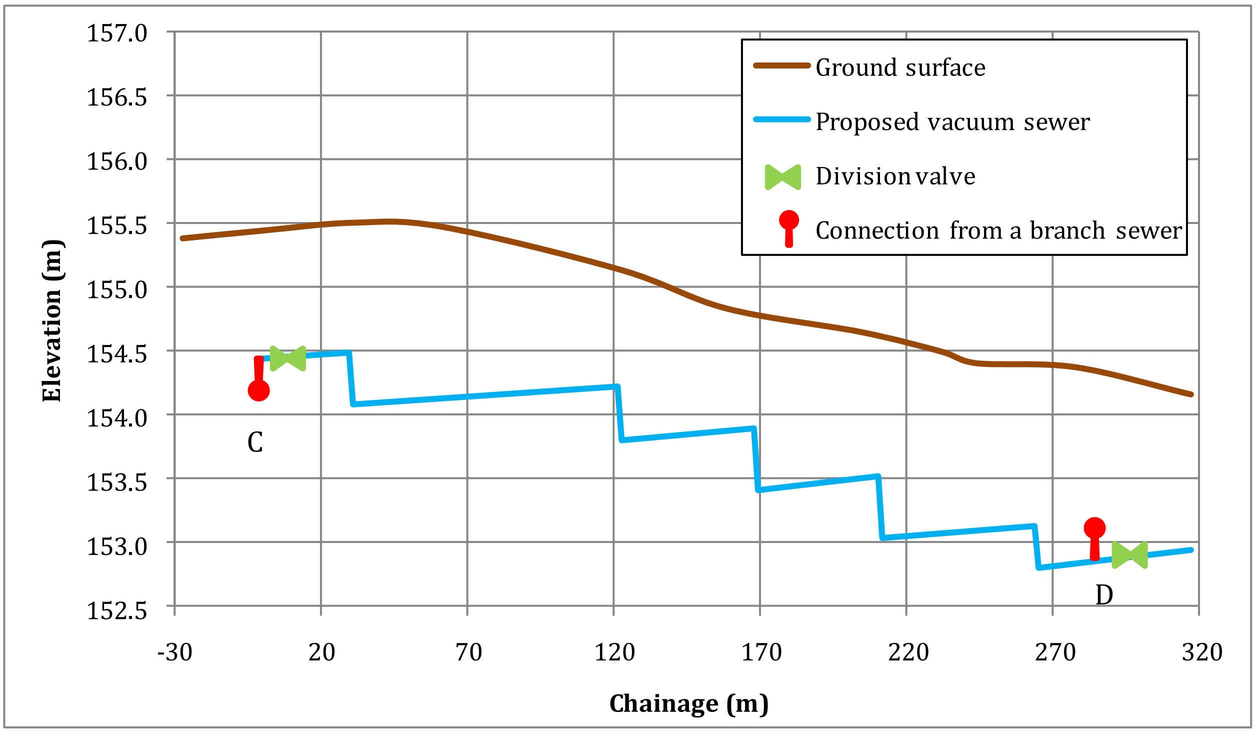

Figure 6.14: Design example profiles (Section 3)

The buffer tank valve installation shown between Points C and D is representative of a high-flow user, such as a laundromat or school; 0.63 ℓ/s is used as the rate for this location.

Main #3 is representative of a sewer main laid down an alley-way, which allows up to 4 homes to be connected to each vacuum installation.

Commence at Point F as shown in **Figure 6.11**. Calculate losses to Point D. Calculate losses and flows from Point E to Point D. Determine the line with the greatest loss and carry forward. In this example, the total line loss from Point F to Point D is greater than the total line loss from Point E to Point D. Therefore, only the total line loss from Point F to Point D is carried forward, and line F-D-C becomes the main sewer for total line loss calculation purposes.

The total line losses for all upstream flow from Point D to Point C are calculated in **Table 6.5** and continue towards the vacuum station. Note that line losses were not calculated for the branch entering Point B; this is because it is clearly evident that there are negligible losses present. This piping and additional flows have been entered at Point B for the remaining main line calculations.

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Table 6.5: Design example line loss calculations (Point F to Point C)

Station to station		Line Length (m)	Flow rate (Q) Mean (l/s)	Flow rate (Q) Accumulated (l/s)	$H_f/100$ (m/m)	Head loss line (m)	Static loss (m)	Total loss line (m)	Accumulated head loss (m)	Diameter (mm)	No. Valves
					Start with losses carried forward from F-D				0.4735		
268.22	262.13	6.10	1.62	1.62	0.0184	0.0011	0.0	0.0011	0.4746	150	
262.13	213.36	48.77	1.70	1.78	0.0198	0.0097	0.2	0.1621	0.6367	150	2
213.36	167.64	45.72	1.85	1.93	0.0250	0.0114	0.3	0.3162	0.9529	150	2
167.64	121.92	45.72	2.33	2.74	0.0352	0.0161	0.3	0.3209	1.2738	150	3
121.92	30.48	91.44	2.90	3.05	0.0528	0.0483	0.3	0.3531	1.6269	150	4
30.48	0.00	30.48	3.09	3.13	0.0594	0.0181	0.3	0.3229	1.9498	150	1
		268.22							**Total**		**12**

$Q_{mean} = Q_{accum\ previous} + Q_{ave\ for\ section}$

$Q_{accum} = Q_{accum\ previous} + Q_{accum\ this\ section}$

Branch sewer G-C joins at this point: calculate G-C before continuing with main.

Calculation for G-C yields an accumulated head loss of 1.531 m, which is less than 1.950 m. Therefore, F-D-C head losses (1.950 m) are carried over for the remaining mainline calculation, shown in **Table 6.6**.

See **Table 6.6** for calculation of line losses in Main #2, from Point C to Point A. The friction losses for slopes greater than 2.0% have been ignored, and calculated static losses due to a profile change equal the lift height minus the pipe inside diameter.

Using the same method, total line loss, flows and pipe sizes can be calculated for Main #1 and Main #3. Flows, pipe sizes, and lengths for these mains have also been estimated to allow piping and vacuum station calculations to be completed.

Table 6.6: Design example line loss calculations (Point C to Point A)

	Station to station		Line length (m)	Flow rate (Q) Mean (l/s)	Flow rate (Q) Accumulated (l/s)	$H_f/100$ (m/m)	Head loss line (m)	Static loss (m)	Total loss line (m)	Accumulated head loss (m)	Diameter (mm)	No. Valves
						Start with losses carried forward from F-D-C				1.9498		
C	918.97	912.88	6.10	6.1	6.1	0.2097	0.0128	0.152	0.1652	2.1150	150	
	912.88	807.72	105.16	6.3	6.5	0.2219	0.2333	0.000	0.2333	2.3483	150	5
	807.72	652.27	155.45	6.6	6.6	0.0718	0.1116	0.000	0.1116	2.4599	200	6
B	652.27	247.80	404.47	7.9	8.1	0.1009	0.4081	0.000	0.4081	2.8680	100	9
	247.80	152.40	95.40	8.3	8.4	0.1101	0.1050	0.116	0.2209	3.0889	200	1
	152.40	60.96	91.44	8.4	8.4	0.1132	0.1035	0.116	0.2193	3.3082	200	2
A	60.96	0.00	60.96	8.6	8.8	0.1180	0.0719	0.177	0.2487	3.5569	200	3
			918.972							**Total**		**101**

$Q_{mean} = Q_{accum\ previous} + Q_{ave\ for\ section}$

$Q_{accum} = Q_{accum\ previous} + Q_{accum\ this\ section}$

Accumulated head losses are less than 4 m, so the design is acceptable.

The last step involves the preparation of piping and vacuum station calculation sheets from the sewer profiles (see **Tables 6.7** and **6.8**).

Table 6.7: Design example piping calculations

Line	Pipe length 100 mm	Pipe length 150 mm	Pipe length 200 mm	Peak flow (LPS)	No. crossovers	No. valves	No. homes
1	731.52	426.72		4.927	31	62	124
2	1467.61	827.532	807.72	8.706	42	102	200
3	1127.76	670.56		3.148	10	31	78
Total	**3326.9**	**1924.8**	**807.72**	**16.78**	**83**	**195**	**402**
Average crossover length					12.19		
Total 75 mm pipe					1011.94		

Volume of pipework (PVC pipe)

Diameter (mm)	Area (m^2)	Length (m)	Volume (m^3)
75	0.0044	1011.94	4.47
100	0.0079	3326.89	26.13
150	0.0177	1924.81	34.01
200	0.0314	807.72	25.38
		Total (V_p)	**89.99**
		2/3 V_p	**59.99**

Table 6.8: Design example piping calculations

Peak flow	=	Q_{max}	=	1.0069 m^3/min
Average flow	=	Q_a	=	0.2877 m^3/min
Minimum flow	=	Q_{min}	=	0.1438 m^3/min
Vacuum pump capacity required	=	Q_{vp}	=	9.0623 a.m^3/min
	Select next standard pump s			10 m^3/min

$$Q_a = \frac{Q_{max}}{\text{Peak Factor}} = \frac{Q_{max}}{3.5}$$

$$Q_{min} = \frac{Q_a}{2}$$

$$Q_{vp} = A \times Q_{max}$$

Note: Minimum Q_{vp} = 4.25 a.m^3/min

Determine A from:

Longest line length (m)			A
0.0	to	1524.0	6
1524.3	to	2133.6	7
2133.9	to	3048.0	8
3048.3	to	3657.6	9
3657.9	to	4572.0	11

Discharge pump capacity	=	Q_{dp}	=	1.007 m^3/min
Collection tank operating volume	=	V_0	=	1.849 m^3
Total volume of collection tank	=	V_{ct}	=	5.558 m^3

Notes: Minimum V_{ct} = 1520 l

$$Q_{dp} = Q_{max}$$

$$V_0 = 15 \times \frac{Q_{min}}{Q_{dp}} \times (Q_{dp} - Q_{min})$$

(for 15 minute cycle at Q_{min})

$$V_{ct} = 3 \times V_0$$

or $V_0 = 1.84 \times Q_{max}$ for 3.5 Peak Factor

or $V_0 = 1.64 \times Q_{max}$ for 4 Peak Factor

Vacuum reservoir/Moisture removal tank	=	V_{rt}	=	1.514 m^3

Sytem pump down time for operating range of 400-500 mm Hg vacuum:

System pump down time	=	t	=	2.204 minutes

$$t = 0.3375 \times \frac{\frac{2}{3}V_p + (V_{ct} - V_0) + V_{rt}}{Q_{vp}}$$

Note: t should be less than 3 minutes. If over, increase Q_{vp} to get under 3 minutes.

If t under 1min, increase V_{rt}

The engineer must then select suitable standard-size pumps and tanks, usually in concert with the manufacturer, and recalculate the vacuum station requirements using the selected equipment sizes. Vacuum and wastewater pump sizes should be selected to allow for additional house connections to be made without overloading. For very large vacuum stations, 3 vacuum pumps may be used to prevent use of extremely large pumps. Typically, 18.5 kW vacuum pumps are the largest used.

7. DESIGN GUIDELINE FOR SMALL-BORE SYSTEMS

Small-bore systems or small-diameter gravity (SDG) sewers or solids-free sewers (SFSs) are also called septic tank effluent gravity (STEG) sewers, and these systems convey effluent by gravity from an interceptor tank (or septic tank) to a centralized treatment plant or pump station from where it is conveyed to another collection system (Pisani, 1998b), as depicted in **Figure 7.1**. Another variation on this alternative sewer system is the septic tank effluent pumping (STEP) concept. All these systems utilize smaller-diameter pipes placed in shallow trenches following the natural contours of the area thus reducing the capital cost of the pipe as well as excavation and construction costs.

The onsite tank could be a septic tank, aqua-privy or anaerobic digester. The sludge remaining in the tank must be removed by means of a vacuum tanker and transported to the treatment works.

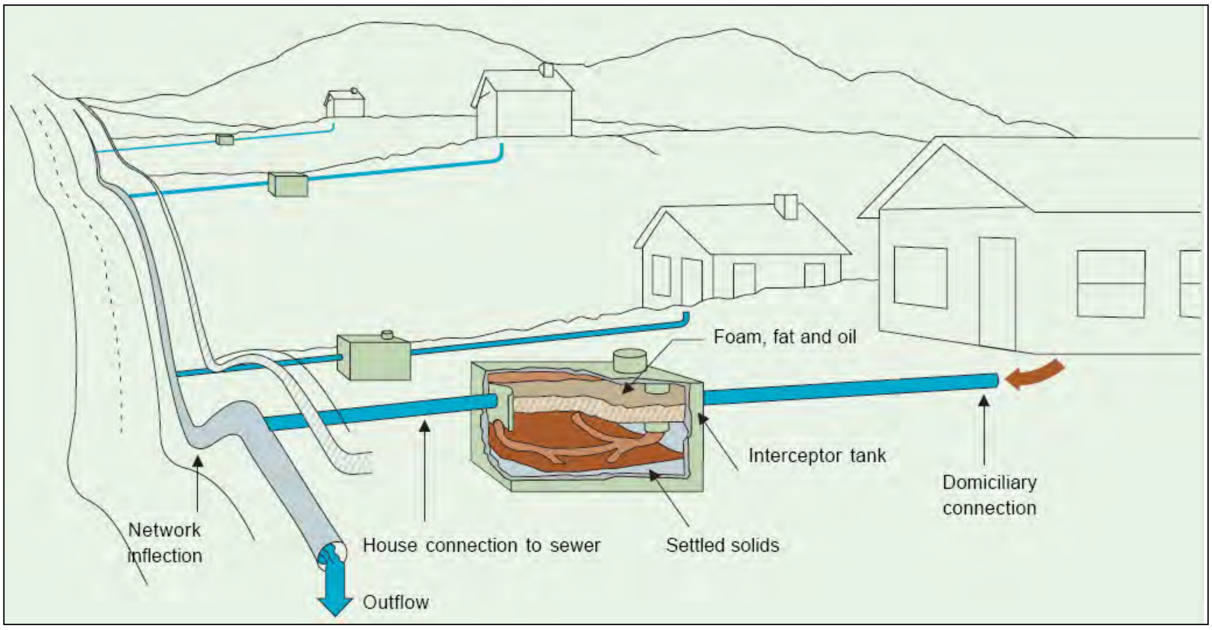

Figure 7.1: Solids-free sewer system (Alvéstegui, 2005)

The following are common conditions where solids-free sewers are more cost-effective:

- Shallow bedrock
- Undulating terrain and adverse slopes
- Low housing density
- Flat terrain
- Independent cluster developments
- Targeted areas in urban and suburban areas

Small-bore gravity sewer systems are made up of interceptor tanks and small-diameter collection mains, which are designed to convey only the liquid portion of household wastewater for off-site treatment and disposal. The grit, grease, and other solids, which could potentially cause obstruction in the sewers, are removed from the waste flow by intercepting it before the effluent discharges into the sewer. The collected solid matter that accumulates in the interceptor tank is removed periodically by vacuum truck and taken to a WWTW.

The settled wastewater is discharged from each tank via gravity (STEG) or pressure (STEP) into the collector mains, which convey the effluent to a WWTW or to a conventional gravity sewer system.

7.1 System components

A part of the small-bore sewer system has to be installed on private property. The septic or interceptor tank for a STEG system and tanks and/or pumps for STEP systems are situated on private property which could potentially be problematic for ensuring proper performance and overall successful operation of the system.

A typical small-bore sewer system consists of the following:

- Building sewers/house connections
- Watertight interceptor/septic tank
- Pumps and controls (STEP)
- Service laterals
- Collection mains
- Cleanouts
- Air-release valves and vents
- Lift stations (in some cases)

The proper design of all these components is essential for ensuring a reliable system which can be easily maintained and operated. Due to the current lack of a national design standard for small-bore sewer systems, the design engineer has a great deal of latitude in planning and designing these types of systems.

7.1.1 Building sewers/house connections

All wastewaters enter the interceptor tank, which is connected to the small-bore sewer system, through the building sewer. The wastewater from the building is conveyed to the inlet of the interceptor tank. Typically, the building sewer is a 100 mm to 150 mm diameter pipe which is laid at a prescribed slope – typically 1:60.

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7.1.2 Interceptor tanks

Septic tanks are usually used as interceptor tanks. The tank is a buried water-tight tank that is used to remove settleable solids, grit, grease, and trash in the sewage, through sedimentation and flotation. It is usually installed on the home owner's property with access for vacuum collection by means of 'honey suckers'. The effluent from the tank is transported to the sewer through the service lateral. The tank must be cleaned and inspected at regular intervals. The pumped contents (sludge) must be transported for treatment and disposal at a WWTW.

Interceptor tanks perform the following three important functions (WEF, 2008):

- *Removal of settleable and floatable solids from the wastewater;*
- *Temporary storage of the removed solids; and*
- *Flow attenuation.*

Precast reinforced concrete tanks are generally used. Fibreglass and high-density polyethylene (HDPE) tanks are also available. Precast concrete tanks are most commonly used in STEG systems, because they are generally cheaper, but polyethylene and fibreglass tanks are gaining popularity, because they are lighter in weight, making for easier installation.

7.1.3 Pumps and controls

Lift stations/pump stations are typically used on conventional gravity collector mains to lift the wastewater from one drainage basin to another as described in **Paragraph 5.3**. Similar pump stations could be required on a small-bore sewer system. Individual STEP units are essentially simplified smaller version lift stations. These can typically be a combination interceptor/lift station only required to pump the effluent from the interceptor tank by means of a submersible pump as shown in **Figure 7.2**. The required pumping head and flow rate are usually low and operation is typically controlled by means of a mercury float switch.

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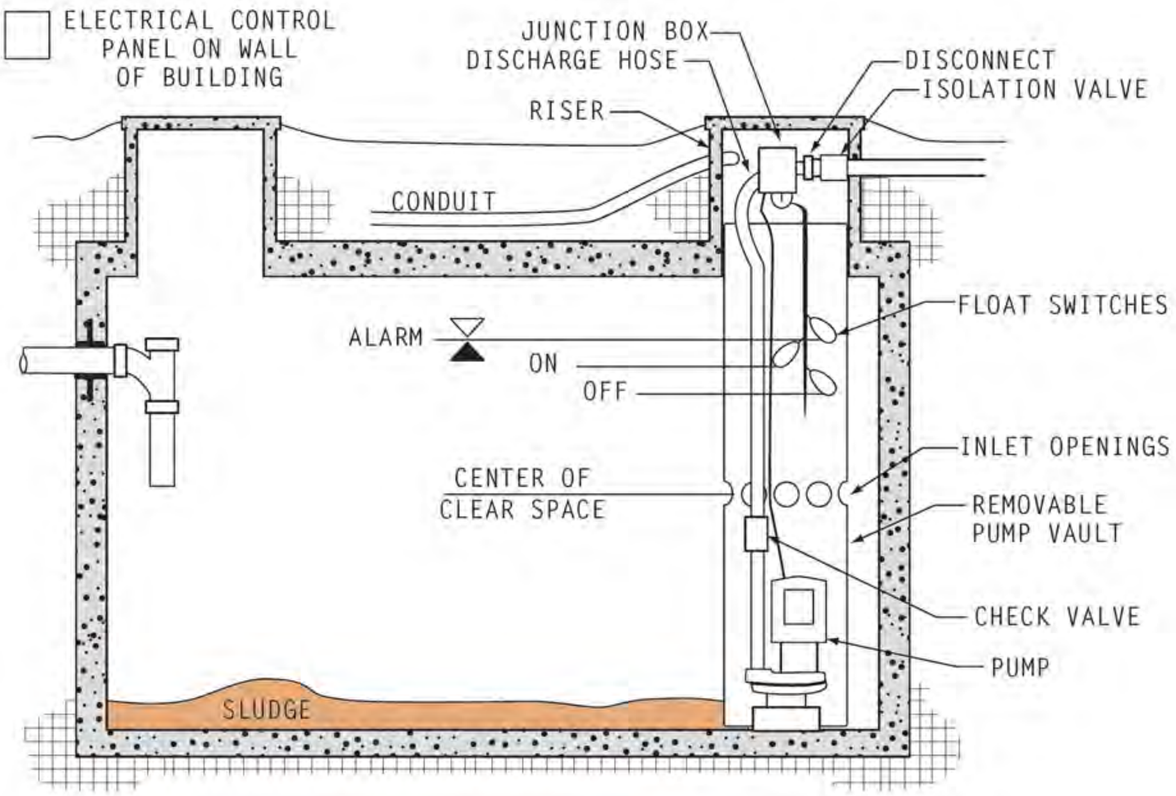

Figure 7.2: Interceptor tank/individual lift pump station (WEF, 2008)

7.1.4 Service laterals

The service laterals connect the interceptor tank to the collector main. The laterals are typically plastic pipes with diameters no larger in diameter than the collector main on STEG systems (minimum recommended is 100 mm) and for STEP systems; they are typically between 25 mm and 40 mm in diameter. Some lateral appurtenances which may be required include check valves and 'p-' or 'running' traps. The check valves are used to prevent backups in sections which have low-lying connections during peak flows and the p-traps can be added on STEG laterals, where odorous gases escape from the collectors and reach the house.

7.1.5 Collector mains

Gravity or pressure collector mains typically convey the settled wastewater to either a conventional gravity sewer system or a WWTW. PVC or HDPE pipe have been used successfully in small-bore sewers.

7.1.6 Cleanouts

Cleanouts are typically used at upstream terminals, major junctions, where there is a change in alignment or pipe size, and at high points. The cleanouts are typically installed with wye fittings, brought to just below ground surface, and enclosed in a valve box very similar to fitting a rodding eye on a conventional gravity sewer. However, the cleanouts used on small-bore sewer systems have to be fitted with a watertight, locking cap.

7.1.7 Valves and vents

Air vents are essential appurtenances used in the design of STEG/STEP collection lines. These lines are vulnerable to air locks. To prevent air locks, the frequency of line deflections below the hydraulic grade line (HGL) must be controlled, or various high points in the line should be equipped with air-release valves or air vents. An example of a combination air-release/ vacuum-valve installation is shown in **Figure 3.18**.

In STEG systems installed with continuously negative gradients, the individual house connections will provide adequate venting, if the house laterals are not trapped. Individual source connections that connect at a summit can also serve as a vent, if the service lateral is not trapped or fitted with a check valve (USEPA, 1991).

Valves can be used in a STEP system to isolate individual drainage areas and individual house connections from the total system. This provides the opportunity to perform routine maintenance and repairs on segments of the system with minimum disruption of system operation.

Check valves are recommended on each service lateral to a STEP system, to prevent backflow from the main line into the homeowner's interceptor tank. Check valves also may be required in STEG systems, in low-lying areas, where potential heads during peak flows might rise to above household drainage elevations.

7.2 Design criteria

7.2.1 Hydraulic design

A small-diameter gravity sewer system conveys settled wastewater to its outlet by utilizing the difference in elevation between its upstream and downstream ends. The sewer should be placed deep enough to enable receiving the flows by gravity from the bulk of the service connections as well as having sufficient capability to convey the anticipated peak flows.

The hydraulic losses must thus be kept within the limits of the available energy by careful selection of the pipe diameter, installation depth and gradient. In some cases the difference in elevation could be insufficient to join the service connection under gravity; a lift station must be added to the system in these cases. A cost comparison could be used to determine whether increasing the pipe diameter to minimise head losses or installing the pipe deeper would be a more economical option than providing individual lift stations.

7.2.1.1 Design-flow estimates

An estimate of the sewage/wastewater (typically the peak hour flow) should be made to hydraulically design the sewer main. As indicated in **Table 5.1** conventional sewer design assumes 900 ℓ/d (for a 3 bed-roomed dwelling) times a typical peaking factor of 3.5 for collector mains. This estimate of the design-flow rate excludes allowances for leakage and groundwater infiltration.

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Experience with small-bore gravity sewers (SBGSs) has shown that these design-flow estimates greatly exceed actual flows because most SBGSs serve residential areas where daily per capita flows are less (USEPA, 1991). The peak to average flow ratio is also less than 3.5 since the interceptor tanks attenuate the peak flows. Household wastewater flow can vary considerably between homes, but is typically 190 ℓ/cap·d.

The collector mains are sized to carry the maximum daily peak flows rather than the average flow. Instantaneous peak flows are typically 0.3 ℓ/s to 0.6 ℓ/s. The interceptor tank in SBGS systems attenuates the peaks considerably. Monitoring of individual interceptor tanks shows that outlet flows seldom exceed 0.06 ℓ/s and most peaks range between 0.03 ℓ/s and 0.06 ℓ/s over periods of 30 min to 60 min (USEPA, 1991).

The attenuation achieved in the interceptor tank depends on the design of the tank and its outlet. The design flow should allow for infiltration which in SBGS systems usually occurs in the building sewer and in the interceptor tank. Experience with SBGS systems has shown that the criteria used to estimate design flows have been conservatively high (USEPA, 1991). Design flows have generally been 190 ℓ/cap·d to 380 ℓ/cap·d with peaking factors of 1 to 4. Recent designs have been based on flows per connection of 545 ℓ/d to 635 ℓ/d.

According to the US Environmental Protection Agency (USEPA, 1991) these design-flow estimates have been successful because the interceptor tanks have storage available above the normal water level to store household flows for short peak-flow periods. The design flows of a new system can be based on similar housing, topography, and subsurface conditions of nearby systems. If comparable systems are not available, the following guideline may be used (USEPA, 1991):

$$Q_{design} = 0{,}0315N + 1{,}262 \qquad \text{...(7.1)}$$

where:

Q_{design} = estimated design flow, residential component only (ℓ/s)
N = number of contributing EDUs

7.2.1.2 Flow velocities

In SBGS systems, the primary treatment provided in the interceptor tanks upstream of each connection removes grit, and most grease and settleable solids and results in obtaining a self-cleaning velocity, as required in conventional gravity sewers, which is not required in SBGS systems.

Studies have shown that the remaining solids which enter the collectors and any slime growths which develop within the sewer are easily carried when flow velocities of 0.15 m/s are achieved (USEPA, 1991). Experience with SBGS systems has shown that the normal flows which occur within the systems are able to keep the pipe free-flowing and thus need not be designed to maintain minimum flow velocities during peak flows.

However, the conventional method for designing small-bore sewers is not based on scour velocities but rather on tractive resistance, see **Paragraph 9.5** and the WRC report *Sewer System Planning Made Simple – For Small Local Authorities* (Jacobs et al., 2010).

Maximum velocities should not exceed 4 m/s to 5 m/s. At flow velocities above this limit, air can be entrained in the wastewater and may gather in air pockets which reduce the hydraulic capacity of the collector. Drop cleanouts or drop manholes should be constructed where the pipe gradient results in excessive velocities.

7.2.1.3 Hydraulic analysis

The hydraulic equations used for design of the sewer mains are the same as those used in conventional gravity sewers, **Table 5.4**. However, unlike conventional gravity sewers, sections of SBGS systems are allowed to be depressed below the hydraulic grade line such that flows may alternate between open-channel and pressure flow. For this reason, separate analyses must be made for each segment of the sewer in which the type of flow does not change.

Design depths of flow allowed in the sewer mains have been either half-full or full. Pipe-size changes are dictated by the relative elevation of the hydraulic grade line to any service connection elevation. Design procedures follow conventional gravity sewer design except in sections where pressure flow occurs. In these sections, the elevation of the hydraulic grade during peak-flow conditions must be determined to confirm that it is lower than any interceptor tank outlet invert in these sections. There are three options to consider when the hydraulic grade line is above an interceptor tank invert:

- The depth of the sewer can be increased to lower the hydraulic grade line
- The diameter of the main collector can be increased to reduce the frictional head loss
- A STEP unit can be installed at the affected connection to lift the wastewater into the collector

In cases where it is simply a short-term surcharge above the interceptor tank outlet invert, then placing a check valve on the individual service lateral may be sufficient to prevent backflow.

7.2.2 Collector mains

7.2.2.1 Layout

The SBGS layout is a dendriform or branched system similar to that of conventional sewers.

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In most cases, SBGS pipes are located alongside the pavement in the street right-of-way. In cases where there are numerous services on both sides of the street, collector mains may be provided on both sides to avoid road crossings.

Another alternative is to locate the collectors down the back property lines, i.e. midblock, to serve an entire block with one collector. The midblock alternative may be the most accessible to homeowners since most septic tank systems are located in the back although the disadvantage of this alternative is that access of the interceptor for maintenance is restricted.

7.2.2.2 Alignment and grade

SBGS pipes don't have to follow straight lines. Some of the more obvious obstacles such as utilities, large trees, etc., can be avoided with careful planning, but unforeseen obstacles can often be avoided by simply bending the pipe (not exceeding the maximum radius recommended by the pipe manufacturer). The gradient of SBGS systems must provide an overall fall which is sufficient to carry the estimated hourly peak flows, but the vertical alignment need not be uniform. Inflective gradients, where sections of the main are depressed below the static hydraulic grade line, are allowable if the invert elevation is controlled where the flow in the pipe changes from pressure to open-channel flow. The elevation of these summits must be established such that the hydraulic grade line does not rise above any upstream interceptor tank outlet invert during peak-flow conditions. Adequate venting must also be provided at the summit. Between these critical summits, the profile of the sewer should be reasonably uniform so that unvented air pockets do not form which could create unanticipated head losses in the conduit and excessive upstream surcharging.

7.2.2.3 Pipe diameter

The pipe diameter is determined through hydraulic analysis. It varies according to the number of connections and the available gradient. The minimum diameter used is typically 100 mm, although 50 mm diameter pipes have been used successfully. If 50 mm diameter pipes are used, the interceptor outlets should be equipped with flow-control devices to limit peak flows from the tank. Check valves are also required to prevent flooding of service connections during peak-flow periods. The costs of these flow-control devices and check valves generally cancel savings realized from the smaller pipe. It is therefore recommended to use a 100 mm diameter pipe as a minimum size.

7.2.2.4 Depth

The depth of burial for the collector mains is determined by the elevation of the interceptor tank outlet invert elevations and anticipated trench loading conditions. Designers usually do not attempt to set the depth such that all connections can drain by gravity. Where gravity drainage from a residential connection is not possible, STEP lift stations are used.

Where the pipe is buried below the pavement or subject to traffic loadings, the minimum depth is typically 750 mm; however, a depth of 600 mm below ground is considered to be the minimum for conventional pipe. Pipe manufacturers should be consulted to determine the minimum depth recommended.

7.2.2.5 Pipe materials

PVC plastic pipe is the most commonly used pipe material in SBGS systems. Usually, elastomeric (rubber ring) joints are used; however, for pipes smaller than 75 mm in diameter, only solvent weld joints may be available. HDPE has been used infrequently, but successfully.

7.2.3 Service laterals

Typical service laterals between the tank and the sewer line are 100 mm diameter PVC pipe. The service lateral should be no larger than the diameter of the collector main to which it is connected. The connection is typically made with a wye or tee fitting. Where STEP units are used, wye fittings are preferred. In some cases, check valves are used on the service lateral upstream of the connection to the main to prevent back-flooding of the service connection during peak flows. If a check valve is used, it is important that the valve be located very close to the collector main connection. Air binding of the service lateral can occur if the valve is located near the interceptor tank outlet.

7.2.4 Interceptor tanks

The geometry of the septic tank influences the flow velocity at which the sewage passes through the tank, the accumulation of sludge as well as the possible presence of stagnant areas in the tank.

It is usually recommended to use double compartments which will reduce the peak flow due to increasing surface area and will allow the higher concentration of solids to first settle. A good design would typically include:

- A liquid depth of between 1.0 m and 1.8 m
- Rectangular shape with length three times the width
- The first compartment twice the volume of the second compartment

It is recommended that the inlet to the first compartment be a sanitary T-piece or baffle wall. The vertical portion of the T-piece should extend below the surface liquid, to minimize incoming turbulence. The lower vertical arm of the inlet should be submerged between 30% and 40% of the liquid depth. The upper vertical arm of the T-piece should extend at least 50 mm above the crown of the inlet and end 15 mm below the cover of the tank. The invert of the inlet pipe should be between 50 mm and 75 mm above the surface of the liquid (SABS, 1993). In the case of a baffle it must be open at the top to allow venting of the interceptor tank through the building plumbing stack.

The sewage in the tank passes from the first compartment to the second through a mid-depth opening. The outlet from the second compartment should also be a sanitary T-piece or baffle wall, see **Figure 3.17**. All arms of the T-piece should have an inside diameter of half to three-quarters of that of the inlet pipe, thus damping peak inflows. The invert of the outlet pipe should be between 50 mm and 75 mm below that of the inlet pipe (SABS, 1993).

Some design criteria indicate that septic tanks shall have an effluent filter placed at the outlet in place of the outlet baffle. The purpose of the filter is to trap suspended solids that are not heavy enough nor have had time enough to sink to the bottom of the tank (as in a tank that hasn't been pumped in a timely manner and has significant amounts of material that reduce its effective volume). Filters must, however, be periodically cleaned so that they do not plug and back sewage into the house (see **Figure 7.3**). Access openings must be provided to service the tank and filter screens.

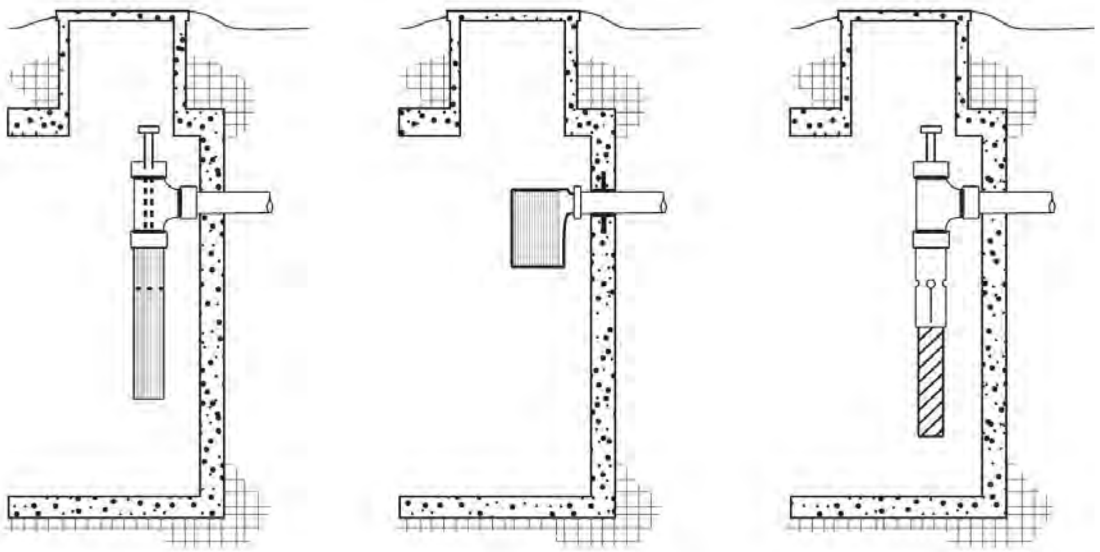

Figure 7.3: Typical effluent screen filters (WEF, 2008)

The capacity of the septic tank should be adequate to store sludge and scum, as well as to retain liquid for a period of at least 24 h just before the tank requires desludging. The flow to the septic tank is directly related to the level of water supply to the residential building. Therefore the level of water supply to the building can be used to determine the capacity required. There are basically three methods to determine the capacity of the tank (SABS, 1993):

- For non-residential systems, estimate the average daily flow from the establishment. The capacity of the septic tank has to be 3 times the estimated average daily flow.

- For dwellings or dwelling units with full in-house water reticulation, relate the capacity of the septic tank required to the number of beds or bedrooms see SANS 10252-2:1993 (SABS, 1993).

- For special residential systems such as multi-home systems or dwellings units without full in-house water reticulation, relate the capacity of the septic tank required to the number of persons to be served by the system, see SANS 10252-2:1993 (SABS, 1993).

The tank is sized to provide sufficient capacity to limit the periodic removal of sludge to between 2 years and 5 years, i.e. creating user convenience and being economical.

The interceptor tanks should be located where they are easily accessible for periodic removal of accumulated solids. A sufficiently large opening over the tank inlet or outlet should be provided to allow inspection and effective sludge removal. However, because of the tanks' septic conditions, unsupervised or unaccompanied personnel must not enter the tank. All applicable safety codes must be followed in the design of these facilities.

The opening should be a minimum of 450 mm square or in diameter. A watertight riser terminating 150 mm above grade with a bolted or locking air-tight cover is preferred to a buried access.

Prefabricated septic tanks are typically used for interceptor tanks. They are available in reinforced concrete, coated steel, fibreglass and high-density polyethylene. All tank joints must be designed to be watertight. The joints include tank covers, manhole risers and covers and inlet and outlet connections. Rubber gasket joints for inlet and outlet connections are preferred to provide some flexibility in case of tank settlement.

7.2.5 Manholes and cleanouts

In most SBGS systems, cleanouts are used instead of manholes, except at major junctions at mains. Cleanouts provide sufficient access to the mains and are less costly to install than manholes and are not a source of infiltration, inflow or grit. Cleanouts are usually placed at upstream termini of mains, junctions of mains, or changes in main diameter and at intervals of 120 m to 300 m. The cleanouts are typically extended to ground surface within valve boxes (**Figure 7.4**).

Manholes, if used, are located only at main junctions. The interiors should be coated with epoxy or other chemical-resistant coating to prevent corrosion. The covers used are typically air-tight covers to limit the egress of odours and inflow of clear water.

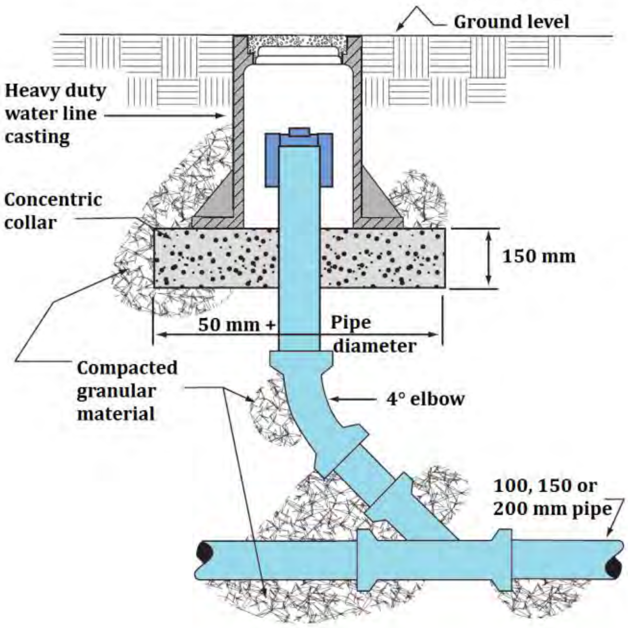

Figure 7.4: Typical STEP cleanout structure (WEF, 2008)

Where depressed sections occur, the sewer must be well vented. Cleanouts may be combined with air-relief valves at high points in the mains or an open vent cleanout may be installed.

7.2.6 Valves

Air-release, combination air-release/vacuum and check valves may be used in SBGS systems. Air-release and combination air-release vacuum valves are used for air venting at summits in mains that have inflective gradients in lieu of other methods of venting. These valves must be designed for wastewater applications such as the Vent-O-Mat RGX combination air-valve range as depicted in **Figure 3.18**. In cases where odours are detected from the valve boxes, the boxes may be vented into a small buried gravel trench beside the boxes.

Check valves are sometimes used on the service connections at the point of connection to the main sewer to prevent backflow during surcharged conditions in the collector sewers. They have been used mainly in systems with 50 mm diameter mains. Many types of check valves are manufactured, but those with large, unobstructed passageways and resilient seats have performed best. Wye pattern swing-check valves are preferred over tee-pattern valves when installed horizontally. Although the systems with 100 mm diameter mains have operated well without check valves, their use can provide an inexpensive factor of safety for these applications as well. According to the US Environmental Protection Agency (USEPA, 1991) in Australia, a ‘boundary trap’ is included at every connection which provides an overflow to the ground surface if backups occur.

7.2.7 Odours and corrosion

Odours are a commonly reported problem with SBGS systems. The settled wastewater collected by SBGSs is septic and therefore contains dissolved H_2S and other malodorous gases. These gases tend to be released to the atmosphere in quantity where turbulent conditions occur such as in lift stations, drop cleanouts or hydraulic jumps which occur at rapid and large changes of grade or direction in the collector main. The odours escape primarily from the house plumbing stack vents, manholes or wet-well covers of lift stations.

The odours can be controlled by minimizing turbulence and sealing uncontrolled air outlets. An effective odour-control measure is to terminate the vent in a buried gravel trench or utilizing carbon filters although these require regular maintenance.

More recent SBGS systems have used wet-well/ dry-well designs for lift stations to reduce the exposure of mechanical components to the corrosive atmosphere.

7.3 House connector to interceptor tank

The design process of the connection pipe from the house to the septic tank is very similar to that for house connection pipes in conventional gravity sewer systems (refer **Paragraph 5.7**), except for the following additional requirements (SABS, 1993):

The incoming drain pipe to an interceptor/septic tank shall:

- Have an inside diameter of at least 100 mm
- Be laid at a gradient not exceeding 1 in 60 for the last 10 m before it enters the tank
- Be fitted with a T-junction at the point of discharge into the receiving chamber

7.4 Interceptor tank

This section provides a step-by-step set of procedures to be followed to design an interceptor tank for a small-bore gravity sewer system.

7.4.1 Step-by-step design procedure (interceptor/septic tank: **Method 1** – middle to upper income groups)

Figure 7.5 presents a flow diagram of the steps required in designing an interceptor tank.

<u>Step 1: Determine number of bedrooms in household</u>

This method, where the capacity of the septic tank required is related to the number of bedrooms in the house, is applicable mainly to middle-income to upper-income areas because there is often a relationship between the number of occupants in a house and the number of bedrooms (SABS, 1993).

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Step 2: Determine the frequency of cleaning

Normally, in a municipal area where maintenance structures provide cleaning services to residents, the frequency of cleaning is known and fairly constant. In cases where frequencies are unknown, the designer should carry out a cost-benefit analysis to determine the optimum between the capital cost of the size of the tank and the maintenance costs of frequent cleaning.

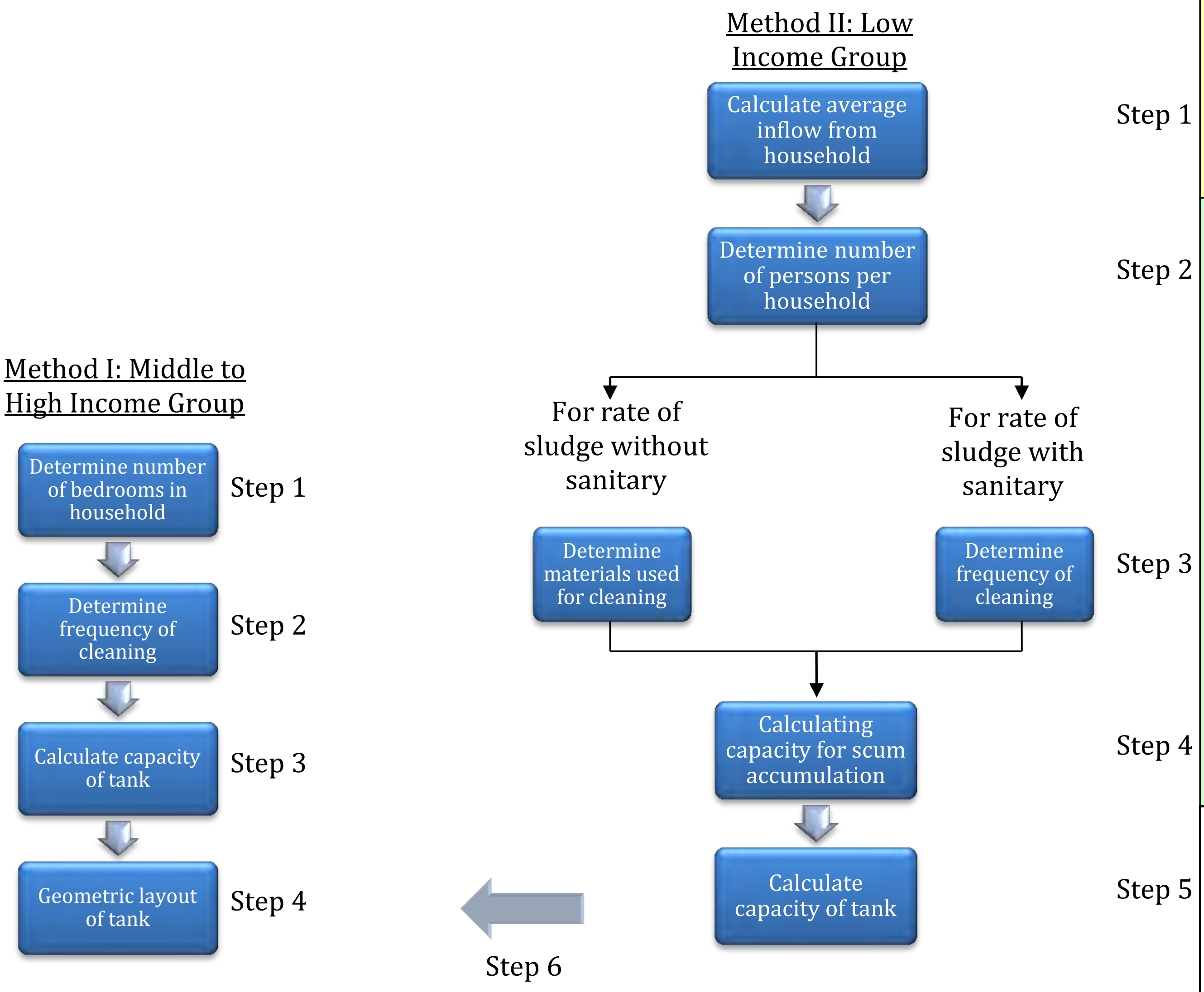

Figure 7.5: Flow diagram - interceptor/septic tank design

Step 3: Calculate capacity of tank

Utilizing **Figure 7.6** the size of the septic tank required can be determined. The graph provides for a 24 h liquid retention period and the septic tank capacities indicated should prevent any appreciable discharge of scum and sludge (SABS, 1993).

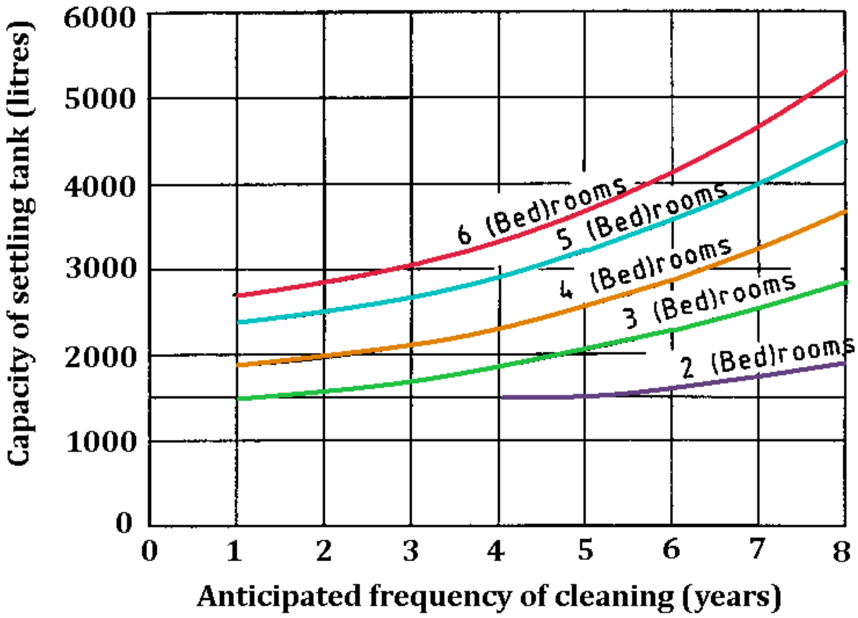

Figure 7.6: Method 1 – Septic tank capacity related to size of dwelling (SABS, 1993)

Step 4: Geometric layout of tank

The geometry of the tank has an influence on the velocity at which the sewage flows through it, on the sludge accumulation and on the possible presence of stagnant pockets of liquid inside the tank. When the tank is too deep in relation to its surface area, the plan dimensions will be too small and a direct flow of sewage (short-circuiting) can take place between the inlet and the outlet, resulting in a reduced retention time for the liquid.

Where the septic tank has too large a liquid surface area in relation to its volume, the clear space between the sludge and the scum will decrease, resulting in too high a liquid flow rate for sedimentation and flotation to take place.

Septic tanks should therefore be designed to have:

- A liquid depth (*L*) of between 1 m and 1.8 m
- A rectangular shape, the length of the septic tank being three times its width (*W*)
- A first compartment of twice the size of the second compartment (if required)

The following calculation (Equation (7.2)) can be used to calculate the septic tank dimensions:

$$W^2 = \frac{C}{3 \times D} \qquad \text{...(7.2)}$$

where:

W	= is the width of the tank (m)
C	= is the required capacity (m^3)
D	= is the selected depth (m)

Figure 7.7 illustrates a typical interceptor/septic tank geometric layout.

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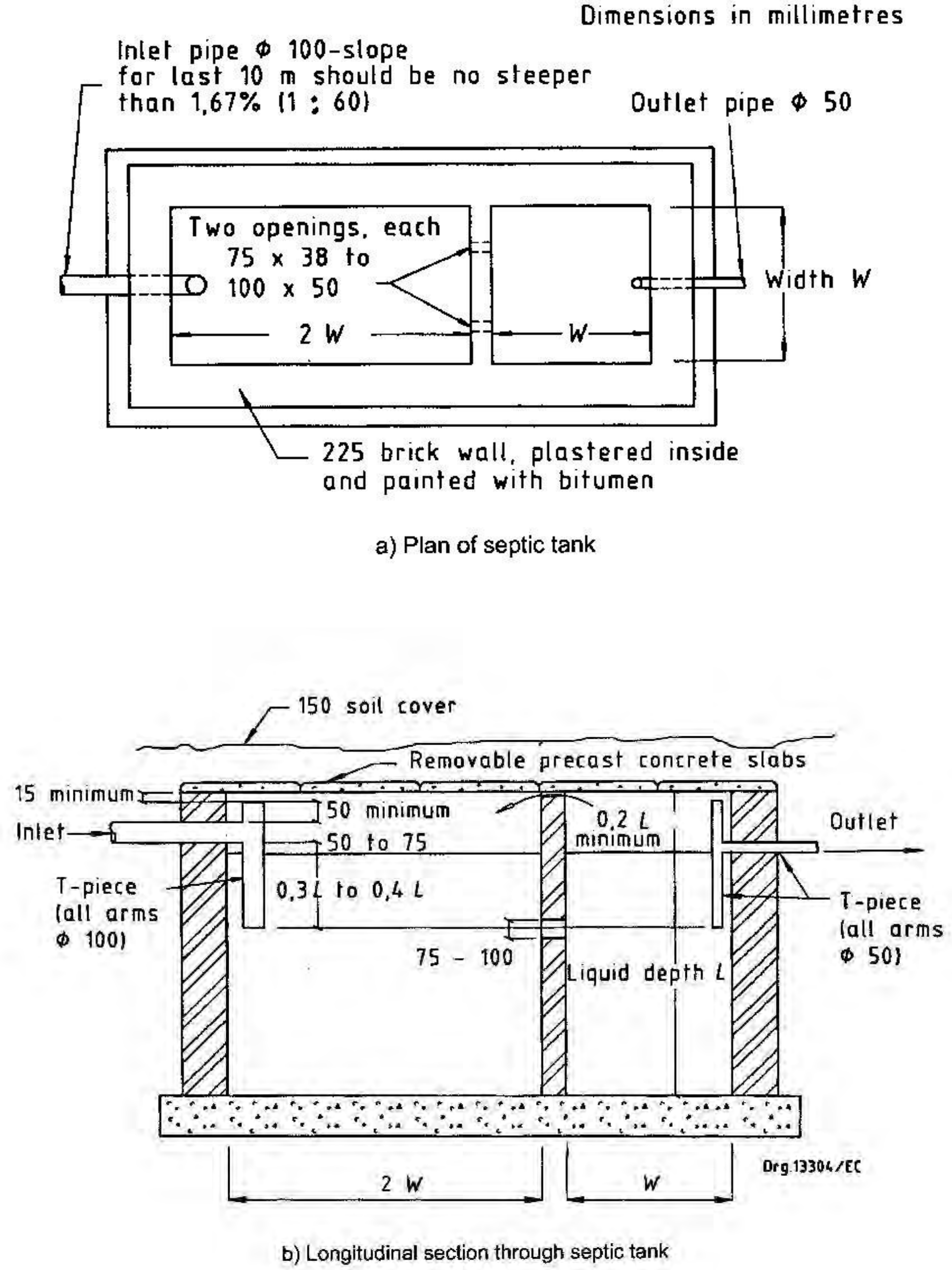

Figure 7.7: Typical interceptor/septic tank geometrical layout (SABS, 1993)

7.4.2 Step-by-step design procedure (interceptor/septic tank: **Method 2** – lower income groups)

<u>Step 1: Calculate average daily sewage inflow from household</u>

The average daily household sewage inflow should be calculated according to the procedures prescribed in **Paragraph 5.7.2**.

Step 2: Determine number of persons per household

With this method, the capacity of the septic tank is determined from the number of persons served. Therefore, population figures based on an on-site census will be needed.

Step 3: Determine materials used for anal cleansing/Determine frequency of tank cleaning

In poorer households that have no multiple sanitary fixtures, materials other than water or paper are often used for anal cleansing. If no census information is available, the designer must estimate a proportion of the population that would be subjected to this form of sanitary disposal.

In households with multiple sanitary fixtures, the frequency of tank cleaning shall be dependent on the availability of cleaning services (such as vacuum tankers) and the cost of cleaning the tank. The designer should carry out a cost-benefit analysis to determine the optimum between the capital cost of the size of the tank and the maintenance costs of frequent cleaning

Step 4: Calculating capacity for sludge and scum accumulation

The rate of sludge accumulation in such circumstances should be determined with the guidance from data given **Table 7.1**.

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Table 7.1: Rate of sludge and scum accumulation for dwellings without multiple sanitary fixtures (SABS, 1993)

Materials used for anal cleansing	Rate of accumulation ℓ/person·yr
Sand, stone, etc.	
Toilet wastes only	55
Additional household sewage	70
Hard paper, leaves and grass	
Toilet wastes only	40
Additional household sewage	50
Water and soft paper	
Toilet wastes only	25
Additional household sewage	40

The rate of sludge and scum accumulation for dwellings with multiple sanitary fixtures is shown in the **Table 7.2**.

Table 7.2: Rate of sludge and scum accumulation for dwellings with multiple sanitary fixtures (SABS, 1993)

Years of service	Rate of collection ℓ/person·yr		
	Sludge	Scum	Total
1	65	20	85
2	105	35	135
3	125	50	175
4	145	65	210
5	170	85	255
6	195	95	290
8	240	120	360
10	295	145	440

Step 5: Calculating capacity of tank

The required capacity of the septic tank required can be determined as follows:

- Estimate the expected average daily sewage flow (from **Table 5.1**)
- Establish the capacity needed for sludge and scum accumulation in the first compartment (refer to **Tables 7.1** and **7.2**)
- Calculate the total capacity of the septic tank as follows:

$$V_{\text{tank}} = Q + P \qquad \text{...(7.3)}$$

where:

V_{tank} = the required capacity of the septic tank, but not less than 3 x Q or 1 700 ℓ

Q = the estimated daily sewage flow from **Table 5.1** (ℓ)

P = the capacity required to store sludge and scum between septic tank cleanings (ℓ)

Step 6: Geometric layout of tank

Refer to Step 4 of the geometric layout of **Method 1** (**Paragraph 7.4.1**).

7.4.3 Worked Example #1 – Interceptor/septic tank design

Design an interceptor tank to pretreat the wastewater from a household of 6 persons. The low-income group house is fitted with multiple sanitary fixtures. The tank is to be desludged every three years. The solution is as follows:

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Step 1: Calculate average daily sewage inflow from household

From **Table 5.1**, the average daily sewage inflow from a 6-person low-income household is:

Q = 70 ℓ/cap·d x 6 persons

= 420 ℓ/d

Step 2: Determine number of persons in household

Given as 6 persons.

Step 3: Determine materials used for anal cleansing

Normal toilet paper is used.

Step 4: Calculate required capacity for sludge and scum accumulation

From **Table 7.2**, the capacity is as follows:

P = sludge (125 ℓ) + scum (50 ℓ)

= 175 ℓ/cap x 6

= 1 050 ℓ

Step 5: Calculate total capacity of the tank:

The total capacity of the tank is:

V_{tank} = $Q + P$

= 420 ℓ+ 1 050 ℓ

= 1 470 ℓ

However, the required capacity should not be less than 3 x Q or 1 700 ℓ, therefore

V_{tank} = 1 700 ℓ (1.7 m^3)

Step 6: Geometric layout of tank

Using Equation (7.2), the dimensions can be calculated as follows:

The width (W) is:

$$W^2 = \frac{C}{3 \times D}$$

where:

C = 1.7 m^3

D = 1.5 m (assume)

Therefore,

$$W = \sqrt{\frac{1,7}{3 \times 1,5}} = 0.615 \text{ m, rounded to } 0.62 \text{ m}$$

The minimum length (L) of the tank is then:

L = 3 x W

= 1.86 m

Therefore, the tank dimensions should be 1.5 m deep, 0.62 m wide and 1.86 m long.

7.4.4 Design hints and tips (interceptor tank design)

A few hints and tips are given herewith for the designer:

- Start with the invert elevation of the house connection to determine the inlet level of the interceptor tank. The slope of the pipe should be 1:60. The closer the tank is to the house, the shallower the inlet level will be. The inlet level dictates the position of the tank, therefore, the shallower the tank, the less excavation is required, reducing the installation cost.

- The outlet elevation of the inspection tank should be well above the junction to the main sewer. Try to maintain a 1:60 slope, in order to prevent backflow into the tank when blockages occur in the main collector sewer.

7.5 Main sewer line (small-bore)

This section provides a flow diagram of the procedures to be followed to initiate and complete a design of collector mains in a small-bore sewer system.

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7.5.1 Step-by-step design procedure (small-bore gravity sewer)

A small-bore sewer system is a method of conveying wastewater from all users to a selected outlet utilizing the pressured gravity flow from the upstream end of the system to the downstream end. **Figure 7.8** presents a flow diagram of the steps required in designing an SBGS system.

Step 1: Determine accumulated peak-flow rates from house connections

Refer to the procedure described under conventional gravity system to determine the accumulated peak-flow rate from house connections.

Step 2: Geometrical layout of pipe network

Unlike conventional gravity sewers which are designed for open-channel flow, small-bore sewers may be installed with sections depressed below the hydraulic gradient line. Thus, flow within a small-bore sewer may alternate between open-channel and pressure flow.

However, the vertical alignment must ensure that an overall fall does exist across the system and the hydraulic gradient line during estimated peak flows does not rise above the outlet invert of any interceptor tank (TAG, 1985).

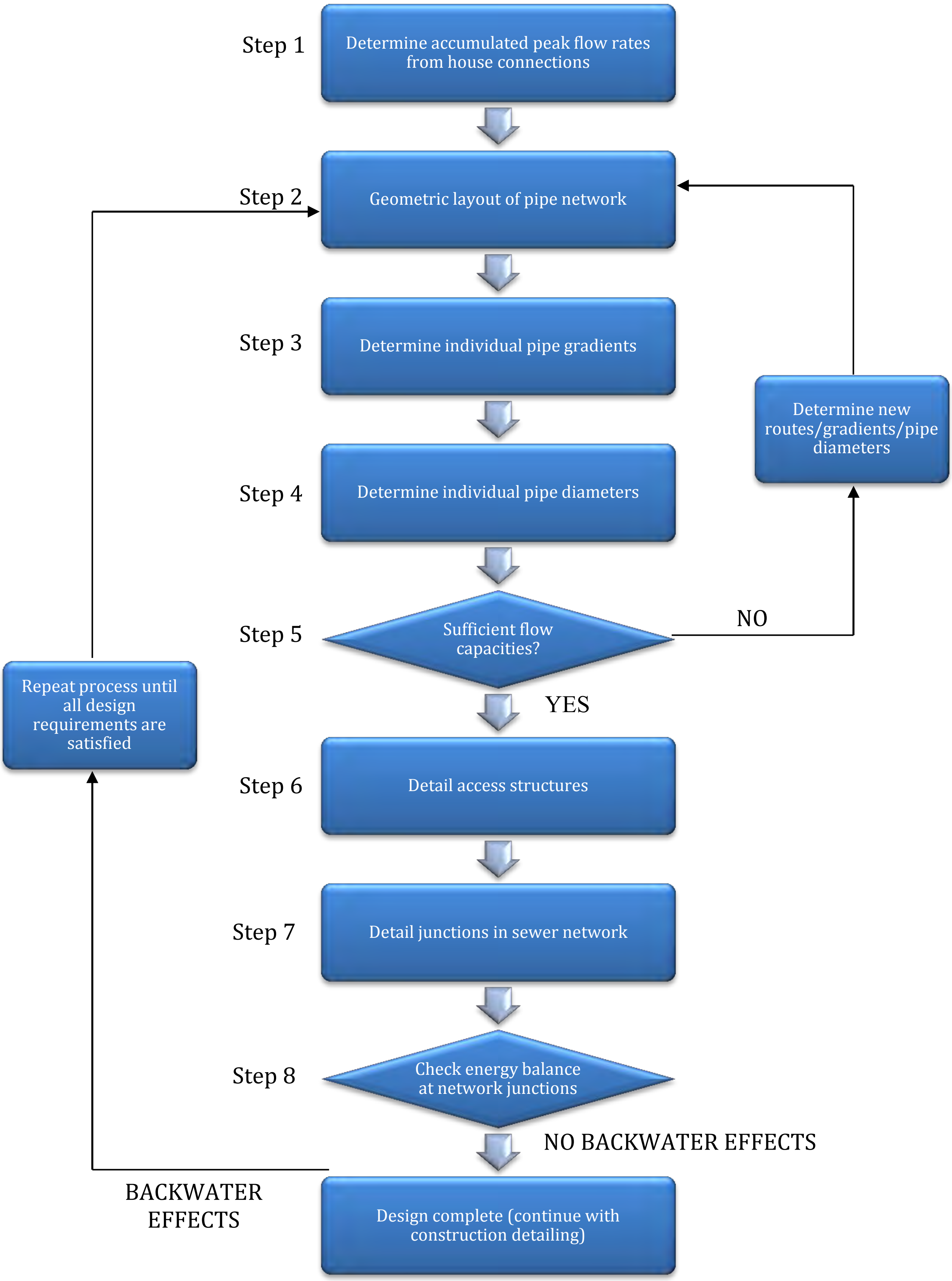

Figure 7.8: Flow diagram – Small-bore main sewer design

High points where the flow changes from pressure flow to open-channel flow and points at the end of long flat sections are critical locations where the maximum elevation must be established above which the sewer pipe cannot rise. Between these points, the sewer may have any profile as long as the hydraulic gradient remains below all interceptor tank outlet inverts and no additional high points are created.

Step 3: Determine individual pipe gradients

Strict sewer gradients to ensure minimum self-cleansing velocities are not necessary, since small-bore sewers are designed to collect only the liquid portion of the wastewater (Little, 2004).

For calculating the hydraulic gradient between two points along the route, refer to the pipe slope calculation procedure as described under the conventional gravity system design steps section.

Step 4: Determine pipe diameter

Pipes with diameter smaller than 100 mm are hydraulically capable of transporting 'greywater' discharge; however, the selection of the minimum permissible size should be based primarily on maintenance considerations and costs. At present, in order to facilitate cleaning of the sewer, a minimum diameter of 100 mm is recommended where specialized equipment for cleaning smaller pipes is not generally available (TAG, 1985).

Step 5: Sufficient flow capacities?

Flow within a small-bore sewer may alternate between open-channel and pressure flow. In making design calculations, separate analyses must be carried out for each sewer section in which the type of flow varies and the slope of the grade line is reasonably uniform. Any of the equations provided in **Table 5.4** can be used in this analysis.

By using computer applications or spreadsheets the design effort for the above process can be less burdensome. Refer to the worked example given in **Paragraph 7.5.2**.

Step 6: Details of access structures

Refer to Step 6 of **Paragraph 5.7.2** for the detailed requirements of access structures.

Step 7: Details of junctions in sewer network

Refer to Step 7 of **Paragraph 5.7.2** for the detailed requirements of junctions.

Step 8: Check energy balance at network junctions

Refer to the procedure described under conventional gravity system to verify the energy balance requirements at each network junction.

7.5.2 Worked Example #2– Small-bore main sewer line design

This design example is a metric version of that supplied by USEPA in their document entitled: *Alternative Wastewater Collection Systems* (USEPA, 1991).

A small-bore sewer line is to be constructed to serve a small new development with no waterborne sewer system. The development is subdivided into erven with 30 m frontage. Sixteen erven are currently occupied with single-family homes. The proposed sewer will serve a total of 25 erven, but an additional 10 erven upstream of the proposed sewer terminus may be subdivided later. Therefore, the small-bore gravity sewer is designed for 35 residential connections.

The first step in design is to draw a system profile, beginning with the ground surface profile. The location and elevation of all interceptor tank outlet inverts should be added. The profile of the sewer is drawn such that it is below all the tank outlet invert elevations and limits depths of excavation (see **Figure 7.9**). Subsequent hydraulic analysis will show whether the proposed profile is satisfactory.

To perform the hydraulic analysis, the sewer is divided into convenient sections. Each section should have a relatively uniform gradient or flow condition (open channel or surcharged) to simplify the computations.

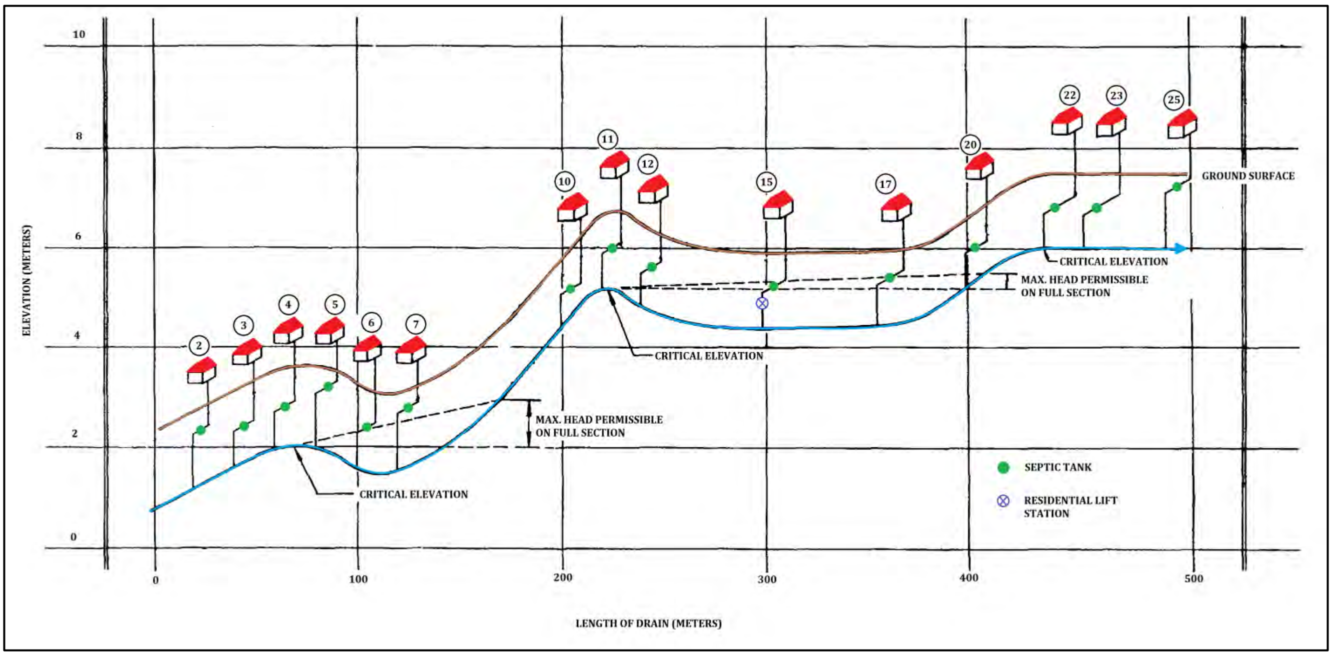

Figure 7.9: Worked Example #2 – Long section of small-bore sewer line

The computations for this example are presented in **Table 7.3**:

Column 1: The selected sections are numbered beginning from the sewer outlet (Station 0 m).

Column 2: Downstream station of the individual section is recorded.

Column 3: Upstream station of the individual section is recorded.

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Column 4: The design flow is based on the total number of connections contributing flow to the section. The estimated flow per connection used in this example is 0.04 ℓ/s.

Column 5: The length of the sewer section is determined by the distance between the downstream and upstream stations (Columns 2 and 3).

Column 6: The proposed elevation of the sewer invert at the upstream station of the section is recorded.

Column 7: The proposed elevation of the sewer invert at the downstream station of the section is recorded.

Column 8: The difference in the upstream and downstream section stations is determined by subtracting Column 6 from Column 7.

Column 9: The average slope of the section is calculated by dividing Column 8 by Column 5. If Column 8 is zero, then surcharged conditions must be assumed.

Column 10: The proposed pipe diameter is recorded.

Column 11: The capacity of the sewer section is calculated. In this example, the Manning equation was used with an '*n*' of 0.013 $s/m^{1/3}$.

Column 12: The ratio of the design flow to the calculated pipe capacity (full pipe flow) is determined by dividing Column 4 by Column 11. If less than 1, the pipe flows partially full. If greater than 1, the pipe is surcharged.

Column 13: The velocity at full pipe flow is determined by dividing the Column 11 by the cross-sectional area of the pipe. This computation is necessary for the subsequent calculations.

Column 14: The depth of flow at design flows is determined by using a hydraulic elements chart from any sewer design or hydraulic handbook (see **Paragraph 5.2.2.4**).

Column 15: The velocity at design flow is determined from the hydraulic elements chart. It is not necessary for design, but is often of interest to the designer.

Those sections that are continuously surcharged (Column 14) and long sections laid level are critical sections in small-bore gravity sewer design. If wastewater backups into individual connections are to be prevented, the slope of the hydraulic grade line during peak-flow conditions should not be allowed to rise above any service connection inverts. If this occurs, the pipe must be increased in diameter, the downstream invert elevation lowered, or STEP units installed at the affected connections.

In this example, Sections 2 and 4 are continuously surcharged. To determine if the pipes in these sections are adequately designed, the maximum slopes of the hydraulic grade lines through the sections are sketched on the profile starting at the outlet from the surcharged sections.

The grade lines must remain below all connection inverts if gravity connections are to be used. Using the maximum slopes, the hydraulic capacities of the sections are calculated. These capacities must be greater than the design flows through the sections. Section 2 is surcharged from Station 0 to 60 m, the outlet where free discharge occurs, to a point where the hydraulic grade line intersects the pipe upstream.

Because all service connections must remain above the hydraulic grade line to allow gravity drainage, the maximum slope of the grade line is established by Connection 6 (**Figure 7.9**). The maximum slope is determined to be 0.005 m/m. A 50 mm diameter pipe would require a slope of 0.026 m/m to carry the design flow. This slope would cause the hydraulic grade line to be above Connection 6, so a 100 mm diameter pipe was selected.

In Section 4, Connections 15 and 17 establish the maximum slope of the hydraulic grade line. The sewer profile originally proposed sets the static water level of this surcharged section at the same elevation as Connection 15. Either a STEP unit must be used at this connection or the sewer invert elevation lowered at Station 220. If a STEP unit is used at Connection 15, Connection 17 will establish the maximum slope of the grade line at approximately 0.002 m/m. A 50 mm diameter pipe is not large enough to carry the design flow. Therefore, a 100 mm diameter pipe is selected. Other options would have been to provide STEP units at both connections or to lower the sewer invert at Station 10. An economic analysis would be necessary to determine the most cost-effective solution.

Table 7.3: Small-bore design calculations

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section nr	**Station (from)**	**Station (to)**	**Design flow (l/s)**	**Length (m)**	**Elevation of upstream station (m)**	**Elevation of downstream station (m)**	**Elevation difference (m)**	**Slope (m/m)**	**Pipe diameter (mm)**	**Flow at full pipe (l/s)**	**Ratio of design to full flow pipe flow**	**Velocity at full flow pipe flow (m/s)**	**Ratio of depth of flow to pipe diameter**	**Velocity at design flow (m/s)**
1	0	60	1.40	60	2.4	0.8	1.6	0.02667	50	1.32	1.06			
									100	0.46	0.17	1.10	0.32	0.66
2	60	143	1.32	83	2.4	2.4	0.0	0.000						
	60	170	1.32	110	2.9[a]	2.4	0.5	0.005[a]						
								0.026[b]	50	1.32				
								0.001[b]	100	1.32	1.00	0.17	Surcharge	0.17
3	143	220	1.16	77	5.2	2.4	2.8	0.036	50	1.55	0.75	0.78	0.73	0.76
4	220	400	1.00	180	5.2	5.2	0.0	0						
	220	407	1.00	187	5.5	5.2	0.3	0.002[a]						
								0.015[b]	50	1.00				
								0.0004[b]	100	1.00	1.00	0.13	Surcharge	0.13
4[c]	220	400	1.00	180	5.2	2.5[d]	2.7[d]	0.015	50	1.00	0.50	1.00	1.00	0.50
1[c]	143	220	1.16	77	2.5	2.4	0.1	0.001	50	0.29	3.90			
									100	1.85	0.63	0.23	0.38	0.16
5	400	433	0.64	33	6.0	5.2	0.8	0.024	50	1.27	0.51	0.63	0.57	0.54
6	433	473	0.56	40	6.0	6.0	0.0	0						
								0.005[b]	50	0.56	1.00	0.28	Surcharge	0.28

a - Maximum rise or slope at the HGL, based on upstream condition.
b - Slope of HGL necessary to carry the design flow.
c - Recomputation of pipe hydraulics due to change in sewer profile.
d - Necessary elevation and elevation difference to carry the design flow.

7.5.3 Design hints and tips (small-bore sewer design)

A few hints and tips are given herewith for the designer:

- First design the house connections (refer previous section). This will give the minimum invert depths the pipe has to connect to.
- In the main sewer line, begin the analyses by starting at the bottom junction first, and then work upstream connecting to all the house connection junctions

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8. DESIGN GUIDELINE FOR SIMPLIFIED SEWERAGE

Heavy reliance on high-cost conventional sewers has produced inadequate sanitation service coverage in many urban areas. In the recent past, low-cost, on-site systems have been gaining increased acceptance as alternatives; however, in areas where housing densities and levels of water consumption are high, waterborne solutions are required.

A modified approach to sewer design based on hydraulic theory, satisfactory experience elsewhere, and redefinition of acceptable risk has been developed. Systems designed according to these new criteria are known as 'simplified sewers'. Bakalian et al. (1994) indicated that they operate as conventional sewers but with a number of modifications such as:

- The minimum diameter is reduced
- The minimum cover is reduced
- The slope is determined by using the tractive force concept rather than the minimum velocity concept
- Sewers are installed under sidewalks where possible
- Many costly manholes are eliminated or replaced with less-expensive cleanouts

These systems have shown that there are significant cost savings when compared to conventional gravity sewers. Operation and maintenance requirements are similar to those of conventional sewers.

To be able to successfully implement a simplified sewer system in a community a model for implementation is required. In a report by the Palmer Development Group (Pegram and Palmer, 1999), a model for implementation based on a successful international model was adopted and applied to a South African project. The components include:

- Institutional and community arrangements
- Cadastral and social characterization
- Health and hygiene education
- Design, task planning and agreements
- Works implementation
- System consolidation
- Evaluation
- Maintenance (social and physical)

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During the implementation of the simplified sewer system, community organisation and participation are obtained in the design, implementation, operation and maintenance of the system. The community come together to own and manage the system; in other words, the condominial pipeline is privately and collectively owned by the community themselves.

8.1 Design criteria

8.1.1 Layout

To avoid deep excavations, long trunk pipes to interceptors, and large pumping stations, serious consideration is given to splitting the network into two or more separate smaller systems; although network layout is also an important part of conventional design, the optimization of pipe lengths and network subdivisions takes on even greater importance in the simplified system.

Where feasible, a project area is defined by individual drainage basins, each with its own collectors and treatment plant. As needs and resources increase, mini-networks can be connected to a common interceptor for conveyance to a regional plant or local treatment system.

Furthermore, to minimize excavation and the cost of pavement restoration, sewers are, to the extent possible, located away from traffic loads, generally under the sidewalks (on both sides of the street, if necessary) rather than down the centre of the street. To save on pipe and excavation costs, sewers extend only to the last connection rather than to the end of the block. **Figure 8.1** illustrates the way in which the sewer can be used. The designer needs to select a layout most suitable for the local situation.

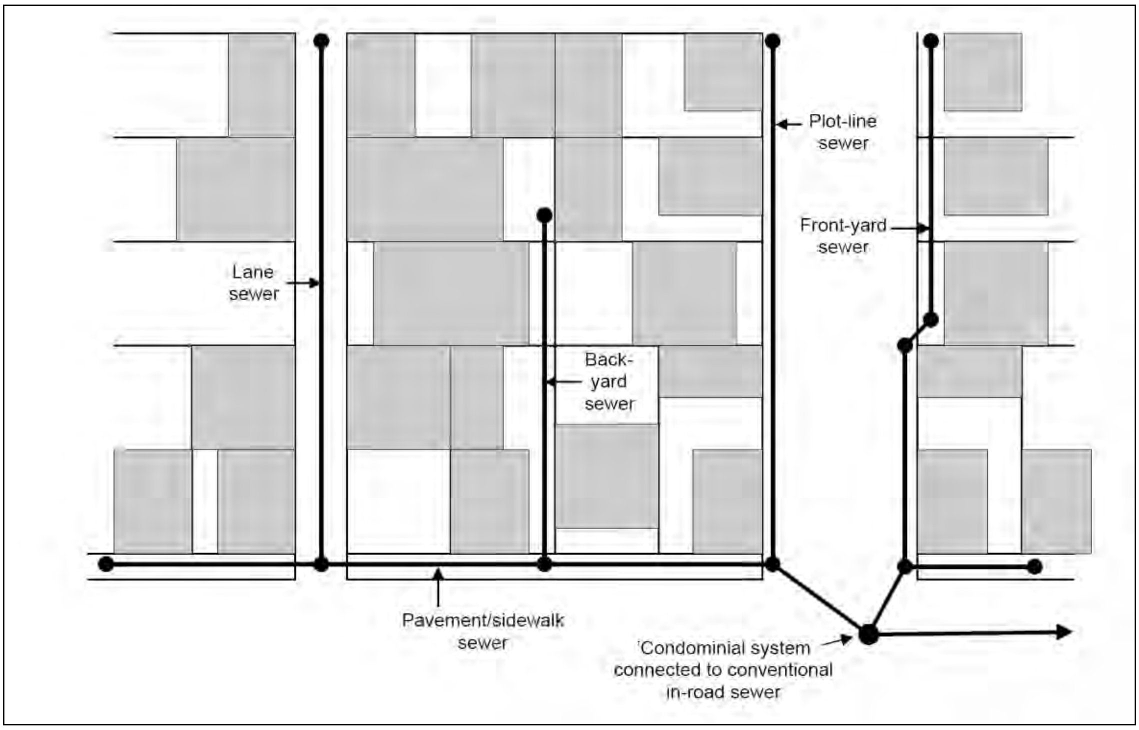

Figure 8.1: Alternative routes for simplified sewers (Mara, Sleigh and Tayler, 2000)

8.1.2 Hydraulics

8.1.2.1 Design period

In conventional design, it is common to design trunk sewers and interceptors for the projected peak flow expected during a 25 to 50 year period or for the saturation population of the area. Such long design periods make it possible to capture economies of scale in sewerage systems. However, these have to be balanced against the opportunity cost of capital, uncertainties in predicting future land-use patterns or directions of growth in developing-country cities, and the high cost of maintaining large sewers with low flows. If shorter design periods are used it avoids such problems and reduces the large capital requirements in sewerage systems, facilitates financing, and enhances prospects of achieving greater coverage with a given investment. With shorter design periods and construction in phases, starting from upstream ends, the effects of errors in forecasting population growth and their water consumption can be minimized and corrected. For these reasons, simplified sewerage employs design periods of 20 years or less. In this regard, it is noteworthy that the USEPA limits the design period to 10 to 15 years (ASCE, 1982).

8.1.2.2 Design flow

Wastewater flow quantities are necessarily lower than the quantity of water supplied because water is lost through leakage, garden watering, house cleaning, etc. To determine the expected amount of wastewater, it is important to keep records of pumping for each day and of fluctuations during the day.

Reliance on estimates of water use from industrialized countries or cities of similar characteristics can lead to erroneous design flows. Information should be obtained from the area under consideration. In arid areas of the United States, for example, the return coefficient is as little as 0.4; in Sao Paulo (Brazil), this coefficient is 0.8. The design flow is based on this returned quantity multiplied by a peak factor, which is inversely related to population size.

In industrialized countries, the peak factor is conservatively estimated to be between 2.0 and 3.3. In Brazil and Colombia, a peak factor of 1.8 has been used in simplified sewerage projects. Where water-use information is not available, the simplified sewerage system is designed for a minimum flow of 1.5 ℓ/s and infiltration is assumed to be 0.05 ℓ/s to 1.0 ℓ/s per km of pipe.

In South Africa the peak sewage flows can be calculated utilizing the information as provided in **Section 5** for conventional gravity sewers. Assuming an infiltration rate of 15% and a peak factor of 2.5, and that approximately 80% of the household water consumption will be return flow.

According to Mara et al. (2000) the value of the wastewater flow used for sewer design is the daily peak flow and can be estimated as follows:

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$$Q = \frac{k_1 k_2 Pw}{(60)(60)(24)} \qquad ...(8.1)$$

where:

Q	=	daily peak flow (ℓ/s)
k_1	=	peak factor (daily peak flow divided by average daily flow)
k_2	=	return factor (wastewater flow divided by water consumption)
P	=	population served by length of sewer under consideration
W	=	average water consumption (ℓ/cap·d)
		and (60)(60)(24) is the number of seconds in a day

A suitable design value for k_1 for simplified sewerage is 1.8 and k_2 may be taken as 0.85 resulting in Equation (8.2):

$$Q = 1{,}771 x 10^{-5} Pw \qquad ...(8.2)$$

In simplified sewer design Equation (8.2) is used to calculate the daily peak flow in the length of sewer under consideration, but this is subject to a minimum value of 1.5 ℓ/s. According to Mara et al. (2000) this minimum flow is not justifiable in theory but, as it is approximately equal to the peak flow resulting from flushing a water closet, it gives sensible results in practice.

8.1.2.3 Flow in circular sections

The Manning equation (Equation (8.3)) is usually used to determine the flow in open-channel/free surface type flow.

$$Q = \frac{1}{n}\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}} S^{\frac{1}{2}} \qquad ...(8.3)$$

where:

Q	=	flow rate (m^3/s)
n	=	coefficient of roughness ($s/m^{1/3}$)
A	=	flow area (m^2)
P	=	wetted perimeter (m)
S	=	slope of the energy grade line (m/m)

Paragraph 9.4 provides details of the properties of circular sections for determining flow in partially full flowing pipes. Details are provided of obtaining the coefficients k_a and k_r represented by Equations (8.4) and (8.5).

$$k_a = \frac{1}{8}(\theta - \sin\theta) \qquad ...(8.4)$$

$$k_r = \frac{1}{4}\left(1 - \frac{\sin\theta}{\theta}\right) \qquad ...(8.5)$$

where:

θ = angle of flow (radians)

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$$a = k_a D^2 \quad \text{...(8.6)}$$

$$r = k_r D \quad \text{...(8.7)}$$

where:

a = area of partially full flowing pipe (m^2)
r = hydraulic radius of partially full flowing pipe (m)

Substituting Equations (8.6) and (8.7) into the Manning equation results in Equation (8.8) for partially full flow:

$$Q = \frac{1}{n} k_a D^2 (k_r D)^{\frac{2}{3}} S^{\frac{1}{2}} \quad \text{...(8.8)}$$

The usual design value of the Manning roughness coefficient is provided in **Table 5.5**. The roughness of the bacterial slime layer which grows on the sewer wall must, however, not be forgotten.

8.1.2.4 Ensuring self-cleansing

Instead of the minimum velocity criterion of 0.6 m/s to 0.7 m/s as in conventional gravity sewer design, simplified sewer design is based on maintaining a boundary shear stress of 0.1 kg/m^2, which is sufficient to re-suspend a 1 mm particle of sand. As described by Bakalian et al. (1994) many authors have proposed the use of critical shear stress for determining the minimum slope of sewers as an economical alternative to the minimum velocity approach. For a minimum shear stress of 0.1 kg/m^2, the minimum sewer gradient for pipes smaller than 1 050 mm can be made flatter than when designed according to the minimum-velocity approach, and the minimum sewer gradient for pipes larger than 1 050 mm should be made steeper to maintain self-cleansing. According to Bakalian et al. (1994) in Brazil, for design of simplified sewers, the following equation (Equation (8.9)) is used:

$$S_{min} = 0{,}0055 Q_i^{-0{,}47} \quad \text{...(8.9)}$$

where:

S_{min} = minimum slope of the sewer (m/m)
Q_i = initial flow (ℓ/s)

For derivation and use of this equation, refer to **Paragraph 9.4** to compare the advantages of this method over the conventional minimum velocity method. This indicates that the minimum sewer gradient is independent of the pipe diameter for any given flow rate.

8.1.2.5 Minimum diameter

A minimum diameter for sanitary sewers is usually specified in order to avoid clogging by large objects.

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In conventional systems in the United States, the house connections are usually 150 mm in diameter, but smaller sizes have been used. Therefore, for conventional sewerage, the minimum diameter commonly specified for street sewers in many countries is 200 mm. In the simplified system, smaller sizes are recommended because, in the upper reaches of a system where flow is low, the use of smaller-diameter sewers results in greater depths of flow and higher velocities, which improves cleansing.

The sewer diameter is determined by checking the following conditions:

- Calculate using Equation (8.2), the initial and final wastewater flows (Q_i and Q_f), which are the flows occurring at the start and end of the design period. If the calculated flow is less than the minimum peak daily flow of 1.5 ℓ/s then use a value of 1.5 ℓ/s for Q_i in the next calculation below to determine S_{min}.

- Calculate S_{min} from Equation (8.9).

- Calculate D from Equation (8.10) using $Q = Q_f$ (in m^3/s), again subject to a minimum value of 0.0015 m^3/s, for $d/D = 0.8$ (i.e. for $k_a = 0.6736$ and $k_r = 0.3042$ from Equations (8.6) and (8.7).

  Equation (8.8) can be rearranged to obtain:

$$D = n^{\frac{3}{8}} k_a^{-\frac{3}{8}} k_r^{-\frac{1}{4}} \left(\frac{Q}{S_{min}^{\frac{1}{2}}} \right)^{\frac{3}{8}} \qquad ...(8.10)$$

In this design procedure, the value of Q_i is used to determine S_{min} and the value of Q_f is used to determine D.

The calculated diameter is unlikely to be a commercially available size, and therefore the next larger diameter that is available is chosen. However, shallow sewers use as a minimum 100 mm piping, only increasing when estimated peak sewage flow dictates.

It is useful to know the maximum number of houses that can be served by a sewer of any given diameter. Calculations indicate that 234 houses can be served for a typical household size of 5 persons, with a consumption of 100 ℓ/cap·d, a peak factor of 1.8, a return factor of 0.85 and assuming an initial d/D of 0.6.

8.1.2.6 Minimum and maximum flow velocities

The minimum self-cleansing velocity in shallow sewers is 0.5 m/s (compared with 0.7 m/s in conventional gravity sewer design systems), if the Bakalian et al. (1994) approach as described in **Paragraph 8.1.2.4** is not followed.

The pressure force of the water backing up behind the solids in the smaller pipes flushes the solids down the system and this is the main reason for the relaxation of the minimum self-cleansing velocity criterion. A maximum flow velocity of 4.0 m/s has been set although the effect of these high velocities, for short periods, on the sewer pipes itself is insignificant.

8.1.2.7 Maximum and minimum flow depths

Based on accepted practice, a minimum flow depth of 20% of the pipe diameter (ensuring that there is sufficient velocity of flow to prevent solids deposition in the initial part of the design period) and a maximum of 80% of the pipe diameter (to provide for sufficient ventilation and surplus capacity at the end of the design period) are suggested.

8.1.3 Service connection

According to Bakalian et al. (1994) in the simplified design, a 600 mm square or circular connection (or inspection) box is placed between the residential building and the service line (**Figure 8.2**). All sewers or drains from the house or building convey the wastewater to this connection box which is usually located under the sidewalk in the public right-of-way. This connection box could also be substituted with a cleanout point.

Figure 8.2: Construction of inspection chamber (Eslick and Harrison, 2004a)

In certain areas of Sao Paulo, where the risk of obstruction is believed to be high (e.g., in commercial establishments), baffled boxes have been added downstream of each building sewer, in addition to the connection or inspection box.

Baffled boxes are usually 0.6 m x 0.6 m x 0.8 m concrete boxes with an underflow baffle located approximately 0.6 m from the inlet (Bakalian et al., 1994). Their function is to prevent trash and other large settleable solids from entering the sewer. Their maintenance is usually the responsibility of the homeowner.

8.1.4 Depth of sewers

At the starting point of laterals the minimum depth at which pipes are laid should be sufficient to allow house connections and have a layer of soil over the crown to protect the pipe against structural damage from external loads.

The minimum cover over a sewer is required for three reasons:

- To provide protection against imposed loads, particularly vehicle loads
- To allow an adequate fall on house connections
- To reduce the possibility of cross-contamination of water mains by making sure that, wherever possible, sewers are located below water mains

In conventional design, there is no one method for determining the minimum depth of sewers as long it satisfies the above criteria. However, some rules of thumb suggest that the top of the sanitary sewer should not be less than 1 m below the basement, and where there is no basement the invert of the sanitary sewer should not be less than 1.8 m below the top of the foundation.

In the simplified system, typical minimum sewer depths are much shallower:

- 0.4 m in properties
- 0.65 m below sidewalks
- 0.95 m to 1.50 m below residential streets
- 2.5 m below streets with heavy traffic

Building elevations are not considered in setting the invert elevation of the sewers. If buildings along the mains are too low for connections by gravity, it is the responsibility of the property owner to find other means of making a connection. In cases where topography permits, it may be necessary to use a longer building sewer to enable connection to a service line, provided easements can be obtained from the neighbouring owners.

8.1.5 Manholes

Manholes are an expensive component. They are the most familiar features of a sewer system and the criteria for manhole use have gradually become more conservative and have contributed significantly to the high cost of sewerage systems.

Manholes are typically placed at:

- The upper ends of laterals
- Changes in direction and slope

- Pipe junctions
- Intervals typically 100 m apart

The simplified sewerage system aims to reduce the number of costly manhole appurtenances required. The following is recommended to achieve this:

- Conventional standard manholes are replaced with ‘simplified manholes’, cleanouts and inspection boxes where possible. A ‘simplified manhole’ is similar to a conventional manhole but is smaller. The ‘simplified manhole’ does not usually permit man entry since the sewers are placed at a shallower depth and thus this is not required.
- Manholes at changes in direction or slope can be replaced with simple underground boxes or chambers
- House connections are also tailored to serve as inspection locations. A box is constructed connecting to the sewer with a wye piece (which can be used for cleaning using a cleaning rod)

There are, however, situations where manholes should not be eliminated, such as:

- Very deep sewers (> 3.0 m)
- Lesser slopes (less than minimum required)
- Sewers with drops
- Where there is a commercial or industrial establishment connection point

8.1.6 Material

The types of material (pipe, manholes, etc.) are similar to those used for the construction of a conventional gravity sewer. The general description for specifying the pipes is simply a pipe suitable for the conveyance of sewage, under the particular working and installation conditions to which they will be subjected in accordance with Sections 3.1 and 3.2 of SANS 1200 LD:1982 (SABS, 1982a). Each type of sewer pipe, its advantages and limitations, should be evaluated carefully in the selection of the pipe material for any given application.

8.2 Simplified sewerage design

8.2.1 Step-by-step design procedure (simplified sewerage system)

A simplified sewer system design procedure is depicted in a flow diagram shown in **Figure 8.3**.

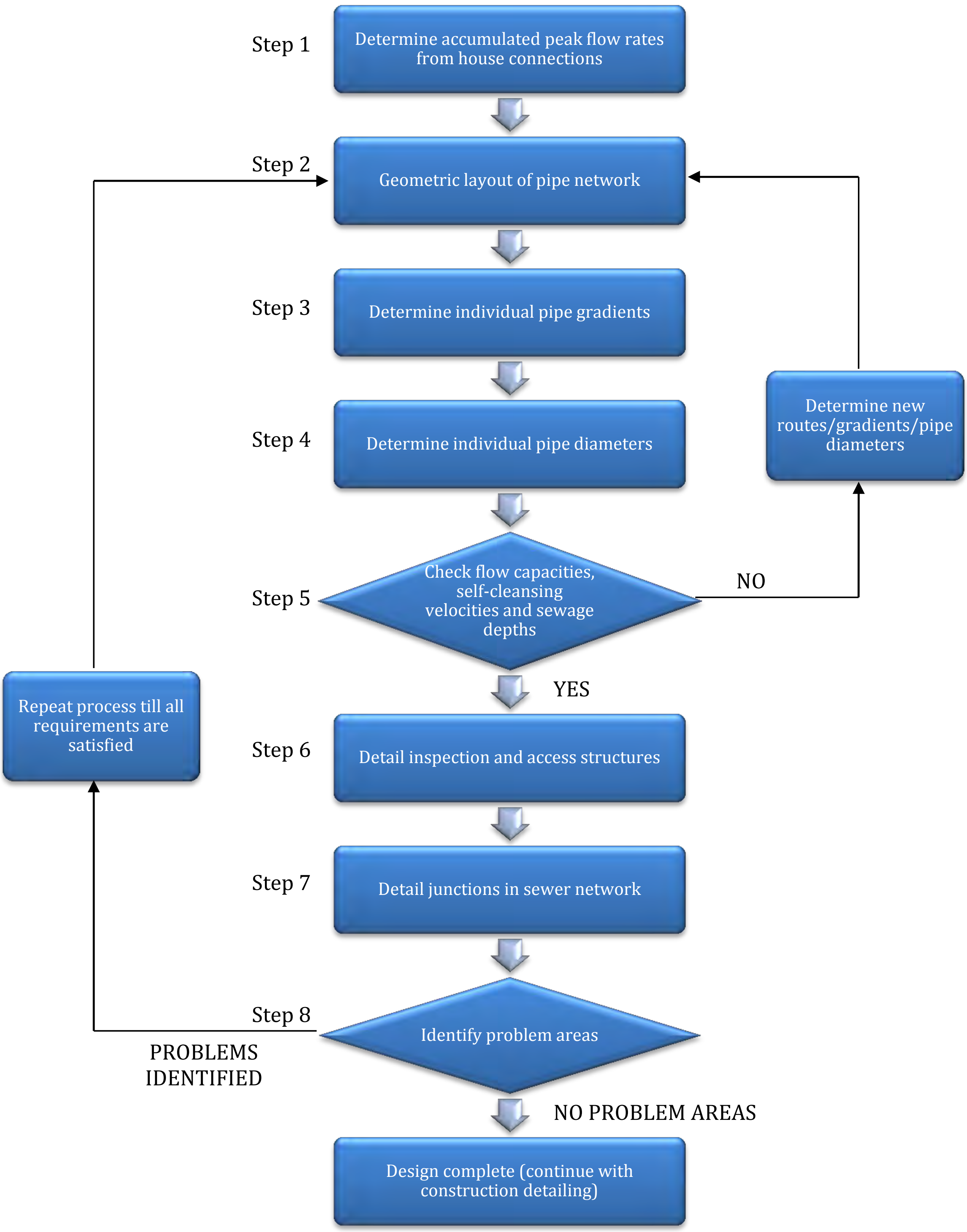

Figure 8.3: Flow diagram – Simplified sewerage system design

The procedure to design a simplified sewerage system is similar to that performed for a conventional sewerage system.

8.2.2 Worked Example #1 – Simplified sewerage design

This design example is an adaptation from an example described in *PC-Based Simplified Sewer Design* by Mara et al. (2000).

This section details the steps necessary to prepare design information for a condominial sewer system. It uses the example of a module forming part of a new sites-and-services housing scheme. **Figure 8.4** shows this module, together with a sewer layout to serve it. Plot boundaries are represented by thin lines and sewers by thick lines.

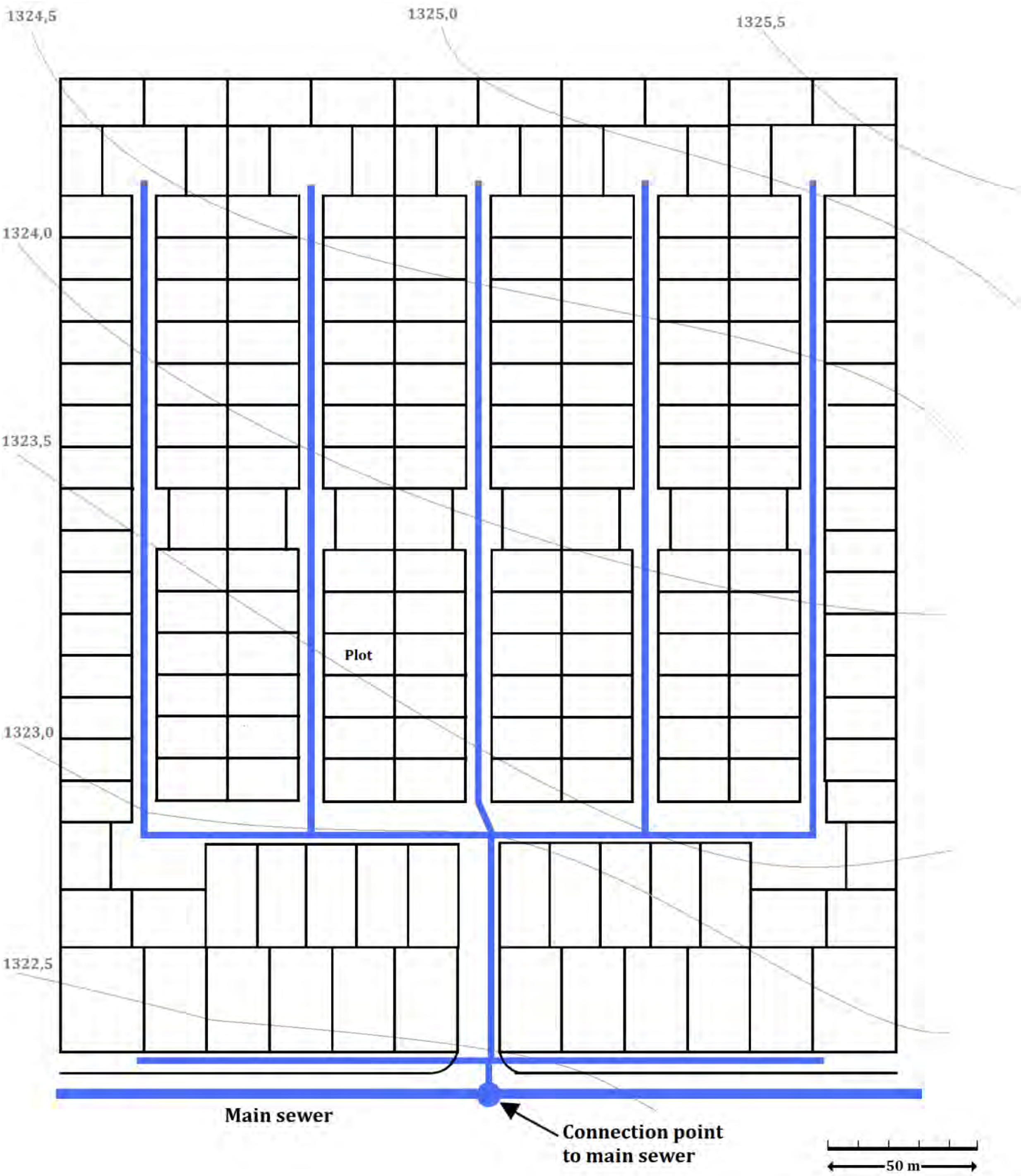

Figure 8.4: Sewer layout for a typical sites-and-services housing module

The plot sizes are small, representing typical practice in a new sites-and-services scheme. The five cul-de-sacs are relatively narrow lanes that are not intended for vehicular traffic (the width of the right-of-way scales about 7.5 m on the drawing but it can be assumed that the actual right-of-way is somewhat narrower). Sewers are proposed along the centres of these pedestrian lanes. Elsewhere inside the module, sewers are alongside the sides of streets, as close as possible to the front plot lines. The housing module fronts onto a main street, along which runs a public collector sewer. The larger plots that face onto the main street are connected to a local sewer that runs under the pavement, rather than directly to the collector sewer.

All the sewers serving the housing module thus form a condominial system that is self-contained and can be analysed and designed regardless of the arrangements that are made elsewhere.

Similar arrangements, but including back-yard and/or front-yard sewers, could be adopted for a scheme with considerably larger plot sizes.

This is, of course, a very regular layout. In practice, many layouts will be less regular with some interconnections between different housing areas so that the limits of each 'condominium' may be more difficult to define. Nevertheless, the basic approach described here is valid for these more complex situations.

The first step in the design process is to represent the system as a series of sewer 'legs' running between junctions or 'nodes'. In theory every house connection could be a node, but this would require a large number of calculations. Fortunately such a detailed approach is not necessary since the change in flow at each house connection will be infinitesimally small. Rather, the need is to develop a 'model' of the system that reduces the amount of calculation effort required, while retaining sufficient accuracy to ensure that the sewers are correctly sized. **Figure 8.5** illustrates this process of simplification for part of the layout shown in **Figure 8.4**. Three nodes have been assumed on the sewer that runs along one of the five pedestrian 'lanes'.

Inspection suggests that the four plots (colour-coded yellow) at the head of the lane will drain to a chamber at Node J3. Fourteen plots (coloured green) will discharge to sewer Leg C1-3 and a further two plots (coloured purple) can be connected directly at Node J4. Twelve plots (coloured pink) will discharge to sewer Leg C1-4. For calculation purposes, the number of connections to any sewer leg can be taken as the connections at the upstream node plus those along the length of the sewer leg itself. Thus, the number of connections to sewer Legs C1-3 and Legs C1-4 will be 18 (4+14) and 14 (2+12), respectively.

This process should be repeated for the entire system. The results are shown in **Figure 8.6**.

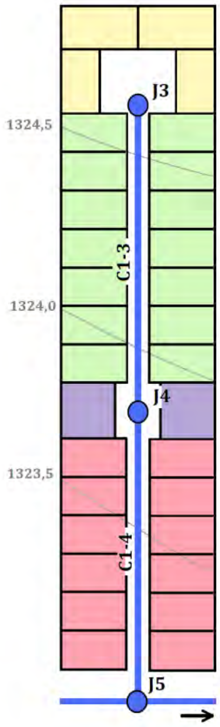

Figure 8.5: Sewer divided into legs running between nodes

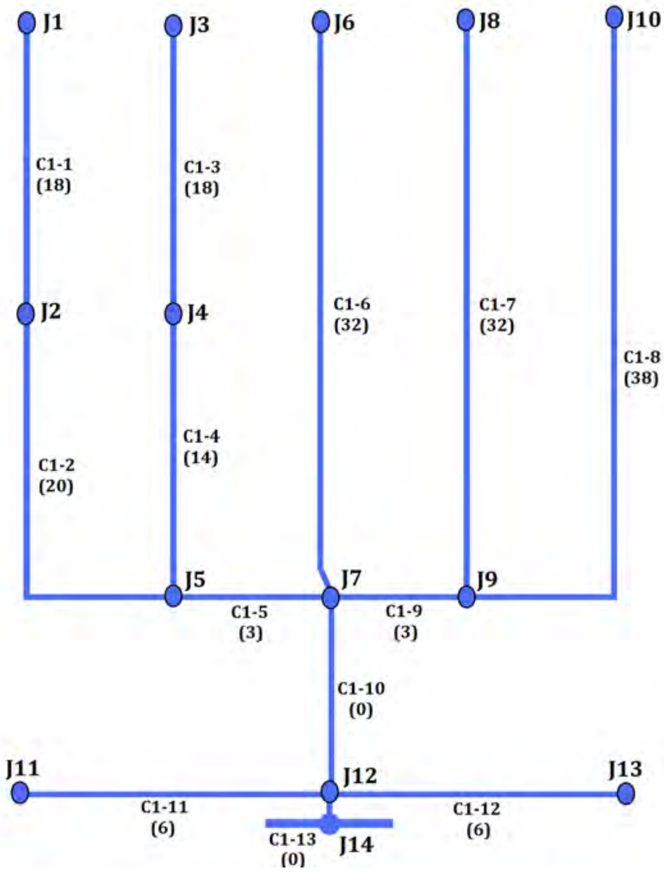

Figure 8.6: Numbering system for sewer legs and nodes

Interpretation of the results of the analyses will be easier if there is some logic to the numbering system. The nodes and sewer legs have been numbered starting from the head of the left-hand sewer. The numbering system used for the sewers indicates that a condominial system, rather than public collector sewers, is being designed. The figures given in brackets beneath the sewer leg numbers in **Figure 8.6** are the number of house connections along those legs of the sewer.

Note that the two lane sewers on the left of **Figure 8.6** have intermediate nodes, which are omitted from the other three nodes. This has been done in order to test the sensitivity of the model to the number of nodes assumed. In practice, the intermediate nodes are not really required if the average ground slope along the sewers is fairly constant. Additional nodes should be inserted where there is a significant change in ground gradient since the sewer slope will have to be changed at this point, and this needs to be reflected in the calculations. This provides sufficient information regarding the layout of the sewer system. Additional information on the sewers themselves is required as follows:

- The lengths of all sewers – obtained by scaling off from the layout drawing
- The ground level at each node – this is available from the physical survey of the area (contours shown on **Figure 8.4**)
- The minimum allowable cover for different situations – see **Paragraph 8.1.4**. The minimum sewer depth for Junctions J1, J2, J3, J4, J6, J8 and J10 was assumed as 0.4 m whilst Junctions J5, J7, J9, J11, J12 and J13 required a minimum depth of 0.65 m.

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The normal procedure will then be to start at the head of the system, in the case illustrated in **Figure 8.4** at J1 or J10, and set the sewer invert at that point such that the cover is the minimum allowable for the particular situation.

As the design proceeds, it will be found that the slope of many sewers near the head of the system will be governed by the minimum wastewater flow (1.5 ℓ/s), while their diameter is governed by the minimum permissible sewer diameter (100 mm). The number of houses that can be connected to a standard minimum-diameter sewer laid at the minimum gradient based on the minimum peak wastewater flow can be calculated (see **Paragraph 8.1.2.5**). Once this has been done, these minimum parameters can be assumed for any sewer leg that receives flow from a smaller number of houses than the number calculated for the minimum diameter and gradient. This reduces the design task considerably since many smaller condominial systems will consist of only minimum-diameter sewers laid at the minimum gradient based on the minimum peak wastewater flow.

Table 8.1 shows the hydraulic calculations performed on the system shown in **Figure 8.4**.

The criteria that need to be checked include:

- Is the sewer depth greater than or equal to the minimum sewer depth (Column 9 and Column 10 at each junction)?
- Is the gradient for each pipe section equal to or greater than the minimum gradient (Column 14)? Pipes placed at minimum gradient will then have sufficient self-cleansing velocity (Column 17).
- Is the maximum capacity of the pipe section (Column 16) greater than or equal to the calculated design flow (Column 6)?

The systematic adjustment of sewer depths is required in order to meet the above criteria.

Table 8.1: Hydraulic calculations for simplified sewer design example

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Sewer reference	Length (m)	Number of houses served	Estimated flow from houses served (l/s)[a]	Estimated accumulated flow (l/s)	Design flow (l/s)[b]	Ground level (m)		Depth of sewer (m)		Invert level (m)		Difference in invert level (m)	Gradient (m/m)	Diameter (mm)[c]	Flow at full section (l/s)[d]	Velocity of flow (m/s)[e]
						Up-stream	Down-stream	Up-stream	Down-stream	Up-stream	Down-stream					
C1-1	112	18	0.30	0.30	1.50	1324.45	1323.55	0.40	0.40	1324.05	1323.15	0.90	0.00804	100	4.63	0.59
C1-2	164	20	0.33	0.63	1.50	1323.55	1322.95	0.40	0.79	1323.15	1322.16	0.99	0.00604	100	4.01	0.51
C1-3	112	18	0.30	0.30	1.50	1324.65	1323.80	0.40	0.40	1324.25	1323.40	0.85	0.00759	100	4.50	0.57
C1-4	108	14	0.23	0.53	1.50	1323.80	1322.95	0.40	0.79	1323.40	1322.16	1.24	0.01148	100	5.53	0.70
C1-5	60	3	0.05	1.16	1.50	1322.95	1323.00	0.79	1.20	1322.16	1321.80	0.36	0.00600	100	4.00	0.51
C1-6	221	32	0.53	0.53	1.50	1324.75	1323.00	0.40	1.20	1324.35	1321.80	2.55	0.01154	100	5.55	0.71
C1-7	220	32	0.53	0.53	1.50	1324.90	1323.50	0.40	0.65	1324.50	1322.85	1.65	0.00750	100	4.47	0.57
C1-8	276	38	0.63	0.63	1.50	1325.05	1323.50	0.40	0.65	1324.65	1322.85	1.80	0.00652	100	4.17	0.53
C1-9	51	3	0.05	1.16	1.50	1323.50	1323.00	0.65	1.20	1322.85	1321.80	1.05	0.02059	100	7.41	0.94
C1-10	76	0	0.00	2.86	2.86	1323.00	1322.45	1.20	1.46	1321.80	1320.99	0.81	0.01066	100	5.33	0.68
C1-11	118	6	0.10	0.10	1.50	1322.35	1322.45	0.65	1.46	1321.70	1320.99	0.71	0.00602	100	4.01	0.51
C1-12	110	6	0.10	0.10	1.50	1322.80	1322.45	0.65	1.46	1322.15	1320.99	1.16	0.01055	100	5.30	0.68
C1-13	12	0	0.00	3.06	3.06	1322.45	1322.40	1.46	1.49	1320.99	1320.91	0.08	0.00667	100	4.22	0.54

Notes:

(a) Estimated flow is based on a low-income house with in-house water reticulation 500 ℓ/dwelling (**Table 5.1**), infiltration rate of 15% and a peak factor of 2.5

(b) Design example used $Q_{\min}$ = 1.5 ℓ/s for flow rate in each pipe section

(c) Sewer diameters given are those rounded up to next available diameter

(d) Assumed Manning roughness value n = 0.013 s/m$^{1/3}$ (see **Table 5.5**)

(e) Minimum self-cleansing velocity of 0.5 m/s

(f) Minimum gradient = 0.006 m/m

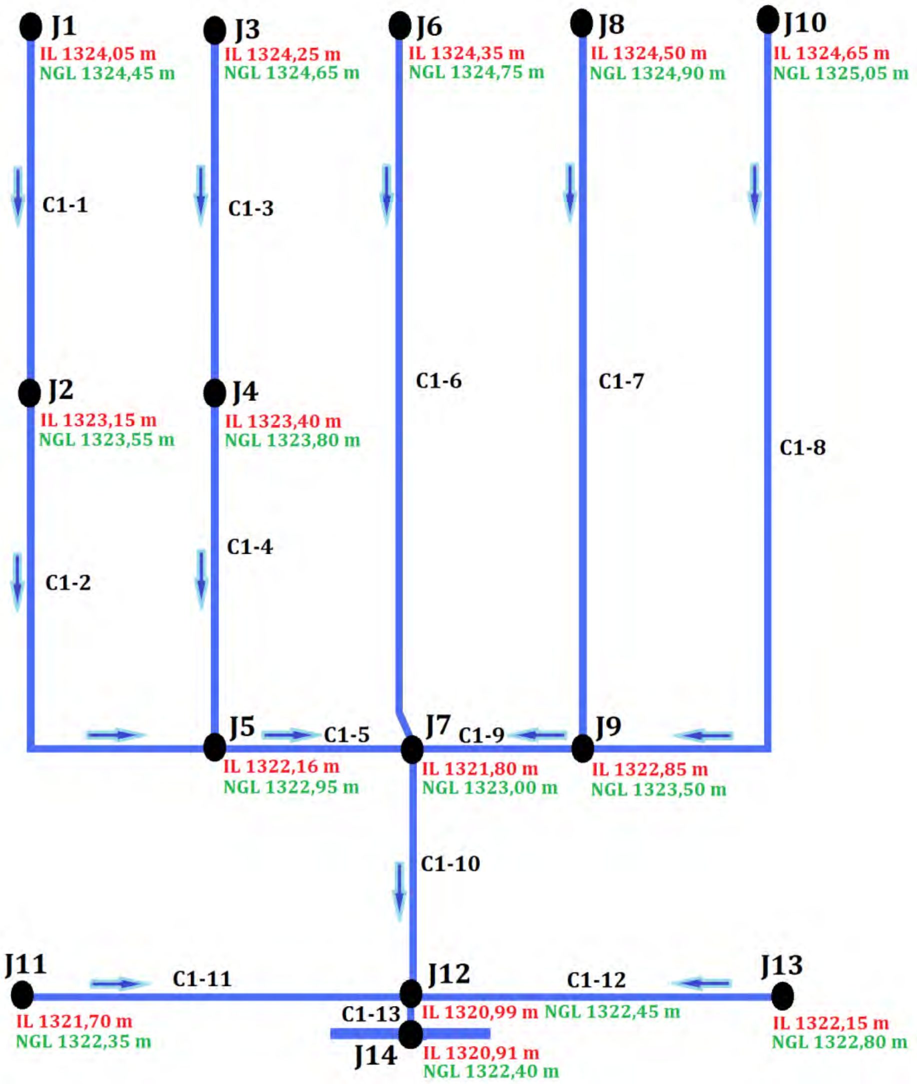

Figure 8.7: Sewer layout for design example

8.2.3 Design hints and tips (simplified sewerage design)

- Sewer gradient and ground slope

The slope of the ground surface (S_g) may be (a) less than, (b) equal to, (c) greater than, or (d) much greater than, the minimum sewer gradient (S_{min}) calculated from Equation (8.9). Furthermore, the depth to the invert of the upstream end of the length of sewer under consideration may be (a) equal to, or (b) greater than, the minimum depth permitted (h_{min}), which is given by:

$$h_{min} = C + D \qquad ...(8.11)$$

where:

h_{min}	=	minimum depth permitted (m), see **Figure 8.8**
C	=	minimum required cover (m)
D	=	sewer diameter (m)

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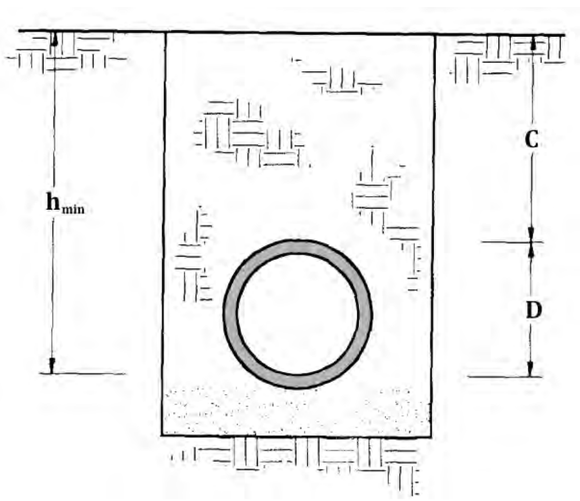

Figure 8.8: Minimum depth definition sketch

As described by Mara et al. (2000), there are six combinations of sewer gradient and ground slope that are likely to be encountered in practice, see **Table 8.2**.

The minimum depth (h_{min}) to which a sewer is laid is the sum of the minimum depth of cover (C) and the sewer diameter (D).

Table 8.2: Combinations of sewer gradient and ground slope

Case	
①	$S_g < S_{min}$ and the invert depth of the upstream end of the sewer $h_1 \geq h_{min}$: choose $S = S_{min}$ and calculate the invert depth of the downstream end of the sewer h_2 as: $h_2 = h_1 + (S_{min} - S_g)\ L$
②	$S_g = S_{min}$ and $h_1 \geq h_{min}$: choose $S = S_{min}$ and $h_2 = h_1$
③	$S_g > S_{min}$ and $h_1 = h_{min}$: choose $S = S_g$ and $h_2 = h_1$
④	$S_g > S_{min}$ and $h_1 > h_{min}$: choose $h_2 = h_{min}$ and calculate the sewer gradient from: $S = S_g + (h_{min} - h_1)/L$ subject to $S \nless S_{min}$
⑤	$S_g > S_{min}$ and $h_1 > h_{min}$: as Case 4, but an alternative solution is to choose $S = S_{min}$ and calculate h_2 from: $h_2 = h_1 + (S_{min} - S_g)\ L$. The choice between these alternative solutions is made on the basis of minimum excavation.
⑥	$S_g >> S_{min}$ and $h_1 \geq h_{min}$: here, it is usually sensible to divide L into two or more sub-stretches with $h_2 = h_{min}$ and $S << S_g$ (but obviously $\geq S_{min}$) in order to minimize excavation. A drop manhole is placed at the sub-stretch junction.

- Minimum diameter

It is useful to know the maximum number of houses that can be served by a sewer of given diameter. This simplifies the calculation process considerably, see **Paragraph 8.1.2.5**.

9. HYDRAULICS OF SEWERS

The main function of a sewer system is to convey wastewater at various rates of flow from the connection point to the wastewater treatment plant. There is seldom control over the content of wastewater that must be conveyed to a treatment plant. The wastewater can contain dissolved solids as well as suspended solids that either settle or float. Most of the dissolved solids and the floating material are transported with the flow. The suspended solids that settle along the sewer pipe invert need careful consideration since the deposition of this material can cause a blockage.

Sewers are usually designed to flow full or nearly full at peak-flow rates and partially full at lesser flows with the flow surface exposed to the atmosphere and thus functioning as an open channel. During extreme peak flows, sewers could in fact surcharge the manholes and the sewers and then become pressurized conduits.

There is thus variation in flow rates, deposition of material and frequent changes in slope, different pipe sizes, manholes and other hydraulic control structures that need to be considered in the hydraulic design of sewers.

9.1 Hydraulic principles

The flow of wastewater in sewers may be open-channel or pressure flow. When flow fills the conduit and the hydraulic grade line (HGL) rises above the sewer crown, the flow is classified as pressure flow. When the conduit is partially full and the HGL is below the sewer crown and a free-water surface develops in the sewer, the flow is classified as an open-channel flow. Open-channel flow will be the basis for general hydraulic design of sanitary sewers.

9.1.1 Types of open-channel flow

The following types of open-channel flow may be found in sewers:

- *Steady flow* occurs when the depth of flow is constant with respect to time
- *Unsteady flow* occurs when the depth of flow is not constant with respect to time
- *Uniform flow* occurs when the depth of flow does not change with respect to location
- *Non-uniform flow* occurs when the depth of flow changes with respect to location

The various combinations of these types are listed in **Table 9.1**.

Table 9.1: Types of open-channel flow

Type of flow	Steady	Unsteady
Uniform	This flow occurs when in a given stretch of a sewer pipe, having a constant shape, size, slope and interior roughness, a constant rate of flow enters the upstream end of the pipe and the same exits at the downstream end of the pipe. In this flow regime, the depth of flow is constant with respect to time and location and the HGL is parallel to the sewer invert slope.	This flow occurs when the HGL remains parallel to the sewer invert and fluctuates up and down as the rate of flow fluctuates with time. This type of flow is not very common in sewer design.
Non-uniform	This flow shall be considered when different constant rates of flow enter a sewer along its length at various locations. However, a simplification of this case is used in the design of such sewers. Accordingly, the sum of all the flows for a given stretch of the sewer is assumed to enter the pipe at its upstream end, thereby reducing the flow regime to a steady uniform case.	This flow develops during the onset and termination of PWWFs. However, design of sewers based on this flow regime is seldom required, as it involves extensive calculations for flow routing, wave and water surface profiles.

In general, the design of sanitary sewers shall be based on steady uniform flow analysis employing typically the Manning equation.

In sanitary sewers, drawdown and backwater curves are encountered in many situations. **Figure 9.1** shows the backwater and drawdown curves encountered in a branch joining a larger sewer. Depending on the flow conditions in the larger sewer, backwater or drawdown curves can develop in the branch.

9.1.2 Supercritical and subcritical flow

The designer should be able to identify supercritical, subcritical and critical flows. Flows within 10% to 15% of critical depth are likely to be unstable and these should be avoided. It is, however, not always possible because of diurnal flows. The designer should, however, be mindful of flow characteristics throughout the flow regime from minimum to PWWF.

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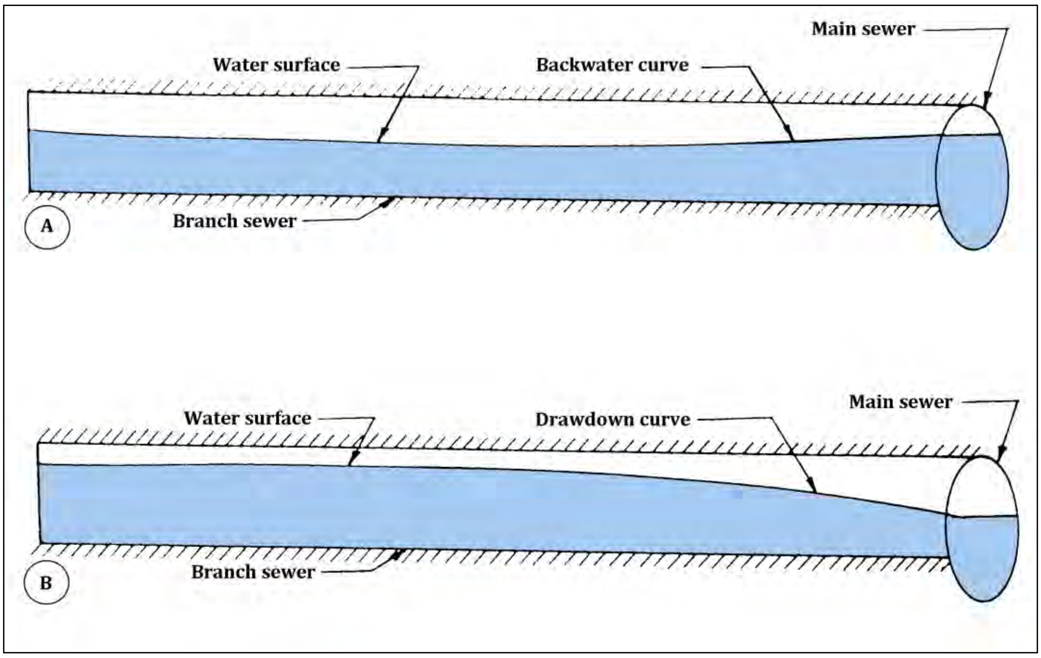

Figure 9.1: Water surface profiles in branch sewer caused by flow in main sewer (ASCE, 1982)

For a given rate of flow and channel cross-section, the specific energy H_o as shown in the following equation is a function of depth:

$$H_o = d + \frac{Q^2}{2gA} \qquad \text{...(9.1)}$$

A plot of this function produces a specific energy curve like the one shown in **Figure 9.2**. There is one depth at which H_o is a minimum. That is the *critical depth* d_c and the corresponding velocity at the depth is the *critical velocity* V_c. Each larger value of H_o can occur at either of two alternate depths. The upper depth d_u is greater than d_c while the corresponding velocity V_u is less than V_c. This flow would be subcritical. For the lower depth, d_l is less than d_c, while the corresponding velocity V_l is greater than V_c with this flow then being supercritical.

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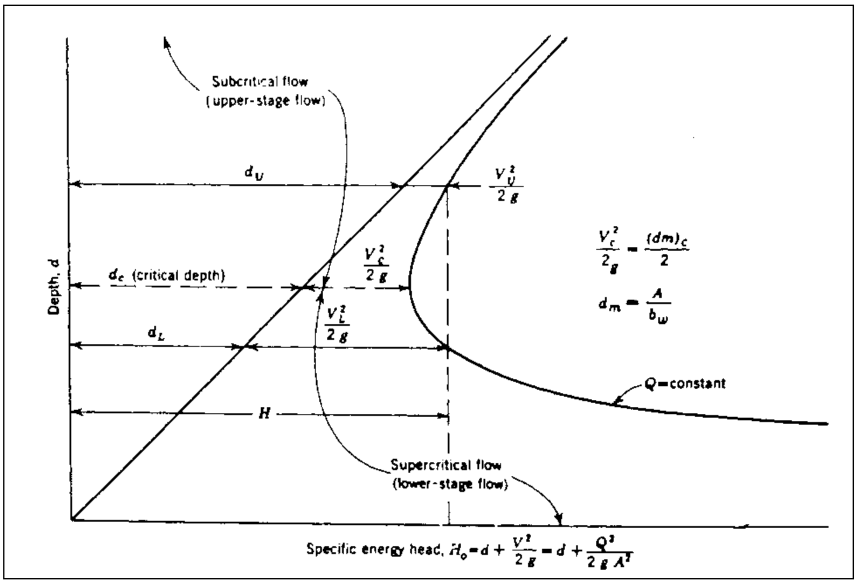

Figure 9.2: Specific energy curve (ASCE, 1982)

Figure 9.3 shows an example profile of a sanitary sewer which transitions from a steep slope to a medium slope and the different flow regimes. Upstream of the change, the steep slope produces a velocity that is greater than a certain critical value and a small depth of flow results, i.e. supercritical flow. For the same rate of flow, the medium downstream slope produces a velocity that is less than the critical value but with a greater depth and this flow is called subcritical. Somewhere near the change in slope, the depth increases abruptly from the smaller depth to the greater depth causing a hydraulic jump.

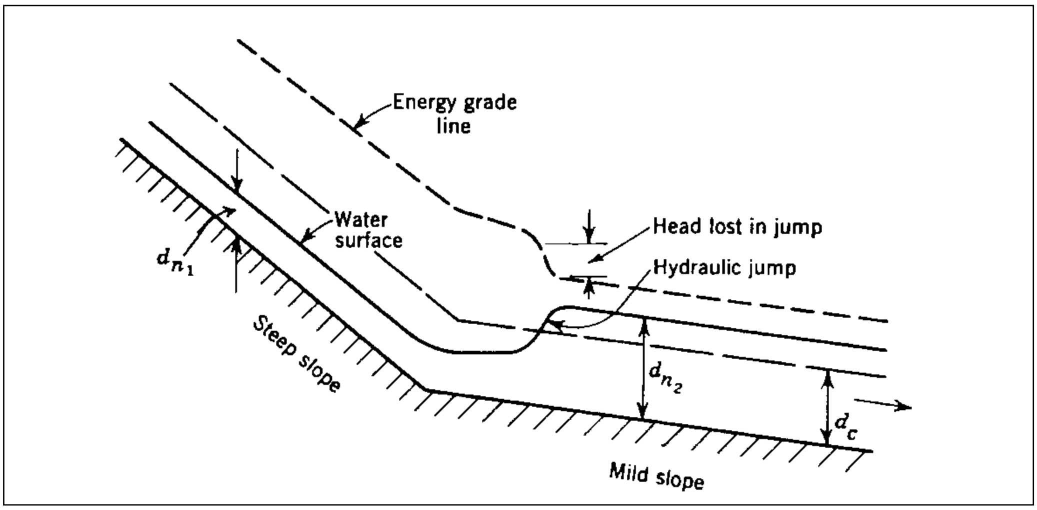

Figure 9.3: Example of flow regimes in sewers (ASCE, 1982)

The hydraulic jump takes place over a relatively short distance. It has an irregular surface with a high degree of turbulent motion, mixing and energy dissipation. Careful consideration should be given in the design of sewers to avoid hydraulic jumps. The rapid decrease in flow velocity across the jump may result in deposition of solids in the downstream conduit and the turbulence causes the release of sulphide gases held in solution. For this reason vertical curves are often used at significant changes in grade to avoid a hydraulic jump.

Computation of *critical depth* is necessary to determine whether flow may be supercritical or subcritical. Normal flow depth is compared with critical depth to determine if flow is supercritical or subcritical. If normal flow depth is above critical depth, the flow is subcritical. If normal flow depth is below critical depth, the flow is supercritical.

For circular pipes, the following formulae (Equations (9.2a) and (9.2b)) can be used to compute critical depth. Critical depth can then be compared to the design depth to determine if flows will be subcritical or supercritical and whether or not a hydraulic jump may occur. Critical depth occurs when energy is at a minimum with respect to depth, dE/dY=0. A numerical solution method is followed to solve for θ_c and then for y_c.

$$16Q\left[2g\sin\left(\frac{\theta_c}{2}\right)\right]^{\frac{1}{2}} = D^{\frac{5}{2}}\left[\theta_c - \sin(\theta_c)\right]^{\frac{3}{2}} \qquad \text{...(9.2a)}$$

$$y_c = \frac{D}{2}\left[1 - \cos\left(\frac{\theta_c}{2}\right)\right] \qquad \text{...(9.2b)}$$

where:

D = inside diameter of pipe (m)
g = gravitational acceleration (m/s^2)
Q = flow rate (m^3/s)
y_c = critical depth (m)
θ_c = angle at critical depth(radians)

The critical velocity for circular conduits can be calculated using Equation (9.3).

$$V_c = \left(\frac{gA}{B}\right)^{1/2} \qquad \text{...(9.3)}$$

where:

V_c = critical velocity (m/s)
A = flow area (m^2)
B = width of water surface (m)
g = gravitational acceleration (m/s^2)

9.1.3 Sewer gases

The fluid in motion in open channels drags along the air and sewer gases in contact with it, creating a flow of air and sewer gases in the space above the wastewater. When sections of the sewer pipe fill with wastewater, this free flow of air and gases in the upper portion of the pipe is inhibited and then, under slightly positive pressure, is forced to the surface through the nearest openings such as maintenance holes, roof vents, yard drains, etc. The sewer gases forced into the atmosphere are heavier than air and have a pronounced rotten-egg odour. Sewer gases can include mixtures of nitrogen, oxygen, carbon dioxide, hydrogen sulphide, ammonia, and methane and may be combustible and toxic.

To avoid the odour problems associated with sewer gases, the sewer system under normal operating conditions should allow for the transport of the air and gases to the wastewater treatment facility. This will require the designer to know where the HGL is for the various stages of flow, especially at confluence or diversion structures. Where the sewer is planned to flow full, such as for inverted siphons, a separate air line should be provided for conveyance of the sewer gases to a downstream portion of the system where they re-join the flow stream.

9.2 Energy losses in sewers

9.2.1 Friction losses

The friction slope of a pipeline that has a constant gradient and no transitions is the energy difference between the inlet and the outlet divided by the pipeline length and will be the same as the conduit gradient assuming there are no backwater effects. At any position along the pipeline where there is a transition an additional energy loss will occur which reduces the available energy to overcome friction. In this case the friction slope will not be the same as the conduit gradient.

Flow in sewers can be calculated using either the Manning or Kutter formula with '*n* - roughness parameter' or the Colebrook-White Darcy Weisbach or Chezy equation with 'k_s - roughness parameter', see **Tables 9.2, 9.3** and **9.4**.

Table 9.2: Friction formulae

Formulae		Parameter and units
Manning	$Q = \frac{1}{n}\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}S^{\frac{1}{2}}$	Q = flow rate (m^3/s) n = coefficient of roughness ($s/m^{1/3}$) A = flow area (m^2) P = wetted perimeter (m) R = hydraulic radius (m) – A/P S = slope of the energy grade line (m/m) ν = kinematic viscosity (m^2/s) k_s = absolute roughness of conduit (m) g = gravitational acceleration (m/s^2)
Kutter	$Q = \left[\frac{\frac{1,81}{n} + 41,67 + \frac{0,0028}{S}}{1 + \frac{n}{\sqrt{R}}\left(41,67 + \frac{0,0028}{S}\right)}\right]A\sqrt{RS}$	
Colebrook-White Darcy-Weisbach	$Q = -2A\sqrt{2gDS}\,\log\left(\frac{k_s}{3,7D} + \frac{2,51\nu}{D\sqrt{2gDS}}\right)$	
Chezy	$Q = 18\log\left(\frac{12R}{k_s}\right)A\sqrt{RS}$	

Factors influencing the friction coefficients

Numerous experiments in laboratories and field tests have been performed to determine the friction coefficients for various materials and conditions. Factors which affect the selection of a representative roughness coefficient are the conduit material, size, shape, and depth of flow. The designer should also take cognisance of the following:

- Rough, opened or offset joints and joint compounds
- Deposits in sewers, particularly grit accumulation in the invert
- Coatings of grease or other matter on the inner surface of the sewer pipe
- Tree roots and other protrusions
- Flow from laterals disrupting flow in the sewer
- Biofilm growth on wetted surfaces of sewer

A study by Guzmán et al. (2007) on the effect of biofilm formation on roughness coefficient and solids deposition in small-diameter PVC sewer pipes indicated that the roughness increased due to the formed biofilm. The typical roughness parameter found in the literature for a PVC pipe is $n = 0.011\ s/m^{1/3}$. The work done by Guzmán et al. (2007) indicates that this roughness parameter needs to be increased.

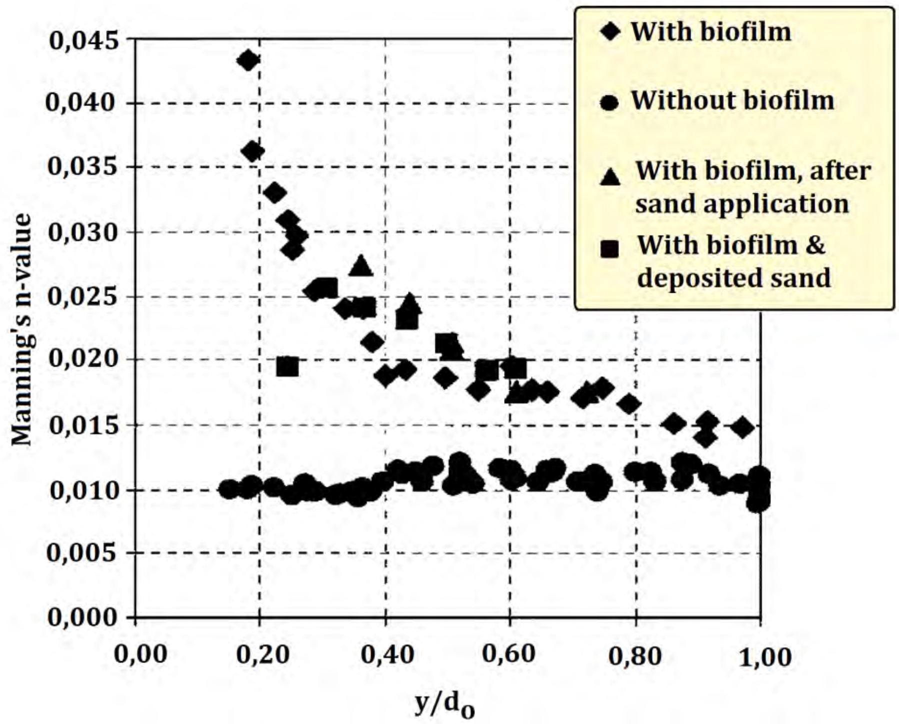

Figure 9.4: Comparison of Manning's n coefficient with and without biofilm (S = 0.1% and d_0=200 mm)

Figure 9.4 is a comparison of Manning's n coefficient with and without biofilm for an 0.1% slope and d_0 =200 mm. **Figure 9.4** illustrates a range of n values observed with biofilm (between 0.014 and 0.043) corresponding to relative depths of between 0.18 m and 0.97 m. There is an inverse relationship between Manning's n and the relative depth. **Figure 9.4** also includes the values of n when sand was added to simulate what happens in real sewers.

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The description in **Figure 9.4**, '*With biofilm after sand application*' means that sand was injected into the pipe and immediately Manning's *n* values were measured, whilst '*Biofilm and deposited sand*' means that the *n* values were measured after the system had been running for several hours. The researchers found that after the sand was introduced, a portion of it was scattered and attached to the biological film and another portion accumulated forming small mounds at several pipe locations. It is, however, interesting that the values of *n* did not change with respect to those corresponding to the pipe coated with biofilm only.

Table 9.3: Recommended roughness parameters (*n*)(Manning)

Pipe	n (s/m$^{1/3}$)
Man-entry plastic lined sewer	0.012
Non man-entry plastic lined sewer	0.013
Plastic	0.013
Standard concrete sewer	0.015
Vitrified clay	0.014

Table 9.4: Recommended roughness parameters (k_s) (Wallingford and Barr, 2006)

Sewer type*	Good	Normal	Poor
New lining of sewers			
Factory-manufactured GRP	0.03	-	-
Brickwork			
Glazed	0.6	1.5	3.0
Well painted	1.5	3.0	6.0
Old, in need of painting	-	15	30
Slimed sewers			
Sewers slimed to about half depth; velocity when flowing half full, approximately 0.75 m/s			
Concrete, spun or vertically cast	-	3.0	6.0
Asbestos cement	-	3.0	6.0
Clay-ware	-	1.5	3.0
uPVC	-	0.6	1.5
Sewers slimed to about half depth; velocity when flowing half full, approximately 1.2 m/s			
Concrete, spun or vertically cast		1.5	3.0
Asbestos cement		0.6	1.5
Clay-ware		0.3	0.6
uPVC		0.15	0.3
Sewer rising main			
Mean velocity 0.5 m/s	0.3	3.0	30
Mean velocity 0.75 m/s	0.15	1.5	15
Mean velocity 1.0 m/s	0.06	0.6	6.0
Mean velocity 1.5 m/s	0.03	0.3	1.5
Mean velocity 2.0 m/s	0.015	0.15	1.5

Notes: *Classification - 'Good' and 'Normal' assumed new and clean unless otherwise stated

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9.2.2 Secondary losses

In addition to the friction along the conduit, there are usually local losses of energy associated with sudden changes such as an expansion or junction, in- and outlets and control devices. These losses occur over a relatively short distance and are usually expressed as a head loss (Equation (9.4)):

$$\Delta H_L = K\frac{V^2}{2g} \qquad \text{... (9.4)}$$

where:

ΔH_L	=	secondary head loss (m)
K	=	loss coefficient
V	=	average velocity (m/s)
g	=	gravitational acceleration (m/s^2)

Transition losses due to converging flow are smaller than those due to diverging flow. The transitions in outfall sewers are usually made at manholes. Benching should be made inside the manholes to conform to the sewer profile, thus minimizing the velocity differences through the sewer and manholes. Under these circumstances the energy loss coefficient will be small. When manhole pipes are used, or the benching matches the flow profile of the sewer, the coefficient will be zero (PIPES, 2009). In cases where benching is not provided in the manholes the energy losses can be excessive. Commonly used energy loss coefficients are given in **Table 9.5**.

Table 9.5: Energy loss coefficients

Description		*K*-value
Pipe exit	Projecting sharp-edged rounded	1.0
Pipe entrance	Inward projecting	0.78
Pipe entrance flush	Sharp-edged	0.50
	$r/d = 0.02$	0.28
	$r/d = 0.04$	0.24
	$r/d = 0.06$	0.15
	$r/d = 0.10$	0.09
	$r/d < 0.14$	0.04

A third type of loss is usually determined from a combination of the energy and momentum equations rather than from a coefficient. This loss includes the hydraulic jump and a junction loss.

Hydraulic jump

The hydraulic jump occurs where there is a flow-regime change from supercritical type flow to subcritical type flow. It is required to apply the momentum equation in order to solve the energy loss in the hydraulic jump. The forces acting on the fluid between two cross-sections are shown in **Figure 9.5**.

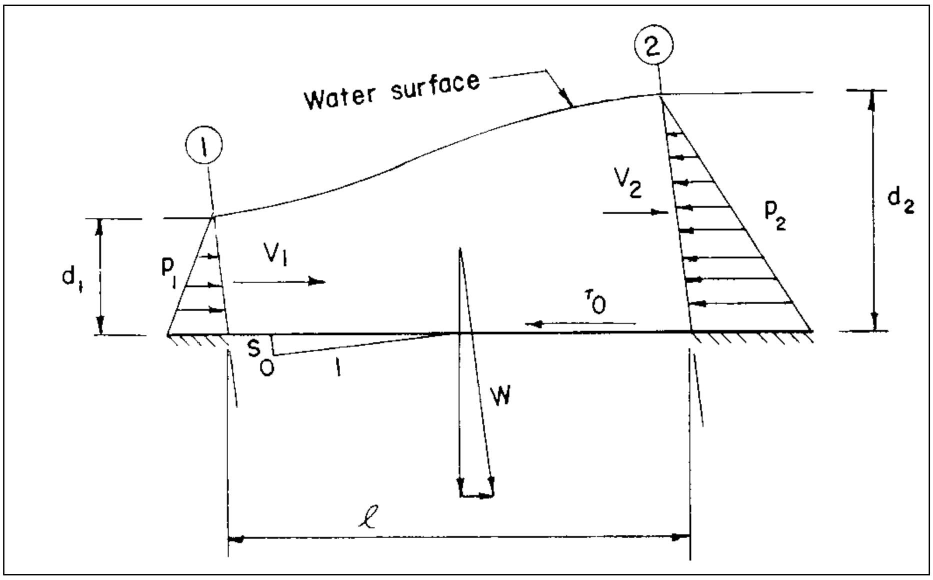

Figure 9.5: Forces acting on fluid at hydraulic jump (ASCE, 1982)

A stable hydraulic jump is formed where the forces at Sections 1 and 2 are equal. The summation of the pressure forces, weight and friction force results in Equation (9.5).

$$\frac{1}{2}\left(B_1 d_1^2 - B_2 d_2^2\right) + \frac{1}{2}(A_1 + A_2)S_o - \frac{1}{4}(A_1 + A_2)(S_{f1} + S_{f2}) = \frac{1}{g}\left(\frac{Q_2^2}{A_2} - \frac{Q_1^2}{A_1}\right) \quad \text{... (9.5)}$$

where:

A_1 & A_2 = flow areas at Sections 1 and 2 (m^2)
B_1 & B_2 = width of water surface at Sections 1 and 2 (m)
d_1 & d_2 = flow depth at Sections 1 and 2 (m)
Q_1 & Q_2 = flow rates at Sections 1 and 2 (m^3/s)
S_{f1} & S_{f2} = friction slopes at Sections 1 and 2(m/m)
So = slope of the conduit (m/m)
g = gravitational acceleration (m/s^2)

The momentum equation is often used in situations in which the length of open channel under consideration is short, or the invert and friction slopes are negligible compared to the other terms in the equation. For these conditions in a rectangular conduit Equation (9.5) reduces to the following form:

$$\frac{Q_1^2}{gA_1} + \frac{B_1 d_1^2}{2} = \frac{Q_2^2}{gA_2} + \frac{B_2 d_2^2}{2} \quad \text{... (9.6)}$$

With a minor modification Equation (9.6) can be applied to non-rectangular conduits (Equation (9.7)). The term $\bar{d}$ is the depth from the water surface to the centroid of area A:

$$\frac{Q_1^2}{gA_1} + \bar{d}_1 A_1 = \frac{Q_2^2}{gA_2} + \bar{d}_2 A_2 \quad \text{... (9.7)}$$

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Junctions

Junctions in conduits can cause major losses in both the energy grade and the hydraulic grade across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may be seriously restricted. The pressure plus momentum theory, which equates the summation of all pressures acting at the junction with the summation of the momentums, affords a rational method of analysing the hydraulic losses at a junction (Bureau of Engineering, 2007).

9.3 Self-cleansing velocity

As indicated above the deposition of suspended material is of particular concern in the design of sewers. The deposited material at the bottom of the sewer does not remain there if the velocity and turbulent motion are sufficient to re-suspend or move the settled particles along the bottom (ASCE, 1982). This velocity which is sufficient to prevent the deposition of material is called the *self-cleansing velocity*. The velocity that is required in a pipe flowing full to transport the sediment is given in Equation (9.8).

$$V = \frac{R^{\frac{1}{6}}}{n}\left[B(s-1)D_g\right]^{\frac{1}{2}} \qquad \text{...(9.8)}$$

where:

s	=	specific gravity of the particle
D_g	=	diameter of the particle (m)
B	=	dimensionless constant (0.04 to start motion of clean granular particles and about 0.8 for adequate self-cleansing of cohesive material)
V	=	velocity (m/s)
R	=	hydraulic radius (m)
n	=	coefficient of roughness (s/m$^{1/3}$)

It is not always feasible to conduct an analysis as indicated above and a minimum velocity is often accepted as the design criterion. Once a minimum velocity for full conduits has been selected, it is possible to determine the corresponding minimum slope for each size and roughness as indicated in **Table 5.6**.

Also see Butler et al. (2003) for self-cleansing sewer design based on sediment transport principles.

9.4 Properties of circular sections

The flows in sewers are usually open-channel type flow, which means that there is always some free space above the flow of wastewater in the sewer. The hydraulic design of sewers requires knowledge of the area of flow and the hydraulic radius. Both these parameters vary with the depth of flow, as shown in **Figure 9.6** and trigonometric relationships can be derived for the area of flow, wetted perimeter, hydraulic radius and breadth of flow.

The hydraulic radius (sometimes called the hydraulic mean depth) is the area of flow divided by the wetted perimeter. The breadth of flow is used for the calculation of the risk of H_2S generation.

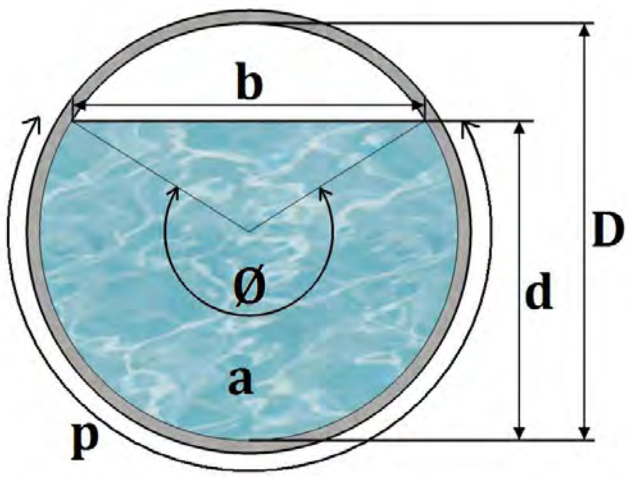

Figure 9.6: Definition of parameters for open-channel flow in circular sewers

The ratio d/D is termed the proportional depth of flow (which is dimensionless). The equations are provided in **Table 9.6**.

Table 9.6: Formulae for partially full flowing pipes

Description	Equation	Eq. No.
Angle of flow	$Ø = 2\cos^{-1}\left(1 - 2\frac{d}{D}\right)$	(9.9)
Area of flow	$a = D^2\left(\frac{(Ø - \sin Ø)}{8}\right)$	(9.10)
Wetted perimeter	$p = Ø\frac{D}{2}$	(9.11)
Hydraulic radius (a/p)	$r = \left(\frac{D}{4}\right)\left(1 - \frac{\sin Ø}{Ø}\right)$	(9.12)
Breadth of flow	$b = D\sin\left(\frac{Ø}{2}\right)$	(9.13)

where:

D	=	pipe inner diameter (m)
d	=	depth of flow (m)
n	=	coefficient of roughness ($s/m^{1/3}$)
A	=	full-flow area (m^2)
P	=	wetted perimeter for full flow (m)
R	=	hydraulic radius full flowing (m) – A/P
a	=	area of partially full flowing pipe (m^2)
p	=	wetted perimeter of partially full flowing pipe (m)
r	=	hydraulic radius of partially full flowing pipe (m)
b	=	breadth of flow (m)
$Ø$	=	angle of flow (radians)

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9.5 Tractive force

This is an extract from *Simplified Sewerage: Design Guidelines* by Bakalian et al. (1994).

The tractive force method is a design process that is widely used in the design of open channels. Like the minimum velocity design methodology, it is based on the concept of 'threshold of movement' and makes use of the minimum force required to move a certain size of settled particle. The resistance equation is given by:

$$\tau = \Gamma RS \qquad \text{...(9.14)}$$

where:

τ = boundary shear stress (kg/m²)
Γ = specific weight of water (kg/m³)
R = hydraulic radius (m)
S = slope of the conduit (m/m)

The minimum design slope is derived by incorporating Equation (9.14) into Manning's equation and solving for the minimum slope with the assumption that the depth of the minimum flow is 20% of the diameter (d/D = 0.2).

Based on this assumption a relationship can be obtained between the hydraulic radius and the diameter using Equations (9.10) and (9.11). Assuming n = 0.013 s/m$^{1/3}$ and τ = 0.1 kg/m^2, Γ = 1 000 kg/m^3 transforms the Manning equation combined with Equation (9.14) into Equation (9.15) for determining the minimum slope required for simplified sewers.

$$S_{min} = 0{,}0054Q^{-0{,}462} \qquad \text{...(9.15)}$$

where:

Q = flow rate (ℓ/s)
S_{min} = minimum slope (m/m)

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10. SPECIAL STRUCTURES

10.1 Siphon design

10.1.1 Introduction

Within the sanitary sewerage system there are numerous special structures serving particular needs. These special structures include inverted siphons crossing rivers, creeks, depressed highways and other obstructions, see **Figure 10.1**. Inverted siphons and airlines (sometimes called an 'air jumper') are constructed to convey sewage flows (liquid and gases) across obstructions where such crossings cannot be attained by a sewer placed on a continuous grade. Inverted siphons and airlines are designed to meet criteria which ensure proper functioning during the design period of the system to be fail-safe and to minimize maintenance and odours.

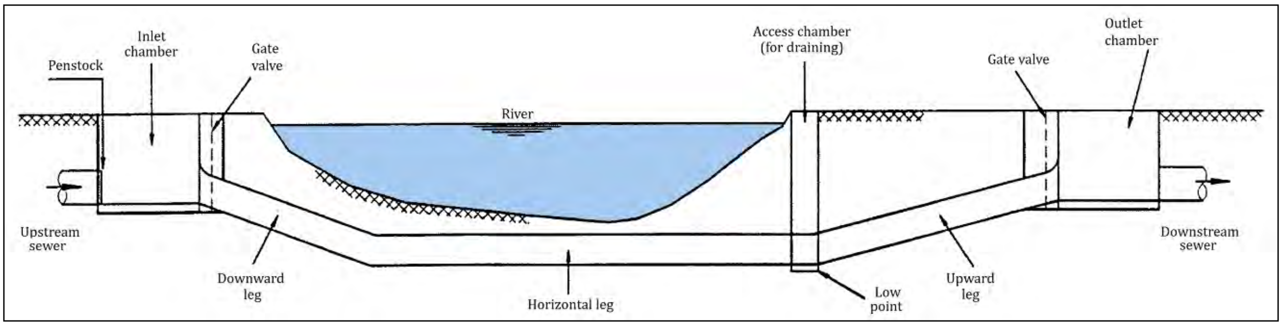

Figure 10.1: Typical inverted siphon

Two considerations which govern the profile of a siphon are the requirements to provide for hydraulic losses and to ensure ease of cleaning:

- The friction loss through the barrel is determined by the design velocity and additional losses due to side-overflow weirs and directional changes are also taken into account

- Siphons need cleaning more often than gravity sewers. The siphon should therefore have no sharp bends (vertical or horizontal) or changes in pipe diameter. The rising leg of the siphon should not be too steep complicating the removal of heavy solids.

The selection of the location of the inverted siphons should be such that there is sufficient space to not only contain the physical structures, but also to allow vehicles, workers and equipment to enter and perform any construction, repair, maintenance and operational activity.

10.1.2 Single- or multiple-barrel design

The design of both inverted siphons and airlines may involve either single or multiple barrels. In general, a single unit is hydraulically and structurally more efficient, and will be cheaper to construct and to maintain, than a multiple system. For inverted siphons, a minimum of two barrels is recommended.

One redundant barrel shall always be provided for bypass capacity, for emergencies, and for use when another barrel is taken off-line for maintenance or repairs. When two barrels are installed, they should be the same size, each one capable of conveying the full design-flow rate. When three or more barrels are installed, they should, if possible, be of the same size, provided minimum velocities can be attained. If this is not possible, the redundant barrel should be of the same size as the largest of the other barrels so as to ensure bypass capacity.

10.1.3 Hydraulics

The hydraulic capacity of an inverted siphon shall never be less than the capacity of the sewer system upstream of the inverted siphon. Hydraulically, inverted siphons shall be designed so that for the ADWF, the preferable minimum self-cleaning velocity is obtained to prevent material from depositing in the conduit, which in turn will result in blockages, higher maintenance costs and a shorter life. The daily PDWF shall always provide a minimum self-cleaning velocity at least once a day. Hydrographs indicating a wide range of values of flow rates and/or velocities usually indicate the need for multiple barrels. Inflows to and outflows from a multiple barrel can be controlled by manual or automatic gates and/or weirs. The minimum size of any inverted siphon conduit shall be 150 mm.

10.1.4 Horizontal alignment

Economically, the most cost-effective system is usually the shortest in length. The shortest system would be one that is normal to, or radial to, in the case of a curved facility, the facility that is being crossed. The alignment should be a single, straight alignment. A curved alignment or one with an angle point should be avoided. Of the latter two, the curved alignment is less objectionable. If a curved alignment is necessary, an access structure for maintenance purposes shall be constructed at both ends of the curve. If an angle point is necessary, an access structure for maintenance purposes shall be constructed at the angle point.

10.1.5 Vertical alignment

The vertical alignment of an inverted siphon should also be a straight alignment with bends and angle points minimized. Obviously, an inverted siphon cannot be constructed without either or both. An inverted siphon with a vertical curve is preferred to one with a sudden change of grade, but this is often difficult to construct with straight sections of pipe. If possible, an access structure for maintenance should be constructed at any change in grade. The best design is for a uniform grade from one end of the inverted siphon to the other end. The maximum slope of the downstream (rising) leg approaching the outlet structure shall be such that solids are allowed to be conveyed upwards of the conduit into the outlet structure.

During the initial design of the sewer siphon the indicative values of minimum self-cleansing velocity as shown in **Table 10.1** can be used.

Table 10.1: Indicative values of minimum velocity and discharge for inverted siphons (May, 2003)

Sediment loading	Internal pipe diameter (m)	Minimum self-cleansing flow conditions	
		Velocity (m/s)	Discharge (m^3/s)
Medium (50 mg/ℓ by weight)	0.15	0.68	0.013
	0.225	0.86	0.034
	0.30	1.02	0.072
	0.375	1.17	0.0129
	0.50	1.39	0.273
	0.75	1.78	0.785
	1.00	2.12	1.660
	1.25	2.43	2.980
	1.50	2.72	4.810
	1.75	2.99	7.200
	2.00	3.25	10.20
High (200 mg/ℓ by weight)	0.15	0.89	0.016
	0.225	1.14	0.045
	0.30	1.36	0.096
	0.375	1.56	0.173
	0.50	1.87	0.368
	0.75	2.42	1.070
	1.00	2.90	2.280
	1.25	3.34	4.100
	1.50	3.76	6.640
	1.75	4.15	9.970
	2.00	4.52	14.20

Note: The indicative velocities include a safety factor which varies with the pipe angle but is not less than 10% and is based on an upward slope of 37.5° and representative particle size of 1.5 mm.

Once the siphon system sizing has been completed a more precise estimate of minimum velocity can be obtained using Equation (10.1).

$$C_V = (0{,}0303 - 0{,}0169\sin\theta)\left(\frac{4}{\pi}\right)\left(\frac{d_{50}}{D}\right)^{0,6}\left(1 - \frac{\sigma V_T}{V}\right)^4\left(\frac{V^2}{g(s-1)D\cos\theta}\right)^{3/2} \qquad \text{...(10.1)}$$

where:

C_V = volumetric sediment concentration (Qs/Q)
θ = angle of pipe to horizontal (positive upward) (°)
d_{50} = sediment size for which 50% of sample is smaller by weight (m)
D = internal diameter of pipe (m)
σ = slope factor for total down-slope force in inclined pipe
g = gravitational acceleration (9.81 m/s^2)
V = mean cross-sectional velocity of flow in pipe (m/s)
V_T = value of V for threshold movement of individual sediment particles in nearly horizontal pipe (m/s) (Equation (10.2))
s = specific gravity of sediment particles (assumed as 2.6)

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$$V_T = 0{,}125\sqrt{g(s-1)d_{50}}\left(\frac{D}{d_{50}}\right)^{0{,}47} \quad ...(10.2)$$

The flatter the rising leg of the siphon the lesser the minimum required velocity will be.

For an inverted siphon crossing a stream or waterway, the top of the inverted siphon shall be not less than 1 m below the level of possible scour in the stream or waterway, nor shall the inverted siphon be located in close proximity to an outlet of a lateral or a drop structure that could cause undesirable effects.

10.1.6 Hydraulic design of inverted siphon

Hydraulically, inverted siphons are designed like any other pipeline or conduit by using any of the Manning, Kutter, Colebrook-White Darcy Weisbach or Chezy equations to determine the friction loss (see **Table 9.2**).

The losses due to bends, angle points, junctions and diversions, and other hydraulic losses still need to be considered.

10.1.7 Structural design

Inverted siphons must be designed structurally to withstand all loads anticipated during the design period. The inverted siphon, which is always buried, may, at various times, be either full and under pressure, or empty, and the conduit should be designed for both conditions. All dead and live loads, internal pressures as well as all other design criteria, including allowable stresses, for conduit materials and soil loading should be adhered to. In some cases it may be required that the inverted siphon be installed within another casing or carrier conduit.

Positive pressure develops in the sewer atmosphere upstream of a siphon because of the downstream movement of air induced by the sewage flow. Air therefore tends to exhaust from the manhole at the siphon inlet. Under all except the maximum flow conditions, there is a drop in water surface elevation into a siphon, with consequent turbulence and release of odours. The exiting air hence causes serious odour problems. Conversely, air is drawn in at the siphon outlet.

10.1.8 Corrosion

In general, a surface exposed to sewer gases is always subject to corrosion. Similarly, any surface that is intermittently wet or dry from liquid sewage is also subject to corrosion. Conduits that normally flow full but may be evacuated intermittently (i.e., during maintenance operations) are also subject to corrosion.

Inverted siphons and airlines are subject to corrosive environments. Failure to protect the system will result in premature failure. Therefore, all components of the system which may, in any manner, be exposed to sewage flows or gases shall be designed to preclude corrosion.

This can be accomplished either by specifying materials for the inverted siphon that are inherently corrosion resistant, or are treated in some manner (i.e., coated for example Anchor Knob sheeting) by material that is corrosion resistant.

10.1.9 Material selection

Materials used for inverted siphons and airlines are many and varied. They can be rigid or flexible. They can be pre-formed, precast or prefabricated, or cast or formed in place. They shall be selected with extreme care to ensure structural integrity, ability to function during the required design period and be either corrosion resistant or amenable to some treatment to properly resist corrosion.

Portland cement concrete is commonly used. The type specified is dependent on the corrosion anticipated during the life of the system, including reactive soils, and availability. A properly designed system will provide the necessary structural strength and meet the design period requirements. Its main drawback is that concrete is not corrosion resistant to sewage atmospheres. When used, surfaces exposed to sewer gases or intermittent liquid sewage flows must be provided with some form of protection against corrosion, such as a PVC or HDPE plastic liner.

Steel is a material that is commonly used for both inverted siphons and airlines, where applicable. Its primary advantage is its high strength. Its primary weakness is that it is subject to severe corrosion and must be provided with some form of corrosion protection, except for certain stainless steels.

Cast iron and ductile iron can also be utilized for inverted siphons. Whilst having similar strength as steel they are usually more brittle than steel, but still stronger than many other materials. They are usually resistant to normal atmospheric conditions but not resistant to most sewage atmospheres. They must be provided with some form of corrosion protection. Reinforced concrete pipe (RCP) and reinforced concrete box (RCB) are commonly and effectively used for inverted siphons, especially in large diameters, and sometimes for airlines when sufficient support is available. Its advantages are its high strength, economy, abrasion resistance and resistance to atmospheric corrosion. Its only disadvantage is that it is subject to corrosion in the presence of sewage atmospheres or gases. This can occur whenever the inverted siphon barrel is emptied of liquid and air is allowed to enter. This disadvantage is easily overcome by lining the entire interior with polyvinyl chloride (PVC) sheets. Any sanitary sewer or inverted siphon or airline should always have joints that are airtight, i.e. using sealing rings such as 'O-ring' gaskets or similar sealing systems.

Cast iron and ductile iron pipe are also utilized in inverted siphon airlines. Their advantages are their high strength, longer precast lengths and their ability to resist atmospheric corrosion. In most cases, especially for smaller to medium sizes, CIP and DIP are the material of choice. They should also be fitted with sealing rings at joints and provided with some form of corrosion protection internally.

Vitrified clay pipe (VCP) can be used for inverted siphons under some circumstances. Their advantage is that they have relatively high strength, are completely corrosion resistant, and depending on the joint utilized, are not only water- and airtight, but can sustain a degree of pressure. Their disadvantage is short laying lengths, and inability to sustain high internal pressures. For inverted siphons, they can be readily utilized provided the internal pressure can be sustained by the pipe and its joints.

PVC and polyethylene (PE) solid wall plastic pipes can be used for inverted siphons. Typically they shall not be used for airlines unless protected from sunlight and ultraviolet (UV) rays. These materials shall be obtained in pressure-rated classes, fabricated from resins that are pressure-rated and are corrosion resistant.

Weirs, stop-logs and similar devices can be fabricated of stainless steel, corrosion resistant plastics or wood. Steps, ladders, maintenance-hole frames and covers, gratings and other appurtenances may be fabricated of stainless steel, cast or ductile iron or carbon steel. Except for stainless steel, these items shall be coated with a corrosion-resistant epoxy, PVC, PE, polypropylene, or non-solvent polyurethane.

10.1.10 Appurtenant structures for inverted siphons

Appurtenant structures in an inverted siphon system include inlet structures, outlet structures, access structures and cleanouts. Inlet structures typically require inlet-control systems which convey liquid sewage from the approach pipeline to a single- or multiple-barrel inverted siphon (see **Figure 10.2**). If a single inverted siphon conduit is selected, an access maintenance hole may be all that is required. The maintenance hole shall be sized sufficiently to allow for any maintenance and operation procedure.

Typically, depending on the approach conduit size, this shall be not less than 1.2 m in diameter to allow adequate work room, and usually not more than 2.0 m in diameter. A rectangular maintenance hole may be selected as this could simplify connection of the inlet to the airline. Typically, for a circular inlet structure, precast concrete pipe is used. The interior surface shall be lined, coated or otherwise protected with a suitable corrosion-resistant material. For a rectangular inlet, cast-in-place concrete is most common, but precast concrete box sections may be utilized. The interior surfaces shall be provided with suitable corrosion-resistant material above constantly submerged surfaces.

For inlets to a multiple inverted siphon, precast round or rectangular concrete maintenance holes may suffice; however, a cast-in-place concrete structure is recommended, again protected against corrosion above submerged surfaces. An access to the siphon inlets shall be provided.

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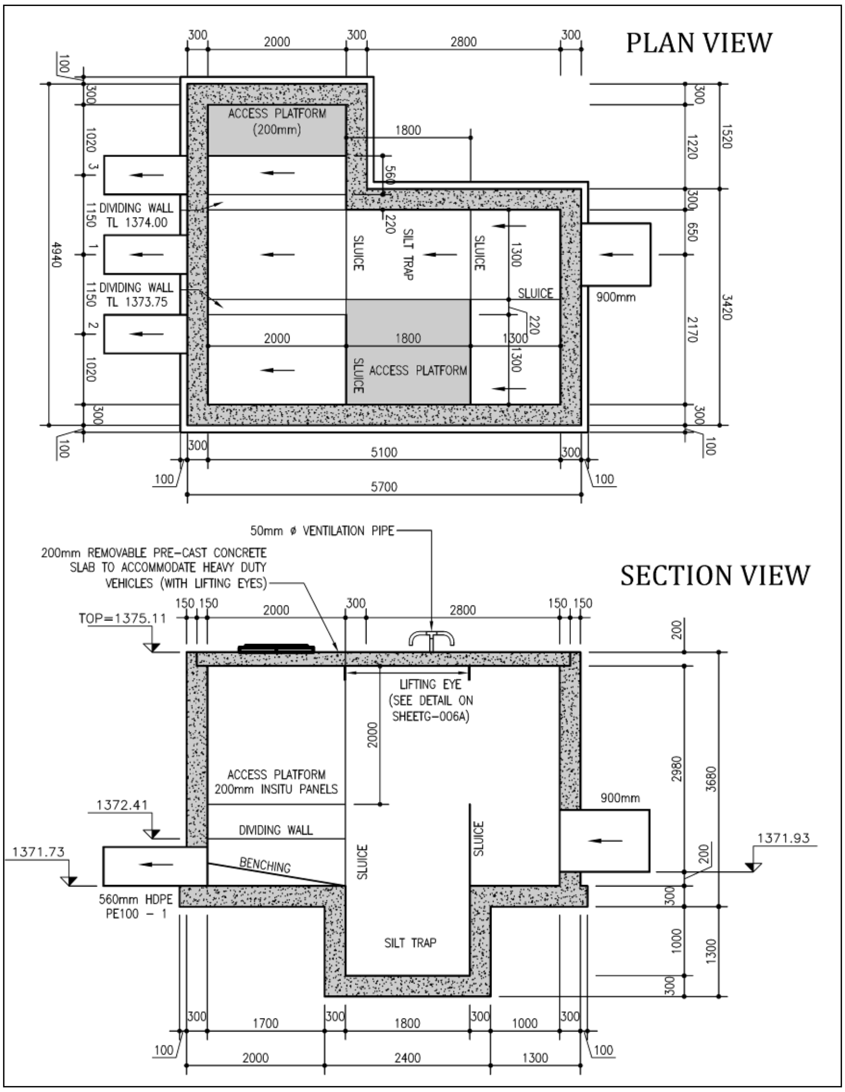

Figure 10.2: Example inlet structure (courtesy of Vela VKE (Bfn))

Inside the inlet structure there are gates, either manually, mechanically or automatically operated, to control flows into the siphons. Similarly, control weirs may be located in these structures. Gauging stations and instruments, either manually or automatically operated, are also often installed at these inlet structures.

Any inlet structure, other than a simple maintenance hole designed for workers to enter, must have positive airflow and other protective measures to prevent hazardous conditions.

The inlet structure could include a silt trap to prevent sand and grit entering the siphon which could cause clogging. The size of an inverted siphon is usually smaller than the inflowing sewer.

Inverted siphon outlet structures are similar to inlet structures except that they are less complex and usually will not have all of the features of inlet structures, see **Figure 10.3**. Size and complexity will determine its configuration and design.

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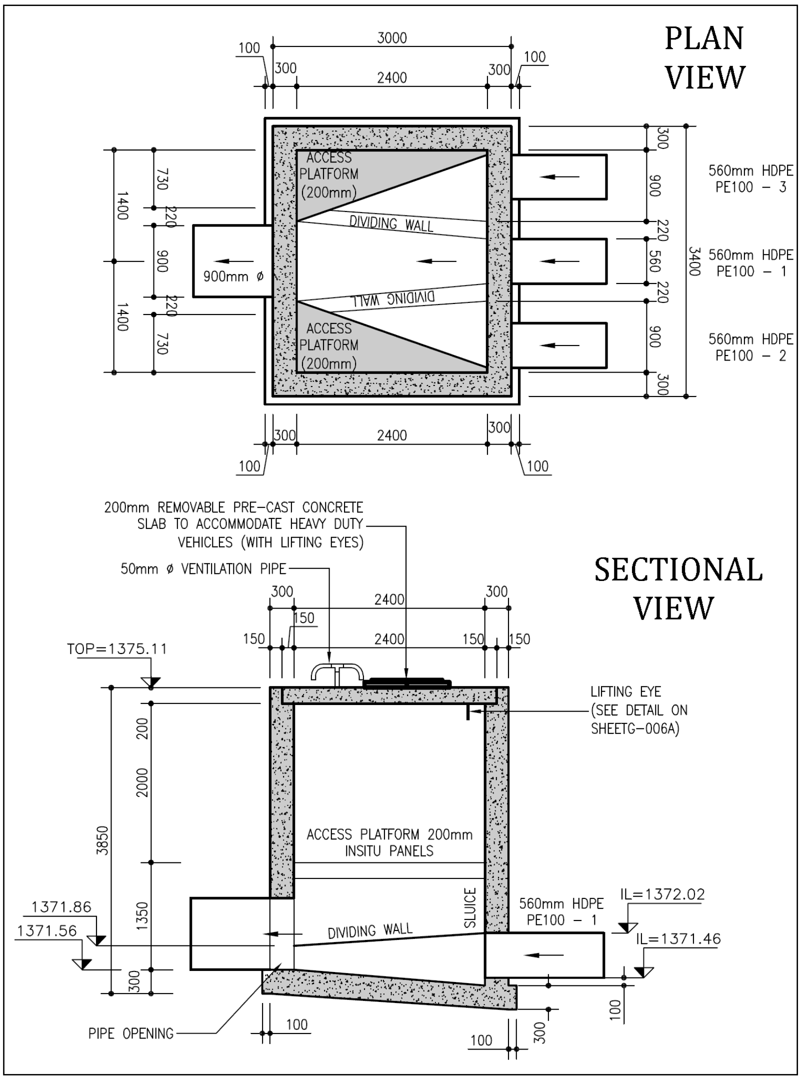

Figure 10.3: Example outlet structure (courtesy of Vela VKE (Bfn))

Cleanouts shall be provided whenever the length of the inverted siphon exceeds 120 m. The size of the cleanout shall be sufficient to handle the debris that may accumulate, and at least as large as the inverted siphon itself. A cleanout is mandatory at any low point. Access structures shall be provided whenever access for maintenance or repairs may be necessary. The size shall be sufficient to allow workers to enter with materials, tools and equipment and to perform their tasks.

10.2 Silt traps

Silt traps are designed to trap sand and grit. This is usually achieved by reducing the flow velocity and allowing enough time/distance for the particles to settle and remain in the trap. Depending on the anticipated volume of silt transported in the sewer system the sizing of the settling bay will in turn determine the number of times it needs to be emptied. During the design phase consider the following:

- The access to the trap should be designed large enough to provide adequate access. Removable concrete cover slabs can be considered or a sliding roof structure for larger types, as shown in **Figure 10.4**.

- Consider providing alternative diversion, i.e. bypass when the main trap area is being emptied

- Select a location where a dump truck or trailer can be brought in close proximity to the trap for ease of cleaning

- Preferably locate the silt trap a distance away from the residential areas for odour control

Figure 10.4: Sliding roof cover for silt trap (courtesy of BKS (Pty) Ltd)

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11. STRUCTURAL DESIGN

11.1 Introduction

The structural design of a sewer requires that the supporting strength of the pipe as installed must equal or exceed the external loading multiplied by a factor of safety. The following criteria for structural design of sewers are based on the assumption that sewers will be laid in open trenches entirely below natural ground surface and backfilled with suitable materials; that the sides of the trench will be nearly vertical below the top of the pipe and will have slopes no flatter than one horizontal to two vertical above the pipe; and that the trench width at the top of the pipe will be relatively narrow. In general, the trench width will be limited to the maximum allowed or that specified by the regulatory body such as the SABS (SABS, 1989). Special cases involving sewer installation in unsatisfactory soil, rock, embankments or fills, sewers requiring jacking, boring or tunnelling, and pipe placed above ground, are too exceptional to warrant lengthy consideration in this guide.

11.2 Loads on sewers

The loads on buried sewer pipelines are divided into primary and secondary loads. There are four kinds of primary loads to which a sewer laid in a trench may be subjected. These are:

- Loads due to trench filling materials
- Uniformly distributed surface loads, such as stockpiled materials or loose fill
- Concentrated surface loads, such as those from truck wheels
- Internal pressure

Secondary loads are more difficult to predict and calculate. These loads can, however, cause considerable damage to a sewer pipeline due to differential movement between pipes and thus it is important that their potential impact be recognised. Some examples of factors that could cause secondary loads are (PIPES, 2009):

- *volume changes in clay soils due to changing moisture content;*
- *unexpected foundation and bedding behaviour;*
- *settlement of an embankment foundation;*
- *the elongation of the pipeline under deep fills;*
- *effects of thermal and moisture changes in surrounding materials;*
- *movements in bedding and founding materials due to changes in moisture content;*
- *restraints caused by bends, manholes and pipelines passing through structures; and*

- *pressure due to the growth of tree roots.*

It is preferable to avoid or eliminate the causes of these loads rather than attempt to resist them. When not possible, particular attention must be paid to pipe joints and the interfaces between pipes and other structures, such as manholes, to ensure sufficient flexibility (Drawing LD-2, SANS 1200LD:1983 (SABS, 1983b)).

The most widely used method of estimating external loads on a buried pipeline was pioneered by Marston, Spangler and Schlick at Iowa State University in the US and is generally termed the 'Marston' or 'computed load' method. It is convenient to classify the various types of installation conditions in order to write specialized forms of the general Marston equation as shown diagrammatically in **Figure 11.1**.

Marston theory states that the load on a buried pipe is equal to the weight of the prism of soil directly over it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prism of soil (ASCE, 1982). The general form of the Marston's equation is:

$$W = CwB^2 \quad \text{...(11.1)}$$

where:

W = vertical load per unit length acting on the sewer pipe due to gravity soil loads (kN/m)

W = unit weight of soil (kN/m³)

B = trench width or sewer pipe width depending on installation conditions (m)

C = dimensionless coefficient that measures the effect of the following variables:

i) The ratio of the height of fill to width of trench or sewer pipe

ii) Shearing forces between interior and adjacent soil prisms

iii) The direction and amount of relative settlement between interior and adjacent soil prisms for embankment conditions

11.3 Rigid conduits

11.3.1 Gravity earth loads on buried rigid pipes

The calculation of gravity earth loads on buried conduits from first order principles is very complex. Specialist literature and SANS 10102-1:2005 (SABS, 2005c) and SANS 10102-2:2011 (SABS, 2011) should be used as reference material. However, the most important factors for establishing gravity earth loads on buried conduits are (PIPES, 2009):

- *the installation method;*
- *the fill height over conduit;*
- *the backfill density; and*
- *the trench or external conduit width.*

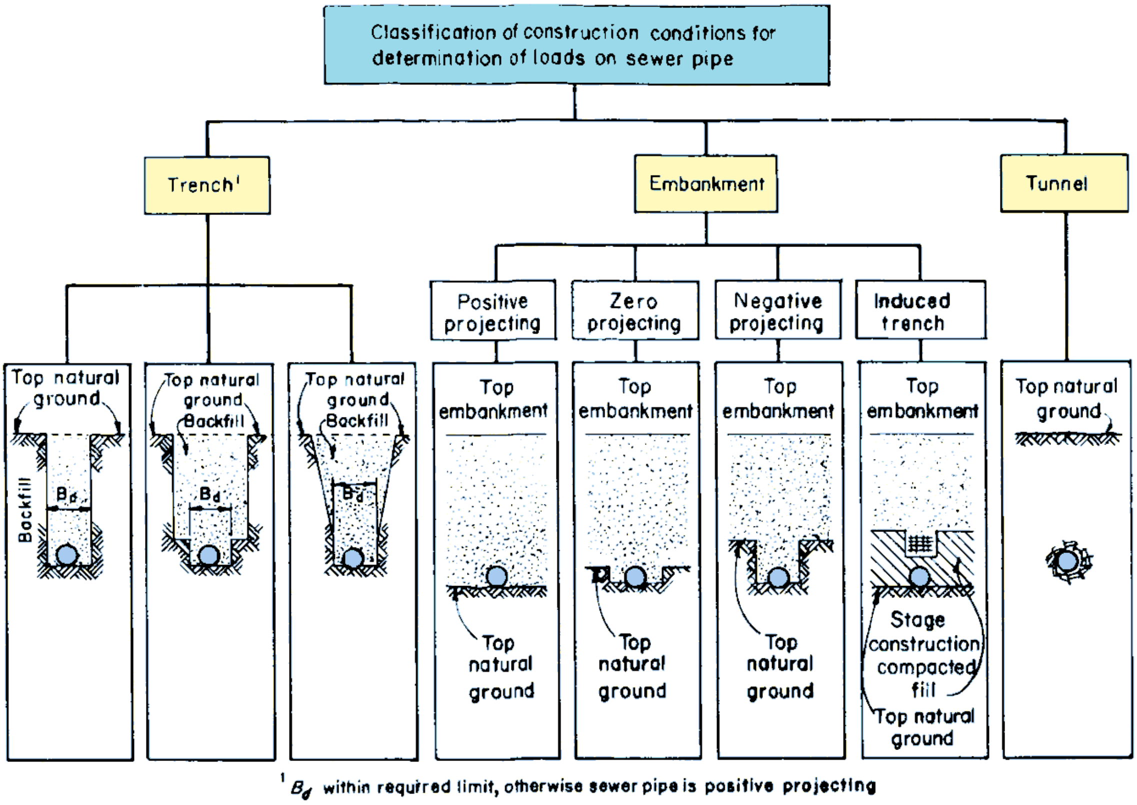

Figure 11.1: Classification of construction conditions (ASCE, 1982)

The basic installation types and corresponding loading conditions are depicted in **Figure 11.1**. These are defined by whether the frictional forces that develop between the column of soil on top of the conduit and the adjacent soil reduce or increase the load on the conduit. The geostatic or prism load is a useful concept to understand the corresponding loading conditions. It is the mass of soil directly above the conduit, assuming there is no friction between this column of material and the columns of soil on either side of the conduit. These loading conditions are illustrated in **Figure 11.2**.

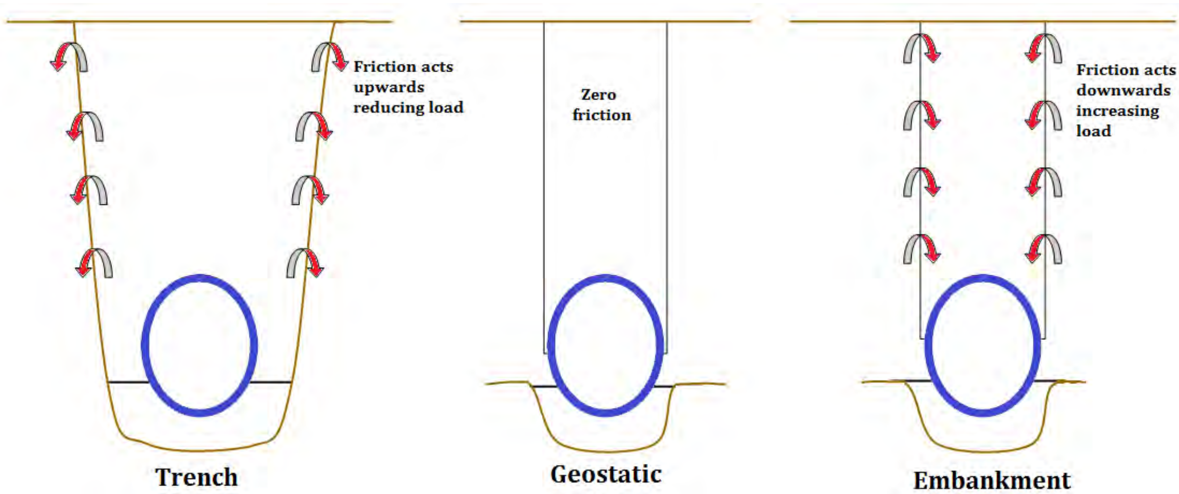

Figure 11.2: Comparison of trench, geostatic and embankment loading (PIPES, 2009)

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The geostatic load has a value between the trench and embankment load and is calculated with Equation (11.1), which is the basis of earth loading equations for the other conditions. Equation (11.1) can be defined for earth loading as follows (Equation (11.2)):

$$W_E = \gamma H B^2 \qquad \text{...(11.2)}$$

where:

W_E = vertical load per unit length acting on the sewer pipe due to gravity soil loads, i.e. load of fill material (kN/m)
γ = unit weight of soil (kN/m³)
B = trench width or sewer pipe width (outside diameter) depending on installation conditions (m)
H = fill height over pipe (m)

11.3.2 Trench conditions

In trench installations upward frictional forces develop between the column of earth in the trench and the trench walls which reduce the load that the conduit has to withstand (**Figure 11.2**). This results in the load on the conduit being less than the mass of the material in the trench above it. The load on the conduit is calculated with Equation (11.3).

$$W_T = \gamma C_T B_T^{\ 2} \qquad \text{...(11.3)}$$

where:

W_T = vertical load per unit length acting on the sewer pipe due to gravity soil loads, i.e. load of fill material (kN/m)
γ = unit weight of soil (kN/m³)
B_T = trench width on top of conduit (m)
C_T = coefficient that is function of fill material, trench width and fill height

The importance of the trench width B_T should be emphasized. The wider the trench width the higher the load on the conduit will be and thus the width should be kept to a practical minimum. For very wide trenches the load on the conduit will be the same as the embankment load. When the fill height over a pipe exceeds 10 times its outside diameter full arching will take place and any further increases in fill will not increase the load (PIPES, 2009). This maximum load can be calculated from (Equations (11.4) and (11.5)):

$$W_T = 2{,}63\gamma B_T^{\ 2} \text{ in sandy conditions} \qquad \text{...(11.4)}$$

$$W_T = 3{,}84\gamma B_T^{\ 2} \text{ in clayey conditions} \qquad \text{...(11.5)}$$

Trench loading on conduits where the trench widths are specified is presented in **Table 11.1**.

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Table 11.1: Loads on any conduit in kN/m for given trench widths (PIPES, 2009)

Trench width (m)	Height of backfill above top of pipe (m)										
	0.6	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0
0.75	8	13	18	22	25	28	30	32	36	38	39
1.00	11	18	25	31	37	42	46	50	56	61	64
1.25	14	23	32	41	49	56	62	68	78	86	92
1.50	17	28	40	51	61	70	79	87	100	112	122
1.75	20	32	47	61	73	85	95	105	123	139	152
2.00	23	38	55	70	85	99	112	125	147	167	184
2.50	29	47	69	90	110	129	147	164	195	223	249
3.00	35	57	84	110	135	159	181	203	243	281	315
3.50	41	67	99	130	160	188	216	242	292	339	382
4.00	47	77	114	150	185	218	250	282	342	397	450
5.00	59	97	144	190	234	278	320	361	440	515	587

Note:

1. This table has been set up for non-cohesive soil; gravel or sand; density = 20 kN/m^3 and K_μ = 0.19.
2. SANS 1200 DB:1989 recommends trench widths (SABS, 1989).
3. If the soil unit weight is known, the loads from the table may be adjusted as follows: Load on pipe = load from table x unit weight of soil / 20.
4. This procedure is valid only if the soil properties other than unit weight do not change.

11.3.3 Embankment conditions

As depicted in **Figure 11.1**, the embankment condition is defined as the conduit being installed at ground level and covered with fill material. Usually the fill material surrounding the conduit is homogeneous and compaction is uniform. As shown in **Figure 11.2** with an embankment installation the frictional forces that develop between the column of soil directly above the conduit and the columns of soil adjacent to the conduit, act downwards and increase the load on the conduit (PIPES, 2009). In other words, the load on the conduit is greater than simply the mass of the fill material directly above it. The load on a conduit is calculated using Equation (11.6):

$$W_E = \gamma C_E B_C^{\,2} \qquad \text{...(11.6)}$$

where:

W_E = vertical load per unit length acting on the sewer pipe (kN/m)
γ = unit weight of fill material (kN/m^3)
B_C = outside diameter of conduit (m)
C_E = coefficient which is a function of fill material, conduit outside width, fill height, projection ratio, and founding conditions

The projection ratio is a measure of the proportion of the conduit over which lateral earth pressure is effective. It is calculated using the equation $p = x / B_C$, where x is the height that the conduit projects above or below the natural ground level.

The founding material under the conduit could yield and partially reduce the frictional forces and hence the load. The settlement ratio, r_s, is a measure of the settlement of the founding material under the conduit and is given in **Table 11.2** (PIPES, 2009).

Table 11.2: Settlement ratios (PIPES, 2009)

Material type	Rock	Unyielding soil	Normal soil	Yielding soil
Settlement ratio, r_s	1.0	1.0	0.7	0.3

The different types of embankment condition, illustrated in **Figure 11.3**, are defined by the pipe projection relative to the original ground level.

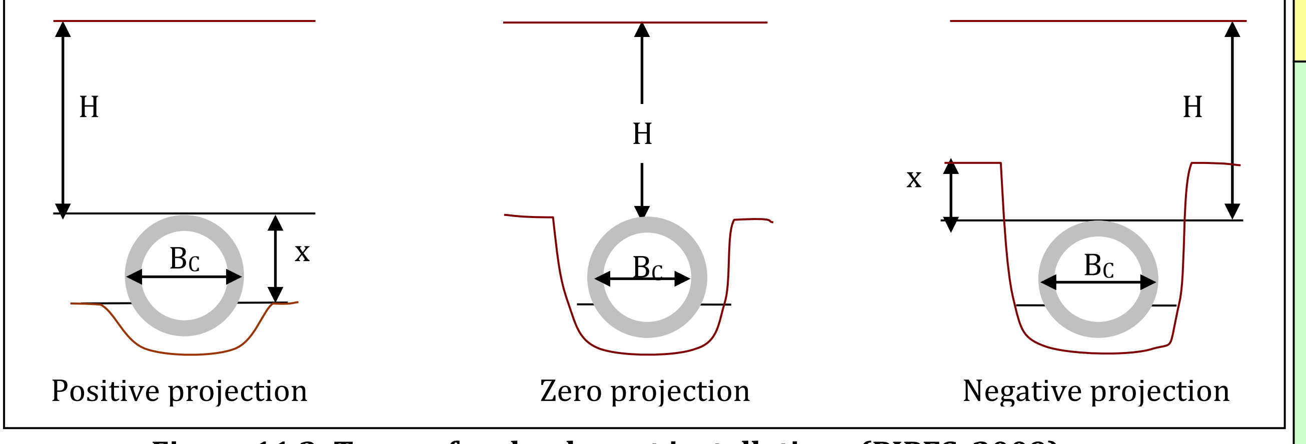


Figure 11.3: Types of embankment installations (PIPES, 2009)

For all practical purposes the earth loading from a positive projecting installation will have a maximum value when the pr_s ratio has a value of 1.0. The earth loading can be calculated from Equations (11.7) and (11.8) where the relationship between load and fill height is linear.

$$W_E = 1{,}69\ \gamma\ B_C H \text{ in sandy conditions} \qquad ...(11.7)$$

$$W_E = 1{,}54\ \gamma\ B_C H \text{ in clayey conditions} \qquad ...(11.8)$$

Equations (11.7) and (11.8) provide upper limits, the smaller of the load calculated from them or Equations (11.4) or (11.5) should be used (PIPES, 2009).

11.3.4 Superimposed live loads

To determine the required supporting strength of rigid pipes installed under asphalts, other flexible pavements, or relatively shallow earth cover, it is necessary to evaluate the effect of live loads, such as highway truck loads, in addition to earth loads.

Where a sewer is installed under transportation routes, the axle spacing and loads, the wheel spacing, loads and contact areas of the vehicles using them, and the type of riding surface and height of fill over the conduits should be determined (PIPES, 2009).

If a rigid pavement or a thick flexible pavement designed for heavy-duty traffic is provided with a sufficient buffer between the pipe and pavement, then the live load transmitted through the pavement to the buried pipe is usually negligible at any depth. If sewer pipe is within the heavy-duty traffic highway right-of-way, but not under the pavement structure, then such pipe should be analysed for the effect of live load transmission from an un-surfaced roadway, because of the possibility of trucks leaving the pavement.

The following is an extract from the *Design Manual for Concrete Pipe Outfall Sewers* (PIPES, 2009) detailing the calculation of the superimposed live loads due to traffic:

Two design vehicles have been considered: a typical highway vehicle that has two sets of tandem axles and the NB36 vehicle, which is associated with abnormal loads on national highways as described in bridge loading code TMH7(5). The legal loads due to typical highway vehicles are not suitable for design, as these vehicles may be overloaded or involved in an accident, in which case the loads may be far greater. The design loads as described in TMH7 for the design of structures under major roads are:

- *Normal loading (NA);*
- *Abnormal loading (NB); and*
- *Super loading (NC).*

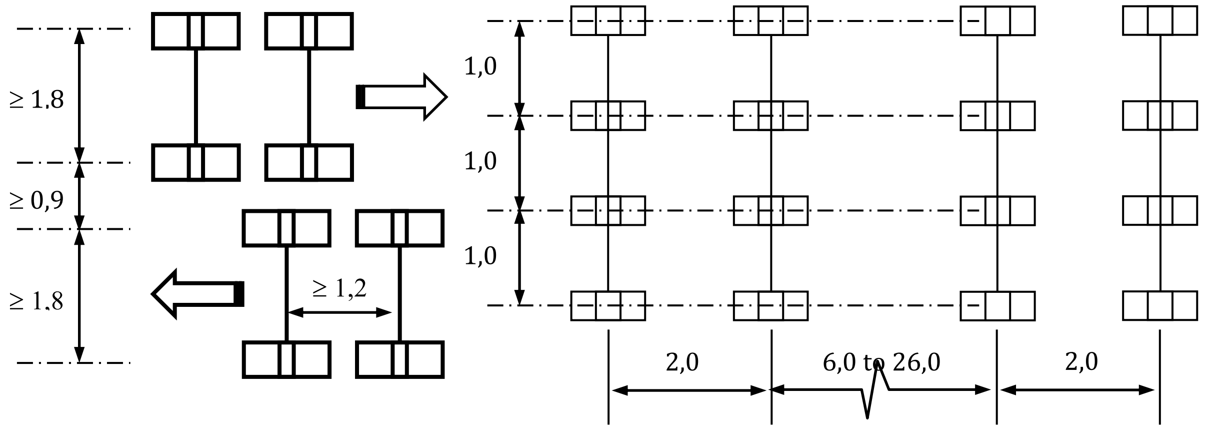
The NA loading for culverts consists of two 100 kN point loads. The NB loading is for a vehicle with multiple wheels. For the NB vehicle, 1 unit = 2.5 kN per wheel = 10 kN per axle and = 40 kN per vehicle. In the case of the NB36 vehicle, there are 36 units = 90 kN per wheel = 360 kN per axle and = 720 kN per vehicle. The NC loading is a uniformly distributed load of 30 kN/m² over a large area. The NB36 load is usually the critical one in the case of buried pipes. TMH7 permits the use of an equivalent point load for this that is dependent upon the outside width and length of the conduit.

$$Q_b = 1{,}25\left(90 + 12L_s^{1{,}8}\right) \quad \text{...(11.9)}$$

where:

Q_b = *equivalent point load (kN)*
L_s = *effective span of conduit (m)*

The values for the typical legal vehicle should only be used for the design of conduits in areas outside public jurisdiction. The most severe loading will occur when two vehicles pass or are parked next to each other. ***Figure 11.4*** *illustrates such a wheel configuration.*



40 kN wheel loads – legal limit NB36 loading – 90 kN wheel loads

Figure 11.4: Traffic loads on roads

An allowance for impact should be made when considering the effect of these loads on buried conduits. In the case of the typical highway vehicle this is usually taken as 1.15. Where greater impact is expected due to a combination of high speed, rough surface and hard suspension, an impact factor up to 1.4 may be used. The effective contact area for each wheel is taken as 0.2 m x 0.5 m in the direction of travel and transverse to this.

*The loads on pipes resulting from 40 kN and NB36 wheel loads with the configurations shown in **Figure 11.4** are presented in **Tables 11.3** and **11.4,** respectively. The loads presented in **Table 11.3** have been calculated by distributing the loads over a pipe at 45° through the fill from the perimeter of the loaded area and assuming a uniform loading intensity on the horizontal plane over the pipe crown. **Table 11.3** can be used for any wheel load (P) provided the wheel arrangement is the same as that of the legal vehicle and the load is multiplied by P/40. Although pipes can be placed at very low fill heights it is unadvisable to build sewers with less than 600 mm of cover. Refer to the notes below the table when the fill height is less than 600 mm or less than half the pipe diameter.*

Table 11.3: Loads in kN/m on buried conduit from group of 40 kN wheels

Diameter (mm)		Fill height over pipes (m)										
Inside	Outside	0.6	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0
300	345	8.1	4.78	2.8	1.8	1.3	1.0	0.7	0.6	0.4	0.3	0.2
375	431	10.2	5.97	3.5	2.3	1.6	1.2	0.9	0.7	0.5	0.3	0.2
450	518	12.2	7.16	4.2	2.8	2.0	1.5	1.1	0.9	0.6	0.4	0.3
525	604	14.2	8.36	4.9	3.3	2.3	1.7	13.3	1.0	0.7	0.5	0.4
600	690	16.3	9.55	5.7	3.7	2.7	2.0	1.5	1.2	0.8	0.6	0.4
750	863	20.4	11.9	7.1	4.7	3.3	2.5	1.9	1.5	1.0	0.7	0.5
900	1 035	24.5	14.3	8.5	5.6	4.0	3.0	2.3	1.8	1.2	0.9	0.7
1 050	1 208	28.5	16.7	9.9	6.6	4.7	3.5	2.7	2.1	1.4	1.0	0.8
1 200	1 380	32.6	19.1	11.4	7.5	5.3	4.0	3.1	2.5	1.7	1.2	0.9
1 350	1 620	38.3	22.4	13.3	8.8	6.3	4.7	3.6	2.9	1.9	1.4	1.0
1 500	1 800	42.6	24.9	14.8	9.8	7.0	5.2	4.0	3.2	2.2	1.6	1.2

Notes:
1. *No impact factor has been included.*
2. *Impact must be considered in the case of low fills (<diameter of pipe).*
3. *The tables do not apply to pipes on concrete bedding.*
4. *Where the cover is less than half the outside pipe diameter, the live load bedding factor must be reduced and precautions such as concrete encasement may be necessary.*

Table 11.4: Loads in kN/m on buried pipes from NB36 group of 90 kN wheels

Diameter (mm)		Fill height over pipes (m)											NB36 point load
Inside	Outside	0.6	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	
300	345	26	12	7	4	3	2	1	1	1	1	0	114
375	431	31	15	8	5	3	2	2	1	1	1	0	115
450	518	35	17	10	6	4	3	2	2	1	1	1	116
525	604	39	19	11	7	5	3	2	2	1	1	1	117
600	690	43	22	12	8	5	4	3	2	1	1	1	118
750	863	49	25	15	9	6	5	4	3	2	1	1	121
900	1 035	55	29	18	11	8	6	4	3	2	2	1	125
1 050	1 208	60	33	21	13	9	7	5	4	3	2	1	129
1 200	1 380	64	36	24	15	10	8	6	5	3	2	2	133
1 350	1 620	67	40	28	18	12	9	7	5	4	3	2	138
1 500	1 800	67	43	31	20	14	10	8	6	4	3	2	144

The NB36 vehicle travels slowly and thus impact does not need to be considered.

11.4 Flexible conduits

Under vertical earth loads, buried flexible conduits deflect downward vertically and outward horizontally, thereby mobilizing passive lateral soil support for the pipe, which in turn precludes significant further downward deflection (Nayyar, 1999). Thus the pipe and surrounding soil interact and behave as a structural system. In this system, pipe deflection is controlled more by soil stiffness than by pipe flexural stiffness, and the soil arching characteristics bear great influence on the system's load-carrying capacity. Flexible pipe properly installed in stable soils can resist very substantial loads.

Since the pipe and the soil interact, design and installation of buried flexible pipe must always consider both the pipe and the soil around it. If a designer allows different pipes in a specification, the suitability of the backfill and installation specifications should be evaluated for each type of pipe.

One of the advantages of flexible pipe is that the quality of installation can readily be checked via a deflection test after installation is complete. A particular benefit of most thermoplastic pipe is its high strain capacity, which allows it to deform considerably and thereby generate further soil support.

To take economic advantage of this benefit, many of the newer larger-diameter thermoplastic pipes are offered with relatively low wall stiffness, which requires that careful attention be given to proper design and installation in order to ensure durable and stable performance. Since the soil and the pipe must always work together to constitute a pipe-soil system, the designer has to consider both when evaluating alternate pipe materials.

11.4.1 Earth loads on buried flexible pipes

The load acting on a buried pipe consists of *earth loads* and *superimposed live loads*. The earth load is the permanent load from the weight of soil and pavement above the pipe and sometimes from any surcharge loads applied at the ground surface. Surcharge loads may, or may not, be permanent. Surface-applied wheel loads are called *live loads*.

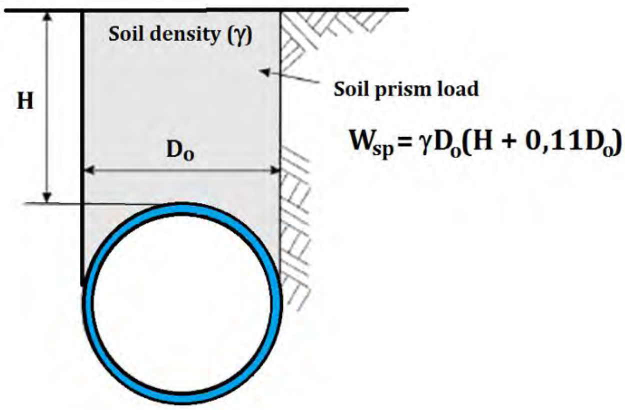


Figure 11.5: Soil prism load

Earth load is measured as the hoop thrust at the spring line of the pipe, and is often characterized as a function of the soil prism load, which is the weight of earth directly over the pipe (**Figure 11.5**). The weight of the soil prism is modified by the vertical arching factor (VAF) to represent the effects of pipe-soil interaction. This is expressed in Equations (11.10) and (11.11).

$$W_{sp} = \gamma D_o\left(H + 0,11D_o\right) \qquad \text{...(11.10)}$$

where:

W_{sp} = soil prism load (kN/m)
γ = soil unit weight (kN/m^3)
D_o = outside diameter of pipe (m)
H = depth of fill over top of pipe (m)

$$W_p = \text{VAF}\left(W_{sp}\right) \qquad \text{...(11.11)}$$

where:

VAF = vertical arching factor
W_p = effective soil load (kN/m)

❸

Flexible pipes are often designed with a vertical arching factor of 1.0 (for reference purposes, rigid pipes installed in embankment conditions are often designed for an arching factor of about 1.4). While VAF = 1.0 is considered conservative for flexible pipe, recent research (Nayyar, 1999) has shown that the VAF can be much lower for some thermoplastic pipes with low moduli of elasticity and low cross-sectional area.

Similar to rigid pipes, the Marston theory for determining the loads on a buried flexible pipe (ASCE, 1982) can be used. This solution assumes that some of the backfill load is carried by the flexible pipe; some is carried by the trench walls; and some by the backfill at the sides of the pipe. Thus in narrow trench installations, the loads are substantially less than the soil prism load. This is consistent with the trench-load theory for rigid pipe.

When subjected to the calculated hoop compression forces, a pipe must meet the following criteria:

- *Wall thrust:* The wall stress due to the hoop-compression forces must be less than the limiting strength of the material. Short-term strength should be used for evaluating short-term load conditions, and long-term strength should be used for evaluating long-term load conditions.

- *General buckling:* If the hoop-compression forces are sufficiently high, the pipe can buckle. This is a function of the pipe flexural stiffness and the soil stiffness. The expression most often used to evaluate this condition is given in Equations (11.12) to (11.14):

$$N = \left(\frac{1}{FS}\right) D_o \left[32 R_w B' E' \left(\frac{EI}{D_m^3}\right)\right]^{0,5} \quad \text{...(11.12)}$$

where:

N	=	allowable wall thrust (kN/m)
FS	=	factor of safety, often taken as 2.5 to 3.0
D_m	=	mean diameter of pipe (m)
E'	=	modulus of soil reaction (MPa)
E	=	modulus of elasticity of the pipe material (MPa)
R_w	=	coefficient for depth of groundwater above top of pipe

$$R_w = 1 - 0,33\left(\frac{H_w}{H}\right) \quad \text{...(11.13)}$$

where:

H_w	=	depth of groundwater above top of pipe ($<H$) (m)
H	=	depth of earth cover over top of pipe (m)
B'	=	coefficient for uniformity of pipe support

$$B' = \frac{1}{1 + e^{-0,2133H}} \quad \text{...(11.14)}$$

Note that the constrained soil modulus M_S may be used as a direct substitute for the modulus of soil reaction, *E*'. For trench installations, a method of computing a 'composite' value of *E*' that considers the stiffness of both the backfill soil and the native soil in the trench wall is presented in the *AWWA Manual of Practice M45* (AWWA, 2005).

11.4.2 Deflection of flexible pipes

A flexible pipe is by definition a pipe which will deflect when it is subjected to external loads as opposed to a rigid pipe which carries all external loads by itself. The degree of deflection of a flexible pipe will depend on the pipe stiffness, support from the surrounding soil and external loads. Most methods for calculating deflection in buried flexible pipes are based on the so-called Sprangler formula which states:

$$\text{Deflection (\%)} = \frac{\text{vertical load on pipe}}{\text{pipe stifness} + \text{soil stifness}} \qquad ...(11.15)$$

As noted, buried flexible pipes deflect (decrease vertical diameter and increase the horizontal diameter) when subjected to earth and superimposed loads. The installation conditions must be designed to assure that the ultimately achieved deflections are within acceptable limits. This will preserve the serviceability of the pipeline and assure that material stress or strain limits are not exceeded. Generally, deflection limits for flexible pipe are limited to between 5% and 10% decrease in the vertical diameter.

By deflecting vertically, the pipe actually sheds some of the vertical load to the surrounding soil. The pipe deflects outwards at the sides at the same time as deflecting downwards under soil load. The lateral extension further consolidates the soil to improve its strength.

To determine that a pipe is suitable to be buried at a specific depth and under specific soil and load conditions it is necessary to calculate the pipe deflection under such conditions.

One of the better-known relationships, sometimes called the *Iowa formula*, was developed for flexible metal conduits at Iowa State University (Nayyar, 1999) as is given in Equation (11.16).

$$\Delta v = \frac{D_1 K W_{sp}}{\left(\frac{EI}{R^3} + 0{,}061E'\right)} \qquad ...(11.16)$$

where:

Δv	=	change in vertical diameter (m)
D_1	=	deflection lag factor to account for time effects (typically taken between 1 and 2.0)
K	=	a function of pipe loading angle and pipe bedding angle (0.083 to 0.110)
W_{SP}	=	soil prism load (MN/m)
EI/R^3	=	pipe flexural stiffness (MPa)
E'	=	modulus of soil reaction (MPa)

❸

The deflection lag factor accounts for change in load with time and is typically taken at a value of 1 to 2.0. The bedding factor accounts for the width of the bedding support and can vary from 0.083 to 0.110. The modulus of soil reaction E' is an empirical measure of the stiffness of the soil in resisting pipe deflection and must be obtained empirically by back-calculating it from measured pipe deflections. While E' is not a true soil property that can be evaluated by laboratory tests, recent work suggests that the constrained modulus of elasticity, M_s may be directly substituted for E'.

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12. *SewerAID*

When setting out to produce the *Waterborne Sanitation Design Guide* and the *Waterborne Sanitation Operation and Maintenance Guide* an additional aim was to provide an innovative education component in parallel to the guides. ***SewerAID*** is a useful design application tool that provides the readers/users of this guide with additional relevant information in the form of:

- Additional literature
- Drawings
- Photo gallery
- Movie clips
- Software

SewerAID, shown in **Figure 12.1**, provides additional valuable information to assist the designer in planning, design and implementation of the most appropriate waterborne sanitation system.

Wherever the icon appears in this report it is an indication that there is some relevant material on the accompanying ***SewerAID*** DVD. The aim is to enhance through other media forms the understanding of the specific concept.

Figure 12.1: ***SewerAID***

12.1 System requirements

SewerAID runs on any personal computer, but requires Windows XP/Vista/7, Windows N.T 4.0, or 2000. The program takes up no hard-disk space since it is run from the DVD. Installing the various design software programs will, however, utilize hard-disk space. The following hardware specification is recommended:

- Pentium IV or higher
- 1 GB RAM or more
- 16X DVD-ROM drive
- 1024x800 display, 24-bit colour or higher

12.2 The package

SewerAID is distributed on a DVD which contains all the files and drivers necessary to install and run the program. The *Waterborne Sanitation Design Guide* and the *Waterborne Sanitation Operation and Maintenance Guide* are available in hard-copy format although an electronic version is distributed with the distribution DVD in Adobe Acrobat format. The latest version of the Adobe Acrobat Reader is also included on the distribution DVD.

12.3 Running *SewerAID*

To run *SewerAID* from the distribution DVD:

- Insert the distribution DVD in the DVD-drive

- Click on Start ›Run and type "d:\SewerAID.pps" (if "d" is the DVD drive) or use the windows explorer to start *SewerAID* on the DVD by double-clicking the file.

- Follow the instruction on the screen

12.4 Disclaimer

The software programs were included for the convenience of its users. Neither the authors nor the Water Research Commission accept any liability of any kind for any results, interpretation thereof or any use made of the results obtained with these software programs. All users of these programs do so entirely at their own risk.

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Appendix A

Bedding details

Figure A1: Pipe bedding details (Ethekwini, 1987)

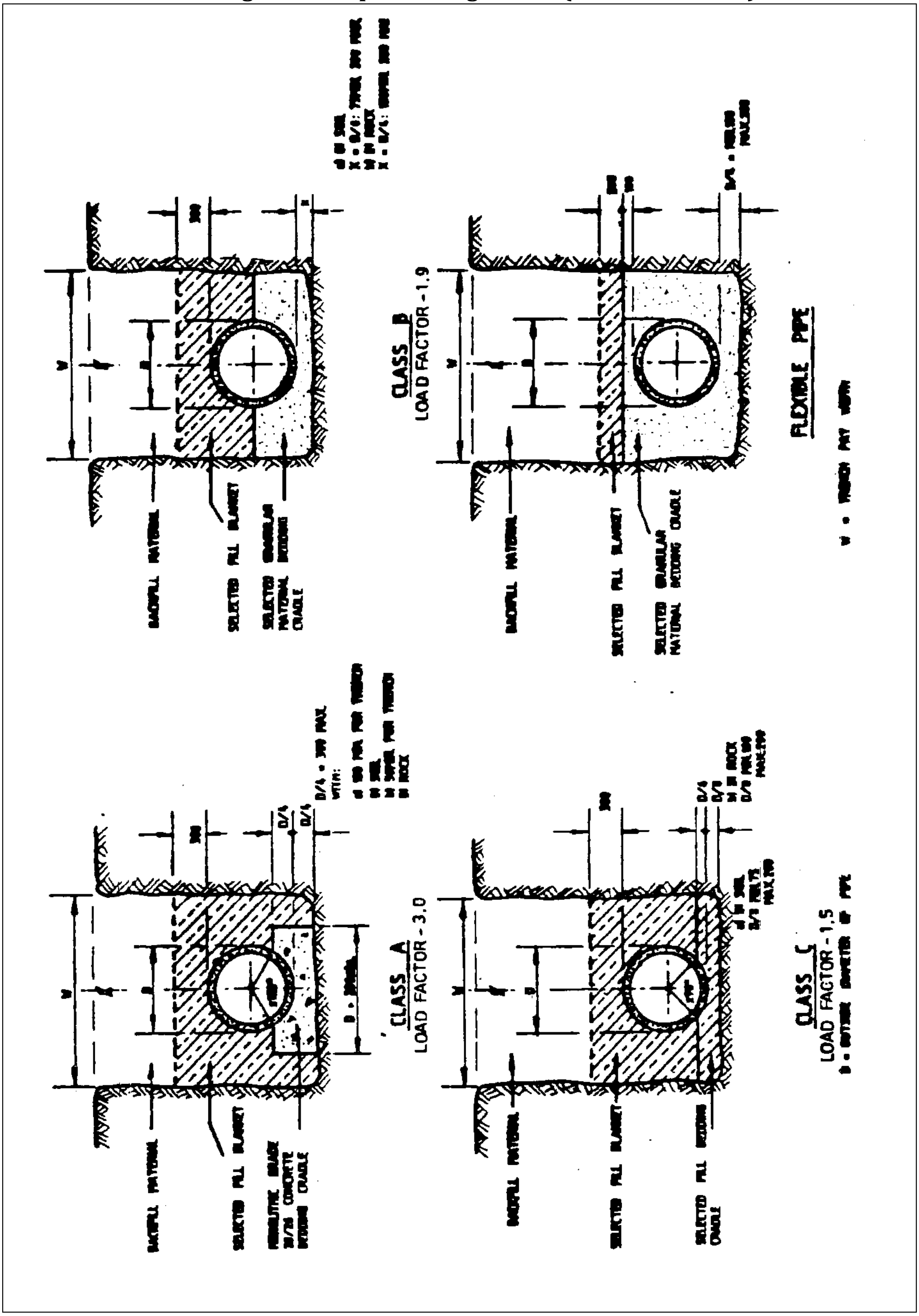
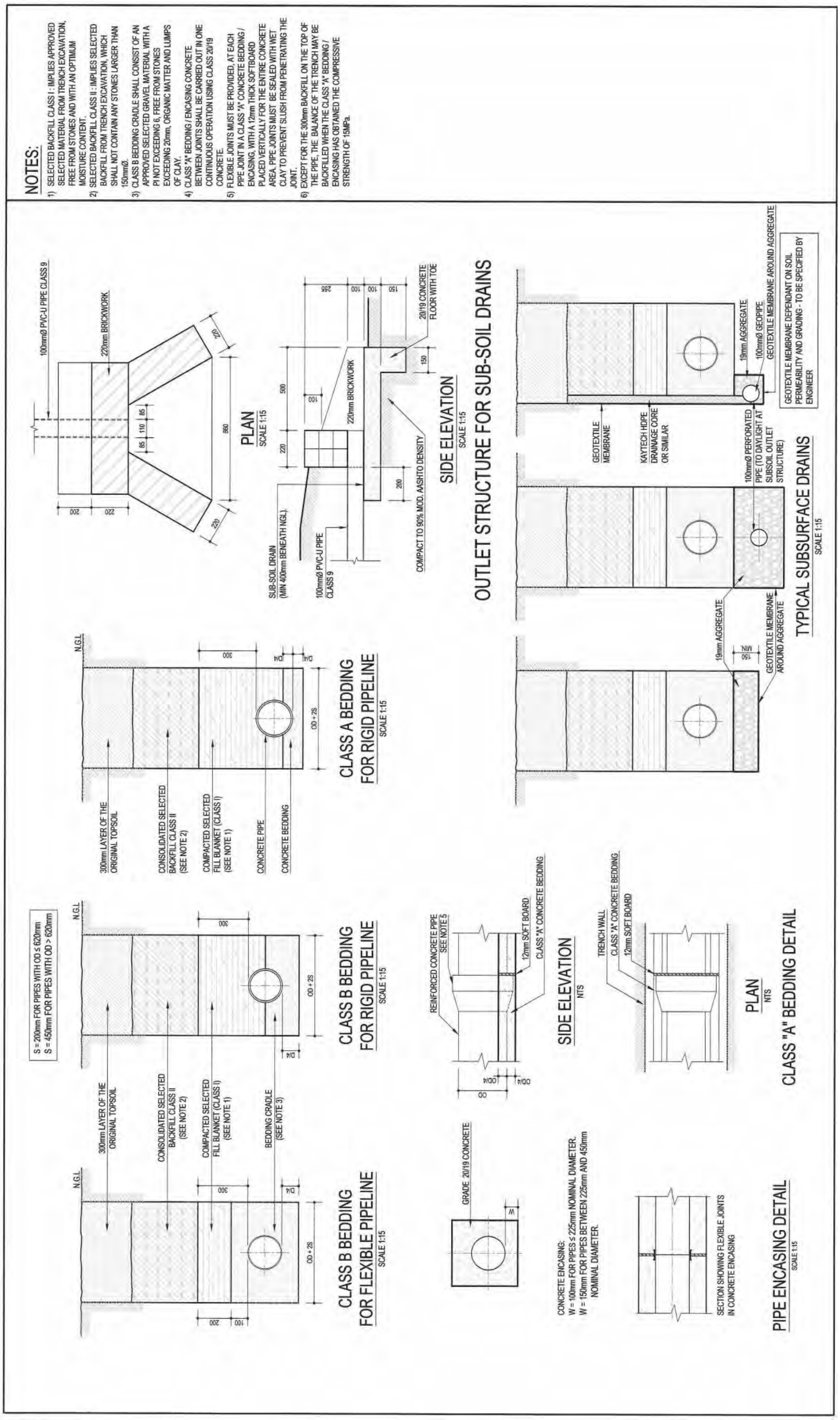


Figure A2: Pipe bedding details (City of Tshwane, 2007)



Appendix B

Standard drawings

Figure B1: Standard drawing: Manhole details for sewers up to 300 mm nominal diameter (City of Tshwane, 2007)

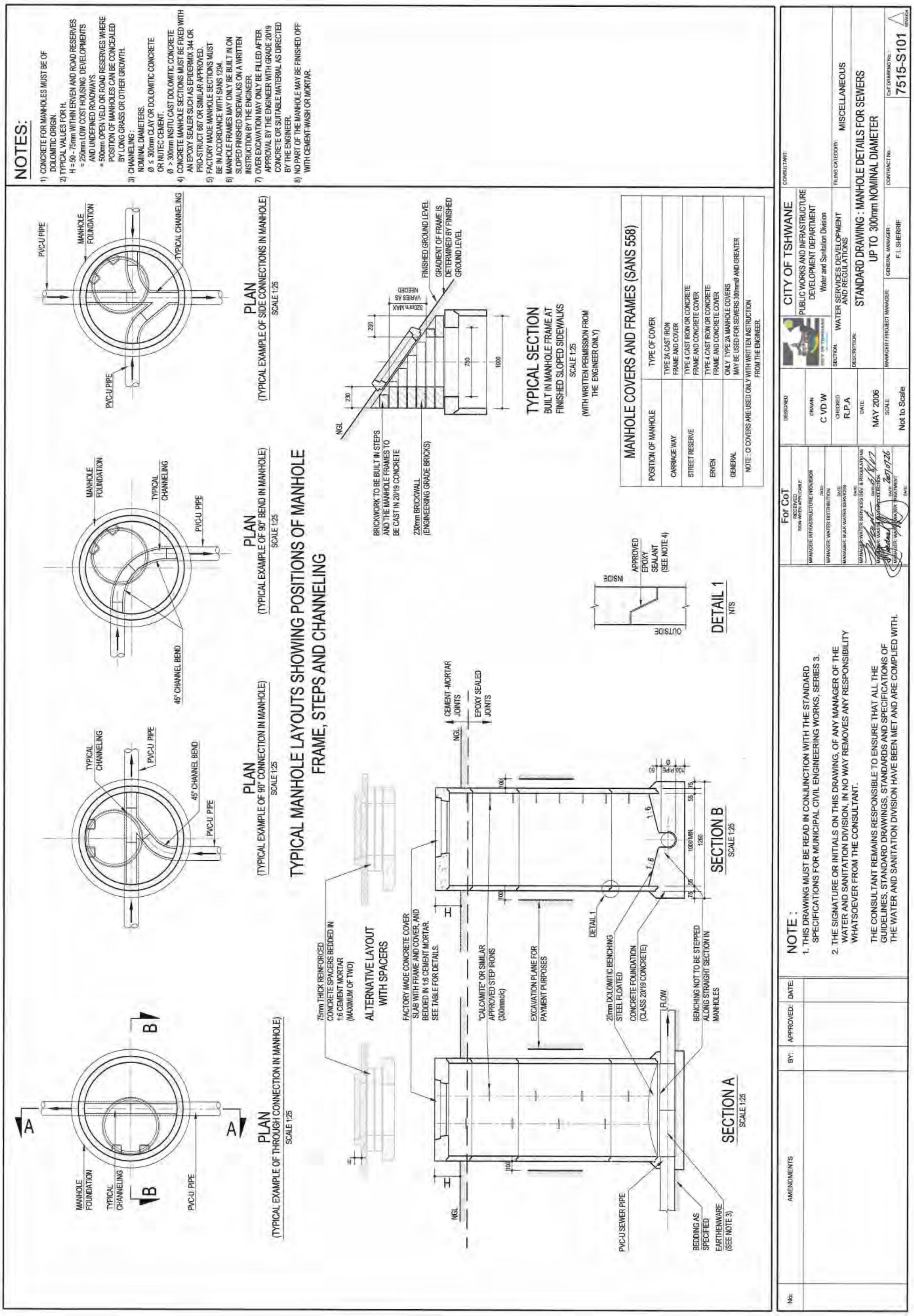


Figure B2: Standard drawing: Manhole details for sewers larger than 300 mm diameter (City of Tshwane, 2007)

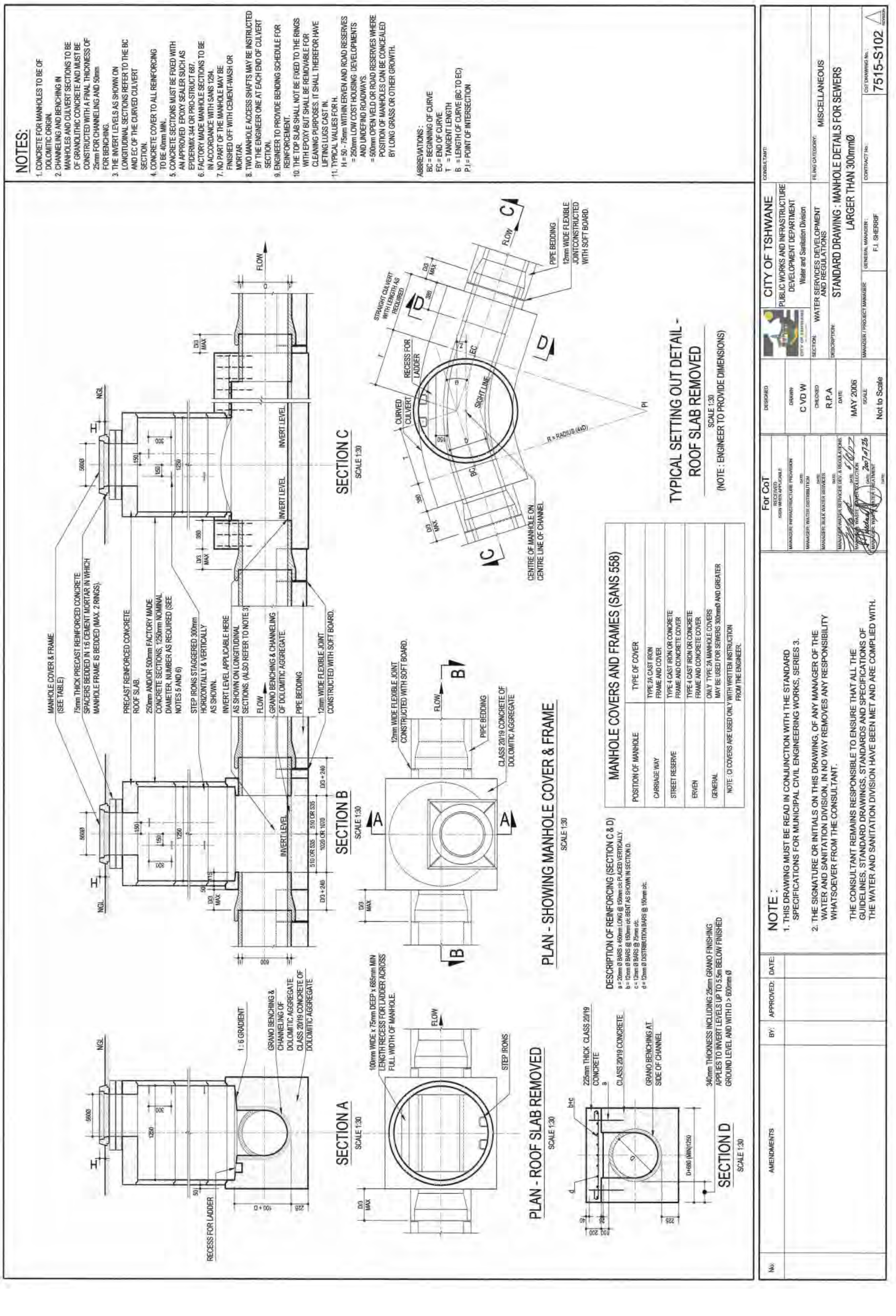


Figure B3: Standard drawing: Sewer pipe trenches and bedding (City of Tshwane, 2007)

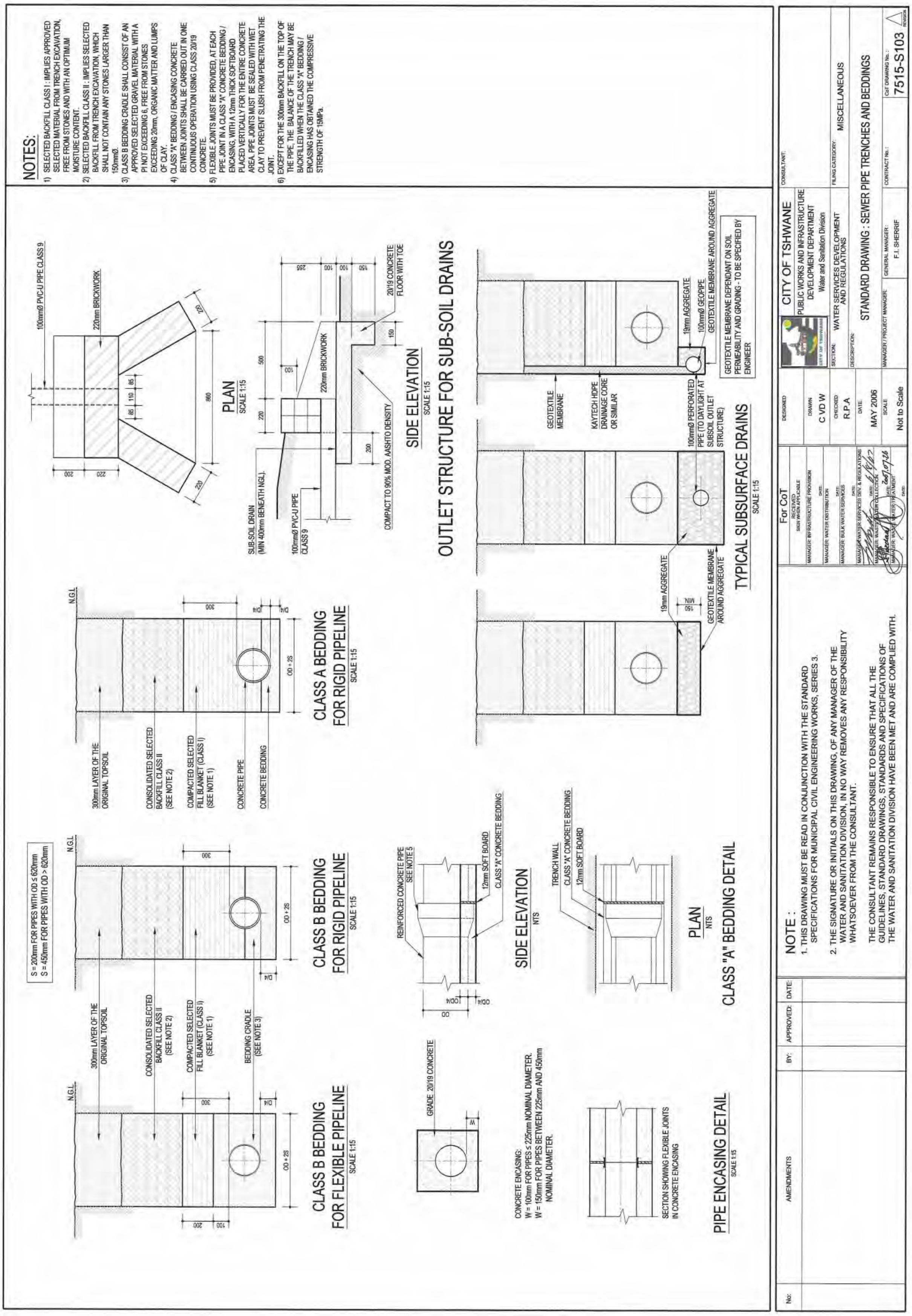


Figure B4: Standard drawing: Sewer house connection details (City of Tshwane, 2007)

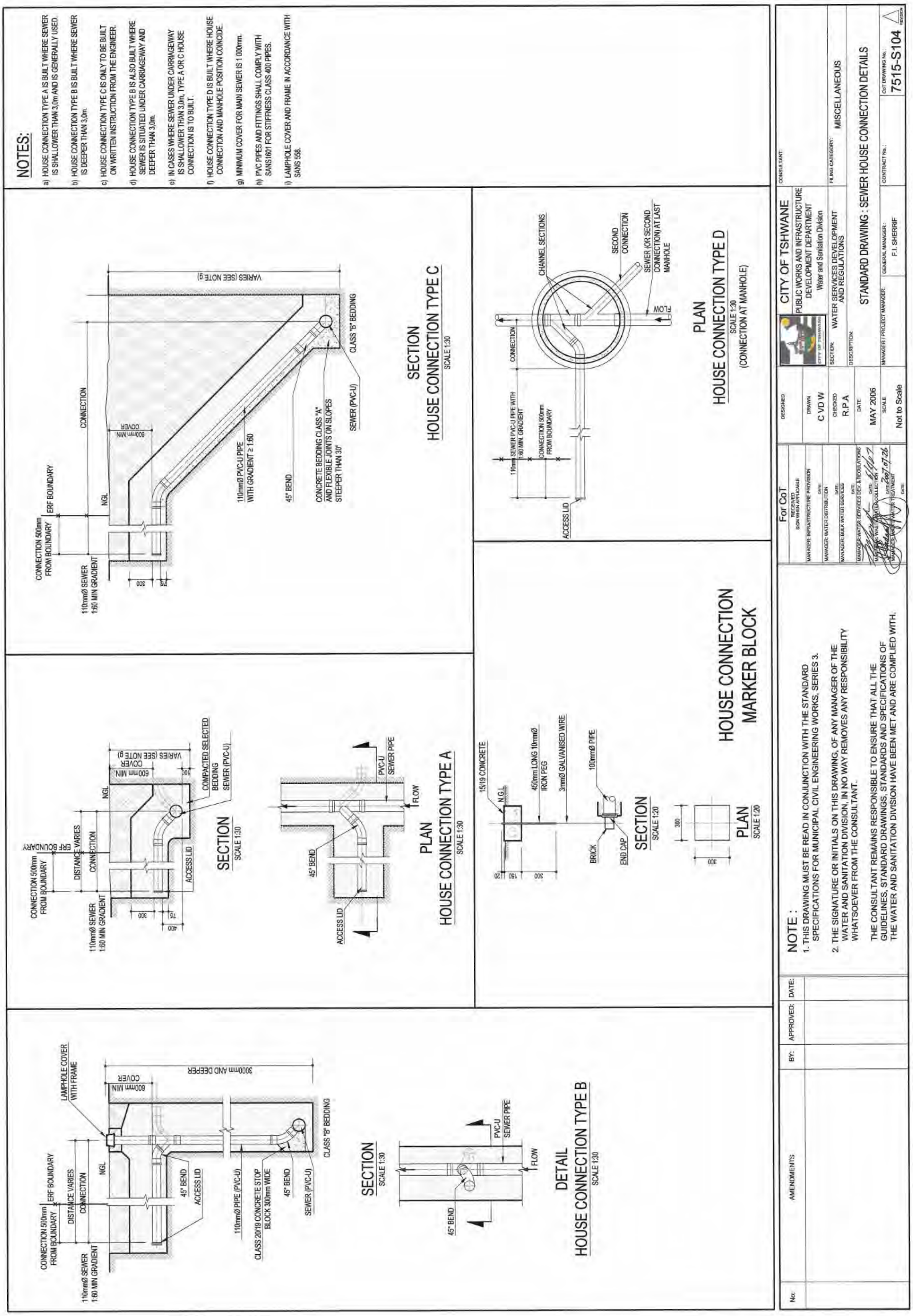


Figure B5: Standard drawing: Toilet structure (City of Tshwane, 2007)

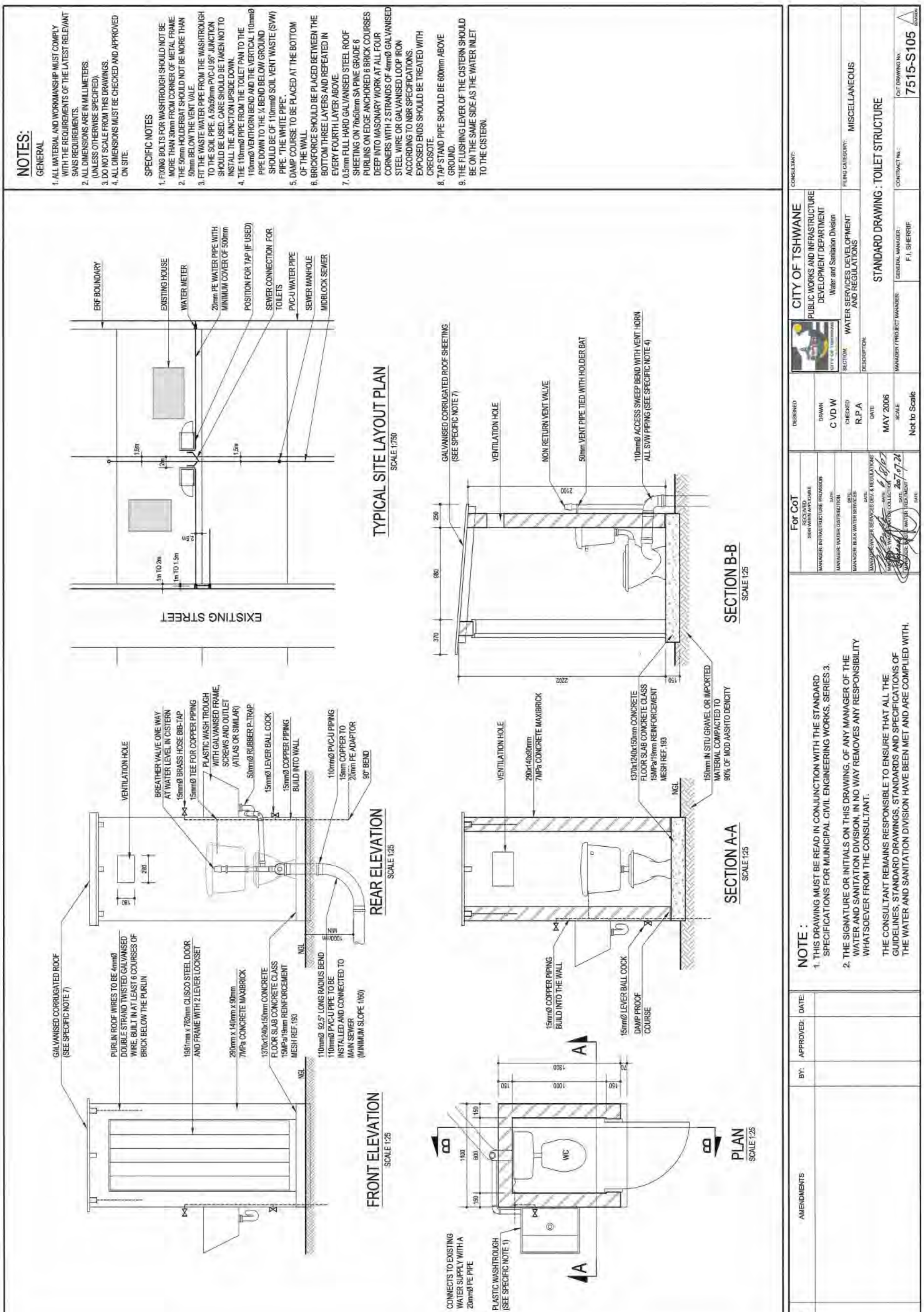


Figure B6: Standard drawing: Typical sewer layout for sub-divisions and lamphole detail (City of Tshwane, 2007)

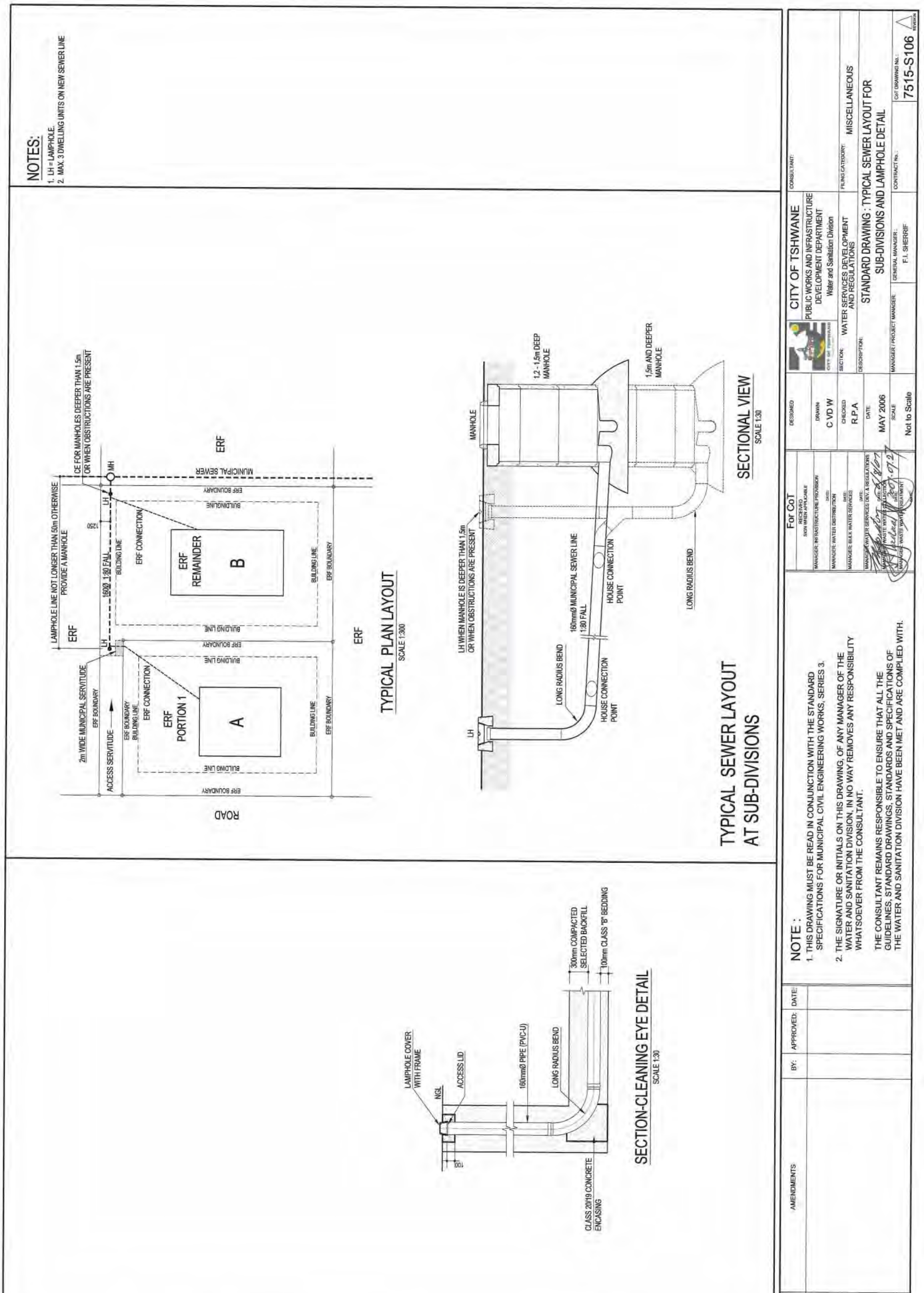


Figure B7: Drawing LD-3 – Brick Manholes for Sewers (SABS, 1982a)

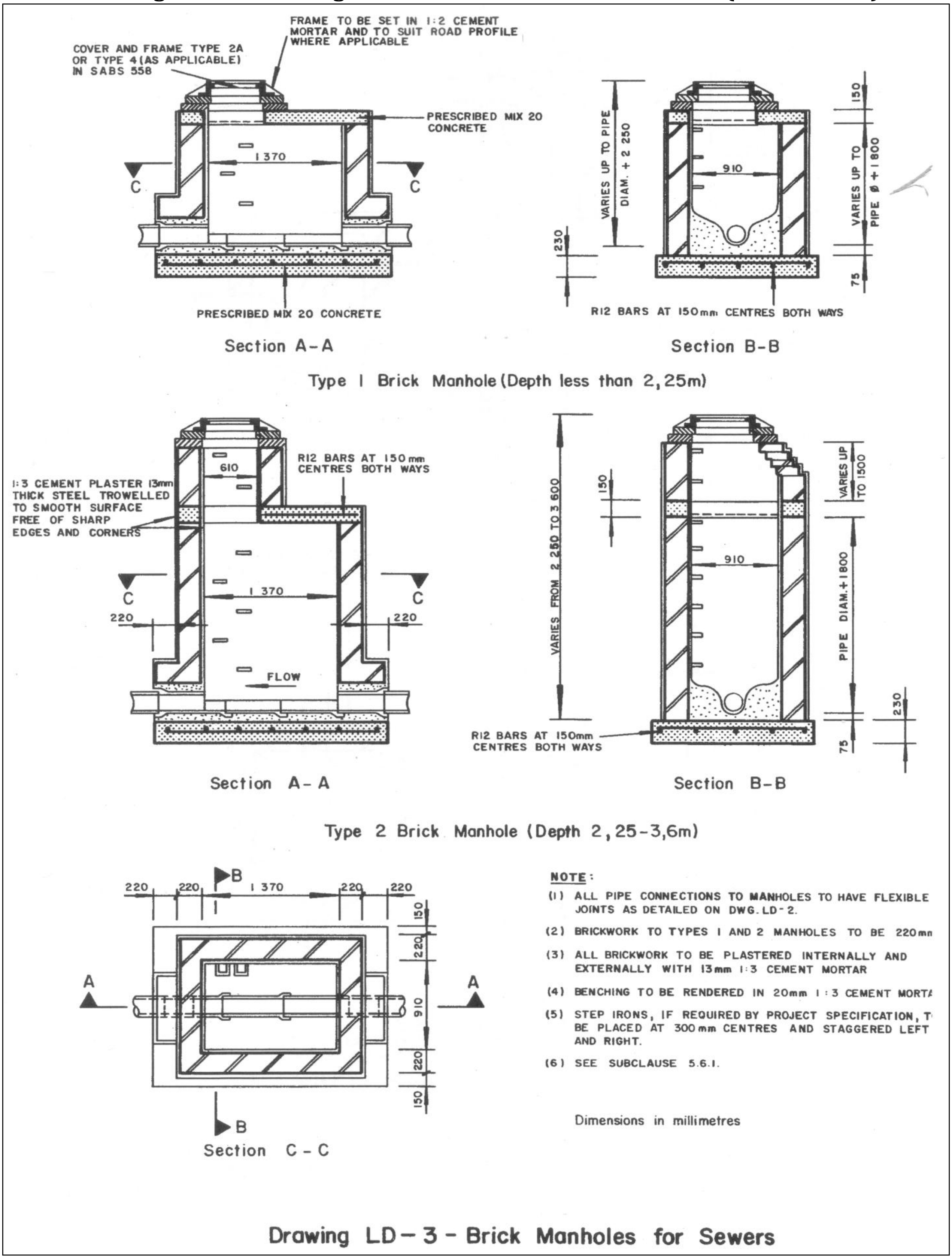
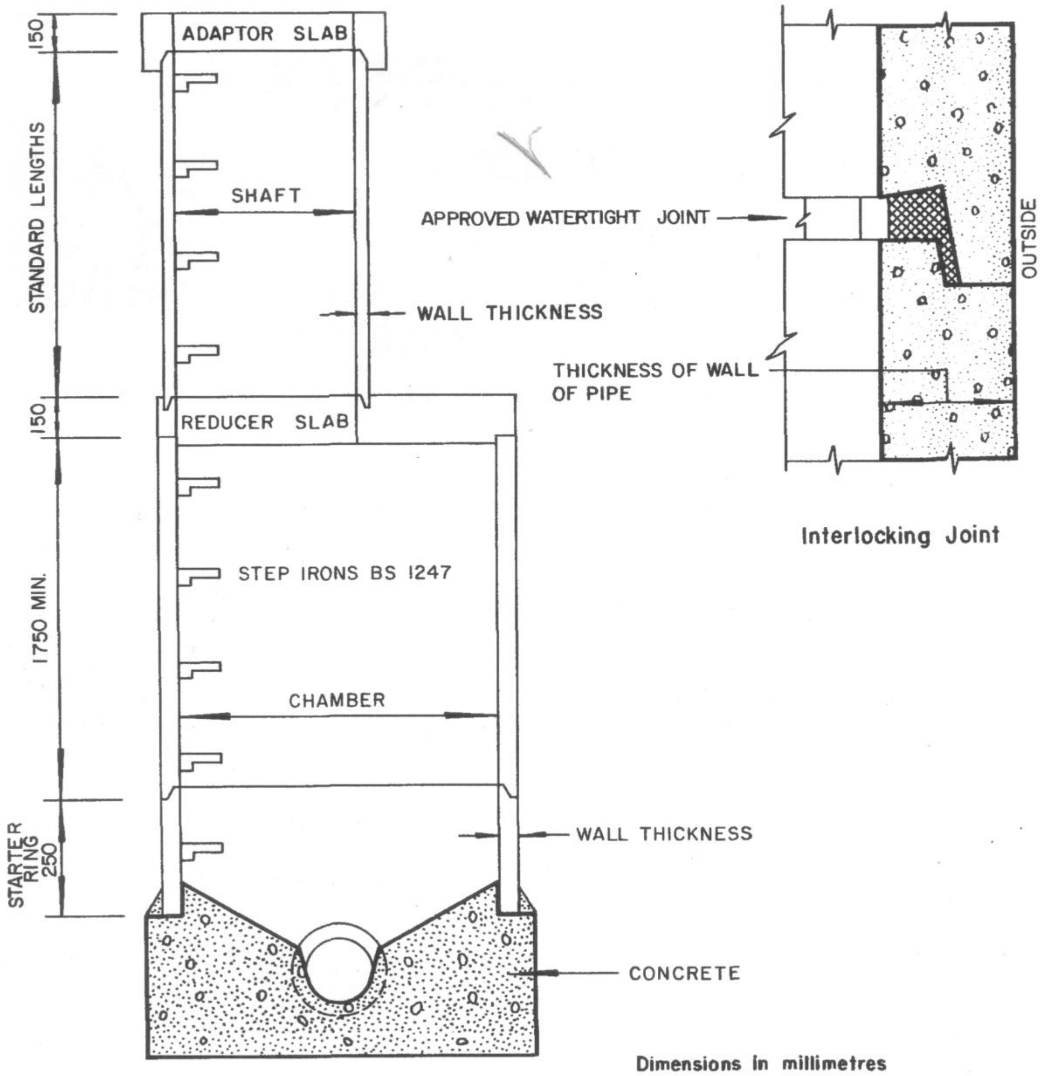


Figure B8: Drawing LD-5 – Precast Concrete Manhole for Sewer (SABS, 1982a)



Dimensions in millimetres

NOTE: SEE SUBCLAUSE 5.6.1.

CHAMBER AND SHAFT SECTIONS			
TYPE OF SECTION	NOMINAL DIAM., mm	STANDARD LENGTH, mm	WALL THICKNESS, mm, MIN.
CHAMBER	1 000	250 , 500, 1 000	65
CHAMBER	1 250	250 , 500, 1 000	75
CHAMBER	1 500	250 , 500, 1 000	85
CHAMBER	1 750	250 , 500, 1 000	95
SHAFT	700 APPROX.	250 , 500, 1 000	55

REDUCER AND ADAPTOR SLABS		
TYPE OF SLAB	NOMINAL DIAM., mm	THICKNESS, mm, MIN.
REDUCER	1 000	150
REDUCER	1 250	150
REDUCER	1 500	150
REDUCER	1 750	150
ADAPTOR	750	150

Drawing LD – 5 – Precast Concrete Manhole for Sewer

Appendix C

Schedule of Quantities

Table C1: Typical schedule of quantities (conventional gravity sewer)

CONTRACT :				PART : GENERAL		
Item No	**Payment reference**	**Description**	**Unit**	**Quantity**	**Rate**	**Amount**
	SABS 1200LD	**SECTION: SEWERS**				
	8.2.1	Supply, lay, joint, bed and test pipelines				
		(a) .... (type) pipes on class ...... bedding				
		(1) ...... mm dia pipes.............	m			
		(2) .... (etc. for other diameters)	m			
		(b) .... (etc. for other types of pipes, class beddings and diameters)				
		(1) .... (etc. for others)	m			
	8.2.2	Extra-over Item 8.2.1 for specials				
		(a) .... (describe special)	No.			
		(b) .... (etc. for other specials)	No.			
	8.2.3	Manholes				
		Manholes as per Drawing No. .... complete with type ..... cover and frame for depths				
		Over and up to:				
		(1) 0 m 0.5 m	No.			
		(2) 0.5 m 1.0 m	No.			
		(3) .... (etc. for increments of 0.5 m)	No.			
	8.2.4	Extra-over Item 8.2.3 for backdrops, etc.	No.			
		Over and up to:				
		(1) 0 m 0.5 m	No.			
		(2) 0.5 m 1.0 m	No.			
		(3) ...... (etc. for increments of 0.5 m)	No.			
	8.2.5	Inspection chambers, etc.				
		(a) Inspection chambers as per Drawing No. .... complete with type ...... cover and frame for depths				
		Over and up to:				
		(1) 0 m 0.5 m	No.			
		(2) 0.5 m 1.0 m	No.			
		(3) .... (etc. for increments of 0.5 m)	No.			
		(b) .... (etc. for other types)				
	8.2.6	Erf connections				
		(a) .... (state type or drawing number)				
		(1) .... m (state length)	No.			
		(2) .... m (etc. for other lengths)	No.			
		(b) .... (etc. for other types)				
		(1) ...... m (etc. for other lengths)	No.			
	8.2.7	Encasing of pipes in grade........... concrete				
		(a) ...... mm diameter pipes	m^3			
		(b) ..... mm (etc. for other diameters)	m^3			
	8.2.8	Anchor blocks				
		(a) .... (state size or refer to Drawing No.)	No.			
		OR:				
		(b) Concrete grade .... including formwork	m^3			
	8.2.9	Marker posts	No.			
	8.2.10	Permanent plug stoppers (provisional)	No.			
	8.2.11	Connection to existing sewer at………	Sum			
		(as described in the Project Specifications)				
	8.2.12	Raising or lowering of existing manholes	No.			