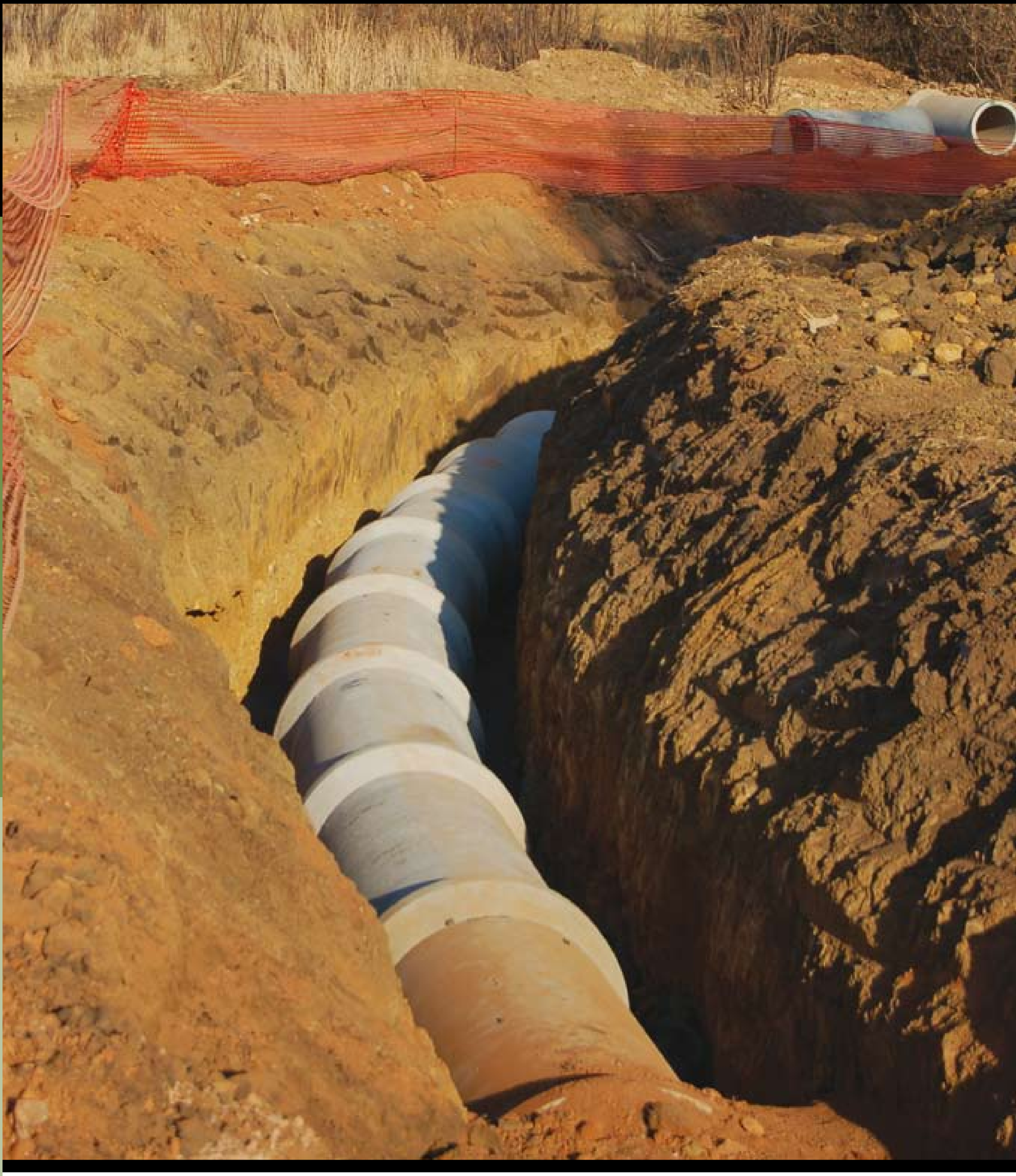
DESIGN MANUAL FOR CONCRETE PIPE OUTFALL SEWERS

P.I.P.E.S



Pipes, Infrastructural Products and Engineering Solutions Division

CONCRETE MANUFACTURERS ASSOCIATION

Quality cast in concrete

PREFACE

Research on sewer corrosion commenced at the CSIR in 1950, supported financially by Town Councils and concrete pipe manufacturers. The research was initiated by a report of the Springs Town engineer on the town's outfall sewer and the discussion of the sewer corrosion problem in general at a special combined meeting of the SA Chemical Institute and the SA Branch of The Institute of Sewage Purification as well as the 29th Annual Conference of The Institution of Municipal Engineers (SA District). The general consensus of opinion was that the matter be urgently researched, the task of which was commissioned to the CSIR, which proposed that the research be tackled simultaneously from the following angles:

(a) The microbiological aspects to elucidate the mechanism of corrosion.

(b) Field aspects that influence corrosion such as design, construction and operation of sewers.

(c) The use of construction materials with enhanced resistance to corrosion.

The work culminated in the publication of the research findings in 1958 in CSIR Series 12 book **Corrosion of Concrete Sewers,** which showed that practical, cost-effective steps could be taken to improve the resistance of concrete pipes to sewer corrosion. Particular mention must be made of the finding by JHP van Aardt of the NBRI, CSIR, to improve the resistance of concrete pipes by using calcareous aggregate instead of siliceous aggregate in them, a finding which is since being used world-wide.

Sewer corrosion research at the NBRI, CSIR was re-opened in 1964 to establish with what success the recommendations made in the above-mentioned book had been implemented and to research new and modified materials and alternative methods of combating corrosion as well as the economy of implementing the latter. Progress was reported in 1967 in CSIR Research Report 250 **Corrosion of Sewers** by JL Barnard. The main conclusion was that the 1958 report was well received and the recommendations applied widely. Unfortunately, the research discontinued and the long-term effect of these recommendations and the comparative performance of the various sewer pipe materials remained unanswered.

In 1986, Ninham Shand proposed via De Wet Shand that an experimental section of sewer pipe be installed at Virginia, OVS (for which they were the consultants); severely aggressive conditions were expected to occur in the sewer. The experimental section, with full access manholes was completed in 1989. The NBRI, CSIR, was commissioned, with financial assistance from industry, to monitor the condition and performance of the different sewer pipe materials. The results, recorded over the first five years (comprising Phase One), were published in **Consolidated Report on Phase 1 of Sewer Corrosion Research; The Viginia Sewer Experiment and Related Research.** The salient outcome was that the various exposed materials could be rated on a scale of 1-10, 1 giving the best resistance to corrosion and 10 the worst.

The CSIR's involvement in sewer corrosion research at Virginia stopped on completion of Phase One, and monitoring of the experimental line continued, firstly by UCT (Phase Two) and afterwards by UCT and an independent consultant (Phase 3).

This manual is a consolidation of important aspect that emerged from research in the RSA and elsewhere on sewer corrosion and the salient aspects that must be considered in the design of sewers to ensure that they perform satisfactorily for their design life, namely hydraulic design, material strengths, imposed loads, sewer corrosion and its control, field testing and the monitoring of sewers. Special mention should be made of the introduction of a material factor in the Life Factor equation to predict the corrosion rate of the material under a given set of corrosive conditions in a sewer to be installed and the section on a procedure for the design process.

The manual would undoubtedly be of great help as a reference to anyone who is involved in sewer corrosion and the design and installation of sewers to deal with corrosion problems.

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Concrete pipes and manholes are the most frequently used products for outfall sewers. South Africa's concrete pipe industry has grown tremendously over the past eighty years to meet sewer requirements and other needs.

Modern technology and SANS (SABS) standards ensure that products with a consistently high quality are produced. Provided sound design and installation methods are followed, these products will give the desired hydraulic, structural and durability performance for a long service life.

This manual is intended to cover all aspects of selecting, specifying and testing concrete pipes for outfall sewers. It does not attempt to replace textbooks or codes, but rather to complement them by providing guidance for designers. The focus is on the prediction and control of concrete sewer corrosion. Sections on hydraulics, pipe strength, product testing and condition assessment have been added so that all aspects of sewer design and specification are addressed. The Companion publications, 'The Concrete Pipe and Portal Culvert Handbook' and 'The Concrete Pipe and Portal Culvert Installation Manual', deal with these other aspects of product selection and product installation.

Publications by the American Concrete Pipe Association have been used freely and acknowledgement is hereby made to that organisation.

The Concrete Pipe, Infrastructural Products and Engineering Solutions (PIPES) Division of the Concrete Manufacturers Association has prepared these publications for the guidance of specifying bodies, consulting engineers and contracting organisations using concrete pipes and portal culverts manufactured in accordance with the relevant SANS (SABS) standards.

This is the first edition of this manual and the information presented has been collected from many sources. Constructive criticism is welcome and, where relevant, will be incorporated into future editions.

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1. INTRODUCTION

1.1. Background

The demand for sewage disposal systems began during the mid-19th century when it was realized that diseases such as cholera were water borne. Outfall sewers were installed in many European and North American cities and many of these are still operating (150 years later). There are numerous technical and economic reasons for the use of concrete sewers, whether they consist of precast pipe or cast-in-place conduits.

Concrete was and still is the ideal material for the manufacture of sewage pipes as it:

- can be moulded into almost any shape and size, and once set keeps its shape
- has a smooth surface that provides hydraulic efficiency
- is inherently strong and pipes can carry loads without support from surrounding soil
- is dense and for all practical purposes impermeable
- is durable over a wide range of conditions and does not deteriorate with age
- can be modified or protected to deal with almost any condition, and it
- is produced from materials that are readily available.

Two concrete properties that pose problems are that:

- although the material is strong in compression it is weak in tension, and
- cement, the binding material in concrete, is alkaline and can be attacked by acids that form in sewers under certain conditions or may, and may like other aggressive agents, such as sulphates, be present in the surrounding soil.

The first problem was resolved by the introduction of steel reinforcement in pipes at the beginning of the 20th century so that pipes can be designed with thinner walls to reduce costs without affecting their strength. In addition, these pipes have a two stage failure mode: initial cracking of concrete followed by yielding of steel, and these occur separately. However, reinforced concrete pipes must have sufficient concrete covering the reinforcement to ensure protection from aggressive elements in the environment.

The second problem is more difficult to address. There are two types of acid attack that can occur inside a sewer: biogenic acid, which occurs above the water line in domestic sewer systems and pure acid, which occurs as a result of industrial effluent and attacks the concrete below the water line. In addition, external attack may result from aggressive elements in the soil. Provided the correct information is gathered during the investigation stages of planning a sewer, the attack from aggressive effluent and from aggressive elements in the soil can be addressed as they would be for any other concrete structure.

However, biogenic attack is a phenomenon that is found only in sanitary sewers and occurs under a unique set of conditions (1). The acid involved is always sulphuric acid, the attack takes place on the portion of sewer above the water level, and although the volume of acid available is small its concentration is high. Although the mechanism of this attack was understood for some time before the EPA manual was published in 1974 (revised in 1985) (2), this document appears to be the first to quantify the procedure for predicting corrosion in sanitary sewers and give guidance on how to address the problems. The application and development of procedures described in these two publications is the focus of this manual.

1.2. Requirements

The primary requirement for a sewer is that its size is sufficient to convey effluent from the source to the discharge location. The secondary requirements are that it must be:

- watertight so that there is no exfiltration or infiltration
- strong enough to withstand the imposed loading
- durable so that it will not deteriorate unduly with time

- capable of rehabilitation should the need arise

Exfiltration of effluent causes ground-water pollution and a public health risk that could have serious social and environmental consequences. Infiltration of ground water adversely affects sewer capacity and this too could result in public health risks if sewers become surcharged and manholes overtopped. This would also mean an overloading of the purification facilities at the outfall works.

In addition, leakages indicate the probability of a flow path through the soil adjacent to the sewer. If the soil surrounding the sewer is fine or disbursable this flow will allow silt to enter the sewer resulting in maintenance problems and could cause cavities to develop around the sewer resulting in sinkholes.

The loads imposed on a sewer consist of external earth and traffic loads, and possible internal pressures resulting from blockages. The most serious loads are usually earth loads because sewers are placed at a depth below other services. If a sewer is overloaded and collapses there are implications for the other services located above it as well as for public health.

To ensure that a sewer meets the primary requirement it is essential that all the secondary requirements are met throughout its life. The decision that has by far the greatest impact on the long-term performance of any buried pipeline is the choice of pipe material.

There is no single pipe material that can effectively meet all the requirements for a sewer under all conditions. Concrete meets the requirements for sewers under most conditions, but it is susceptible to sulphuric acid attack under certain conditions and special precautions must be taken to ensure that the long-life requirement is met.

Although a thorough investigation may be done during the design stage of a sewer, operating conditions can change at a later stage with detrimental effects on the pipe material causing a sewer to deteriorate before the end of its planned design life. Replacement or rehabilitation is then necessary. The latter option is preferable when the area above the sewer has been developed since the original installation. Concrete pipes, which keep their shape and provide a basic structure to support a liner, are a distinct advantage.

1.3. Purpose and Scope

The purpose of this manual is to provide the designer with the basic guidelines and tools needed for the cost-effective design of concrete sewers, including the selection of the most appropriate pipe material. The related topics of sewer size, strength and jointing are also addressed. The chapter sequence follow the typical design procedure used on a sewer project. The various factors are interdependent and thus the process will be iterative to ensure that all the utility owner’s requirements are met.

Some concrete sewers installed before the corrosion analysis procedures presented in this manual were available, or under unfavourable conditions where the necessary precautions were not taken have collapsed and have had to be replaced; others have been or are in need of rehabilitation. The procedures for analysing the potential corrosion in new sewers are also applicable to existing sewers that have deteriorated, especially where it is necessary to determine their remaining life so that rehabilitation priorities can be set.

While this publication focuses on the design of outfall sewers, the principles given are also applicable to the reticulation and collector pipelines that transport the effluent to the outfalls. However, the designer of these smaller components should determine what the sewer owner’s requirements are as several of the large local authorities in South Africa have compiled guidelines and procedures based on local experience.

2. HYDRAULIC CONSIDERATIONS

2.1. Some Basic Definitions

Sanitation system: All the components including the pipelines that convey sanitary waste water away from where it is generated to the outfall works where it is treated and purified before being discharged into the natural watercourses.

Reticulation: The smallest element of a sanitation system consisting of small diameter (110 to 225mm) pipelines that convey effluent from individual properties and along streets.

Collector: Intermediate sized pipeline (150 to 450mm diameter) that conveys effluent from the reticulation to the main outfall sewers.

Outfall sewer: Large diameter sewers that convey the waste water from collectors to the purification works.

Hydraulic grade line: In a part-full conduit this line represents the height above datum plus flow depth. In a full conduit it represents height above datum plus flow depth and pressure head.

Total energy line: The line represents the hydraulic grade line plus velocity head.

Friction slope: Slope of the total energy line.

2.2. General

The hydraulic design of sewers needs to consider both minimum and maximum velocities and energy losses. Vertical alignment may need adjustment to ensure that the appropriate values of these parameters are met. As outfall sewers are generally gravity pipelines that flow partly full, they follow natural watercourses and this poses additional alignment problems. Watercourses often have alignments that are not ideal for sewers. They usually consist of relatively flat sections interspersed with short steep sections such as rapids or waterfalls and they do not flow in straight lines, but bend and twist.

Thus outfall sewers should be designed to cope with both vertical and horizontal changes in alignment and the energy losses that occur at these changes must be added to friction losses to ensure effective performance. There are two additional constraints that complicate these problems. Outfall sewers are frequently constructed through urban areas and are surrounded by variable soil conditions due to cycles of erosion and deposition.

Sewers are usually designed to flow at between 50% and 61% of pipe diameter under peak dry weather flow (PDWF) conditions and completely full under peak wet weather (PWWF) flow conditions. Storm water infiltration under PWWF amounts to between 30% and 50% of full flow capacity. These represent the maximum flow conditions needed to determine the sewer capacity. Most of the time flow through a sewer is significantly less than these values and the critical issue may be the minimum velocity required for self cleansing. In this case, the average dry weather flow (ADWF) which may be only 25% of the PDWF. A ratio of ADWF to PWWF for an outfall sewer of between 0.12 and 0.18 can be expected. The determination of these parameters and the relationship between them is beyond the scope of this publication.

The designer has to address all the alignment issues and it may not always be economically viable to meet the ideal technical requirements. The different issues at stake can be grouped as those relating to vertical and horizontal alignment respectively.

It is assumed that the reader understands continuity, energy and momentum principles, their application to the hydraulics of gravity and pressure pipelines, the concepts of critical depth and hydraulic length, the friction formulae derived from Chezy, such as Manning and Colebrook White and how to use the relevant hydraulic charts.

2.3. Vertical Alignment

2.3.1. Basic Principles

The maintenance of minimum velocity so that sewer does not become silted is an accepted hydraulic principle for sewers design. If this principle is not adhered to there will be a continual maintenance problem, although the sewer itself will probably remain sound. In contrast, the maximum velocity requirement is seldom given sufficient attention and the resulting problems due to supercritical flow, hydraulic jumps and invert erosion can have serious long term implications.

The hydraulic factors that should be considered to ensure effective sewer operation are:

- The velocity at low flow, not at average flow, should be self-cleansing.
- The total energy line should be contained within a sewer.
- Uncontrolled hydraulic jumps inside a sewer should be avoided.

As velocity is proportional to both diameter and gradient small diameter sewers are often at gradients that are insufficient to ensure self cleansing velocities. Although a generally accepted value for minimum velocity is 0.6 to 0.8 m/s this may be inadequate when flat gradients are downstream of steep gradients where high velocities have allowed a heavy bed-load to be transported.

At the other extreme, large diameter sewers are sometimes at gradients that result in excessive and possibly supercritical velocities. This can result in two problems. The first manifests itself when velocity is supercritical and the total energy line is not contained within the sewer. Any obstruction to flow could cause a hydraulic jump resulting in the sewer flowing full and the possibility of manholes surcharging. The whole flow regime within the system may then become unstable. The second occurs when the invert of the sewer erodes, resulting in a long-term problem with water tightness and structural integrity.

In the case of small diameter pipes such as those used in reticulation and collector sewers, high velocities are seldom a problem as the velocity heads are not significant and surcharging can usually be accommodated by manholes. This does not mean that precautions to deal with the effects of transitions and changes in horizontal alignment (as described in a section that follows) should be ignored. In the case of outfall sewers the cross-sectional area of the pipes approaches that of manholes and surcharging will result in overflow. This problem may be avoided by allowing the sewer to operate under a nominal pressure for short sections or keeping the total energy line within the sewer. The former is described in section 2.4.3 on page 13 on manhole pipes where the openings in pipes are covered with lids. The latter involves flattening the gradient and may require that larger diameter pipes be used.

On sewers ≥ 900mm in diameter supercritical velocities are excessive and it can be difficult to contain the total energy line within the sewer. An effective way to reduce these velocities is to construct drop structures to break the velocity and reduce the energy by forcing a hydraulic jump to occur at a fixed location. The maximum velocities in sewers and the use of drop structures to eliminate supercritical flow need to be evaluated in relation to the pipe diameter. Drop structures are described in section 2.3.3 on pages 7 to 10 of this document.

2.3.2. Velocity and Discharge

Velocity is determined using one of the friction formulae, such as Manning, given below:

$$v = (1/n)\,(R)^{2/3}S^{1/2} \qquad (1)$$

Where v - velocity in m/s

n - Manning's roughness coefficient

R - hydraulic radius (m)

S - gradient of conduit (m/m)

The friction slope of a pipeline that has a constant gradient and no transitions is the energy difference between inlet and outlet divided by the pipeline length and will be the same as the conduit gradient. If there are any transitions in the pipeline, the energy losses resulting from them will reduce the amount of energy available to overcome friction. The friction slope will not necessarily be the same as the conduit gradient. The energy losses due to transitions in a conduit can be determined theoretically by comparing flow areas before and after the transition. For most applications using a coefficient as shown in formula below is adequate:

$$H_L = k\,v^2/2g \qquad (2)$$

Where H_L - head loss in metres (m)

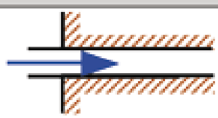
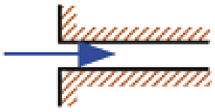
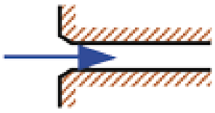
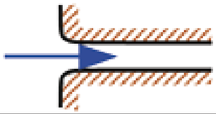
k - energy loss coefficient, between 0.0 and 1.0, dependent upon details

v - conduit velocity in metres per second (m/s)

g - the gravitational constant in metres per second per second (m/s/s)

Transition losses due to converging flow are smaller than those due to diverging flow. The transitions in outfall sewers are usually made at manholes. The manholes should be benched 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. If such benching is not included through manholes the energy losses can be excessive. Commonly used energy loss coefficients are given in Table 1 below.

TABLE 1: ENERGY LOSS COEFFICIENTS FOR PIPELINE FLOW

Shape	Detail	Inlet	Outlet
Protruding – stormwater with no inlet or outlet structure		0.80	1.00
Sharp – stormwater with headwalls, but no wingwalls, or sewer with butt end built in and no benching		0.50	1.00
Bevelled – sewer with socket end built in and no benching		0.25	0.50
Rounded – sewer with socket end built in and benching		0.05	0.20

Sewer capacity is determined by using one of the friction formulae in combination with the continuity equation:

$$Q = Av \qquad (3)$$

Where Q - discharge in cubic metres per second (m^3/s)

A - cross-sectional area in square metres (m^2)

v - velocity in metres per second (m/s)

Most sewers flow partly full under the influence of gravity and therefore have no additional energy inputs. Under these conditions the friction slope and the pipeline gradient between transitions will be the same. If a siphon or pumped section is incorporated in the sewer it will flow full, there will be a pressure head and the hydraulic grade line will be above the sewer and the friction slope and the pipeline gradient will be different. The hydraulic properties of a sewer flowing full can be determined by using a chart, such as that shown in Figure 1. Energy inputs or losses are either added or subtracted. For any sections of the sewer under pressure the friction slope, calculated from the energy difference between the inlet and outlet of those sections should be used.

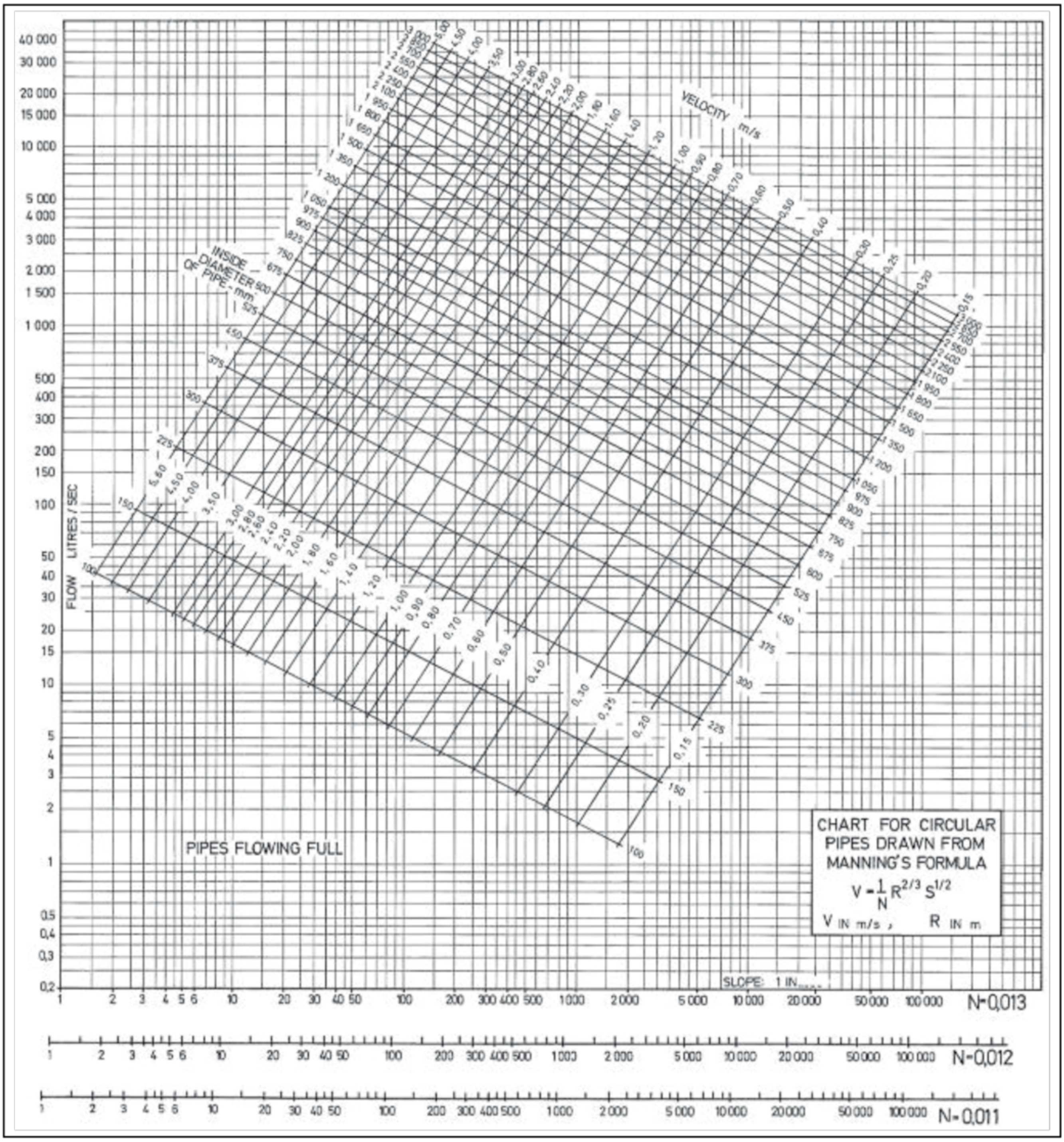


FIGURE 1: FLOW CHART FOR CIRCULAR PIPES BASED ON MANNING (SHOULD YOU NOT EXPLAIN HOW TO USE THE FIGURE?)

It should be noted that Figure 1 employs a log-log scale and has been compiled for a range of ‘n’ values. Due to scaling effects it will not give accurate results and it should be used for estimates only. The values from this chart should be adjusted for the actual ‘n’ values. The recommended ‘n’ values for sewers are:

- 0.013 for a standard concrete sewer
- 0.011 for a non man-entry plastic lined sewer
- 0.010 for a man-entry plastic lined sewer

For more accurate values of velocity and discharge equations (1) and (3) should be used or other equivalent formula. This is very important because actual dimensions, gradients and roughness values should be used in the formulae.

The hydraulic properties of sewer sections flowing partly full can be determined by using the 'pipe flowing full' chart or the appropriate formulae, and then adjusting the values using this proportional flow given in Figure 2, which shows the relationship between the relative depth d/D and the other parameters as hydraulic radius, velocity and discharge. This graph is adequate for most applications where sewers are flowing partly full and values can be accurately interpolated because of the linear scale. It should be noted that it is based on pipe geometry with no allowance for changes in roughness coefficient due to depth of flow. Such adjustments would have a slight effect on the flow properties.

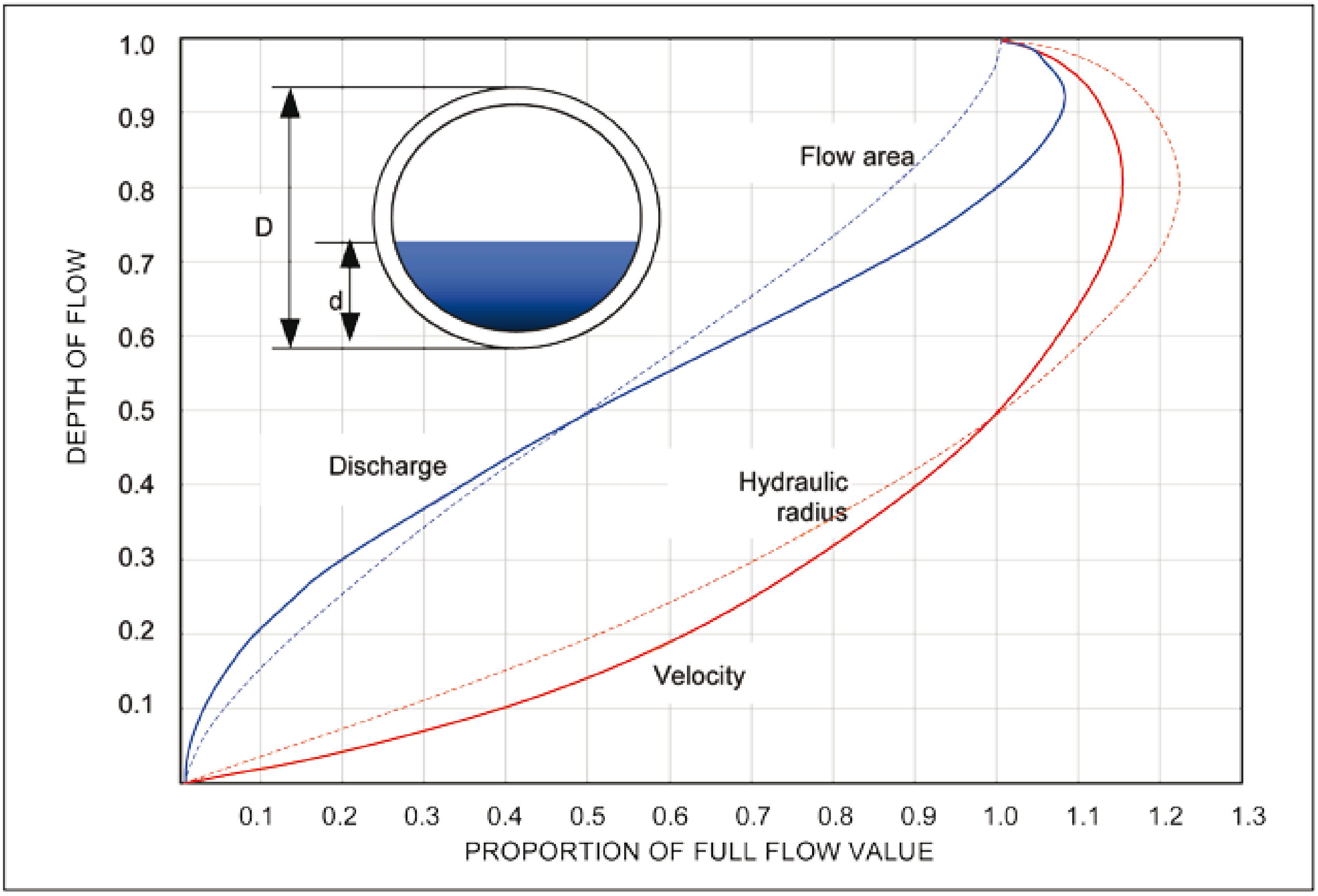


FIGURE 2: FLOW PROPERTIES OF CIRCULAR PIPE FLOWING PARTLY FULL

2.3.3. Drop Structures

As these structures are buried it is important, for economic reasons to keep both their length and depth to a minimum. The length is calculated to ensure that the planned hydraulic jump at any flow less than the maximum design value occurs within the structure. Placing a sill at the downstream end of the structure achieves this. Should the maximum design flow be exceeded the jump will still be initiated within the drop structures, but at the risk that the pipe will flow full if the total energy line is above the pipe overt.

The critical vertical dimensions of a drop structure are the drop and the sill heights. These are determined by hydraulic performance at maximum design flow. The Froude (Explain it) number at the bottom of the drop should not exceed 8 or 9 and the sill height must ensure that the energy levels just downstream of the jump and downstream of the sill are the same. This forces the jump to occur at the desired location. A typical section through a drop structure is given in Figure 3.

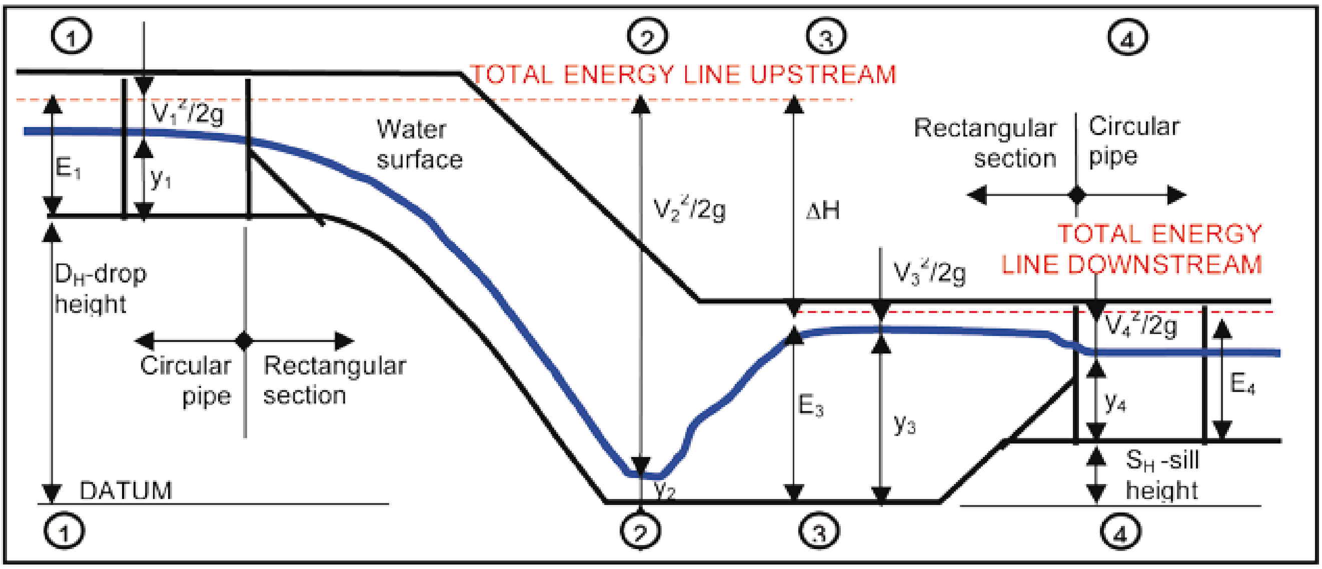


FIGURE 3: TYPICAL SECTION OF DROP STRUCTURE

At each of the sections 1 to 4 the energy equation holds:

$$E = y + v^2/2g \qquad (4)$$

Where E is total energy at a section. The other symbols were defined above

Although there will be small energy losses due to friction between sections 1 and 2 and between sections 3 and 4, it has been assumed that these are negligible. Hence the total head at sections 1 and 2 and at sections 3 and 4 is the same.

$$H_U = D_H + y_1 + v_1^2/2g = y_2 + v_2^2/2g \qquad (5)$$

$$H_D = y_3 + v_3^2/2g = SH + y_4 + v_4^2/2g \qquad (6)$$

Where H_D is the total head above the datum, just downstream of the drop structure

S_H is the sill height

H_U is the total head above the datum, just upstream of the drop structure

D_H is the drop height

Equations (5) and (6) have to be solved using an iterative process.

Between sections 2 and 3 there will be a hydraulic jump and an energy loss. The relationship between y_3 and y_2 (Should it not be y_2 can be determined from equation (7).

$$y_3/y_2 = 0.5\,((1 + 8\,Fr_2^2)^{1/2} - 1) \qquad (7)$$

Where Fr_2 is the Froude number at section 2

The Froude number indicates the relative influence that momentum and gravity have on the flow of water. In a rectangular channel the Froude number is determined from equation (8):

$$Fr^2 = v^2/gy \qquad \text{or } Fr = v/\sqrt{gy} \qquad (8)$$

Where the symbols are as described above.

The energy loss due to the hydraulic jump can be calculated from:

$$\Delta E = (y_3 - y_2)^3/\,(4y_2\,y_3) \qquad (9)$$

Where the symbols are as described above.

When the calculations are completed the energy levels at the four sections should be checked to ensure that the total energy levels including the loss balance:

$$H_U = D_H + y_1 + v_1^2/2g = y_2 + v_2^2/2g = y_3 + v_3^2/2g + \Delta E$$
$$= S_H + y_4 + v_4^2/2g + \Delta E \qquad (10)$$

A critical aspect of a drop structure is the spillway shape. This should follow an ogee curve to ensure that the water jet does not separate from the spillway surface as in the long-term this will cause cavitation. This profile is given by equation (11):

$$x = 1.4545\ y^{0.5405}\ E1^{\ 0.4595}, \text{ or } y = 0.5\ x\ 1.85\ E1^{\ -0.85}. \qquad (11)$$

Where E1 is the energy above the sewer invert at section 1

x is the horizontal distance from start of curve

y is the vertical drop from start of curve

At a certain height the curve becomes tangential to the line at 60° to the horizontal. At heights greater than this the spillway profile follows this straight line. This happens at a spillway height that is dependent on the energy level immediately upstream of the drop. Hence the same profile can be used for a structure that has a total drop in excess of this.

When the Froude number at the bottom of the drop exceeds 8 or 9 the velocity will be excessive and an alternative means of dissipating energy, such as multiple drop structures or a vortex structure, should be considered. This choice will be based on topography and cost.

The drop structure length is directly related to the hydraulic jump height at maximum flow. Its apron length is set to contain 30% of the jump length as shown in equation (12).

$$L_A = 0.3\ L_j = 0.3\ 5.6\ y^3 = 1.86\ y^3 \qquad (12)$$

Where L_A is the horizontal distance from end of the drop to the start of the sill

L_j is assumed to be the length of a stable hydraulic jump.

In plan the drop structures is the width to the internal diameter of the sewer over their whole length. There is a transition from a circular section to a rectangular section at both entrance and exit from the structure. From a practical perspective a drop structure will have to at least 1000 mm wide for construction purposes. When the sewer diameter is less than this minimum width it may be necessary to bench the bottom of the structure to ensure that the velocity at section 3 is self-cleansing. There is, however, no practical limit to the maximum diameter sewer for which these structures could be built. The maximum drop height will be limited by the hydraulics and will be approximately 3.6m.

The approach to the problem of high velocity and the inclusion of drop structures should be based on several factors: namely whether or not: flow is supercritical; the total energy line is within the pipe; the velocity exceeds 3.0 to 3.5m/s.

2.3.4. Further Considerations for Drop Structures

The design of a reinforced concrete drop structure should be based on the maximum fill height as given on the drawings and once the levels are finalised. In sewers where the corrosion potential is severe, this potential will be more severe in the drop structure than the sewer itself due to the increased gas release as a result of the turbulence generated by the hydraulic jump. The lining of internal surfaces above the low water line with HDPE should be considered, any pipe ends that are exposed at the inlet to or outlet from the structure and steps cast into the walls should be protected. If the drop structures are placed solely to break the velocity and corrosion is not a problem, these measures are not necessary.

The main advantage of using drop structures is that the velocity in a sewer can be limited to sub-critical values and ideally kept at a Froude number < 0.8. This will mean that:

- steep gradients and unpredictable flow conditions are eliminated;
- super-critical flow and hydraulic jumps are confined to fixed locations; and
- severe turbulence and gas release leading to severe local corrosion are confined

The benefits of these structures will be greater for large diameter sewers because they achieve high velocities at flatter gradients and the problems associated with high velocities are greater that those experienced with smaller sewers. In addition, they provide an effective way of avoiding obstructions in the path of a sewer, such as rivers or other services, by being placed under them.

The main disadvantages of drop structures is that they are built in situ and take longer to construct than an equivalent length of sewer and the technical benefits may not justify the additional expenditure. Problems with blockages and siltation are more likely to occur at such locations than elsewhere in a sewer, so there must be access for maintenance.

2.4.Horizontal Alignment

2.4.1. Basic Principle

Velocity is a vector so when there is a change in direction one component is altered and there is an energy loss. Theoretically this loss can be calculated from equation (13) below:

$$\Delta E = (1 - (\cos \theta)^2)\,(v^2/2g) \qquad (13)$$

Where ΔE is the energy loss at bend

θ is the angle of deflection

v is the velocity in m/s

g is the gravitational constant in m/s/s

The application of this equation to a 5° deflection when the flow is 1 m/s produces an energy loss of 0.4 mm. The value of the resulting energy loss for a range of deflection angles and velocities on a single bend is presented in Table 2. The relationship between neither energy loss and angle nor energy loss and velocity is linear; they are both exponential relationships.

If the bend is negotiated incrementally by having a small deflection at each pipe joint such that the bend approximates a curve then the total loss will be less than that which will occur at a single sharp bend. The loss around such a curve can be calculated from equation (14):

$$\Delta E = N\,(1 - (\cos \theta)^2)\,(v^2/2g) \qquad (14)$$

Where ΔE is the energy loss around curve

θ is the angle of deflection at each joint

N is the number of deflected joints and the other symbols are defined above.

TABLE 2: ENERGY LOSSES IN MM AT SEWER BENDS AT VARIOUS VELOCITIES

Velocity	Total Deflection at Bend in Degrees									
m/s	5°	10°	20°	30°	40°	50°	60°	70°	80°	90°
1.0	0.4	2	6	13	21	30	38	45	49	51
2.0	1.6	6	24	51	84	120	153	190	198	204
3.0	3.5	14	54	115	190	270	345	406	445	459
5.0	9.7	38	149	319	527	749	957	1127	1237	1276

The energy losses based on the number of joints with 5° deflections used to incrementally negotiate the deflections in Table 2 to approximate a smooth curve are given in Table 3. They are calculated using equation (14). These energy loss predictions are more conservative than those published for pipes flowing full and will hold provided the curve that is approximated does not have a curve radius/pipe diameter (R/D) ratio that is less than 8.

TABLE 3: LOSS ALLOWANCES IN MM FOR CURVES WITH 5° DEFLECTION PER JOINT

Velocity	Total Deflection of Curve in Degrees (No of joints)									
	5°	10°	20°	30°	40°	50°	60°	70°	80°	90°
m/s	(1)	(2)	(4)	(6)	(8)	(10)	(12)	(14)	(16)	(18)
1.0	0.4	0.8	1.6	2.3	3.1	3.9	4.6	5.4	6.2	7.0
2.0	1.6	3.1	6.2	9.3	12.4	15.5	18.6	21.7	24.8	27.9
3.0	3.5	7.0	14.0	20.9	27.9	34.9	41.9	48.8	55.8	62.8
5.0	9.7	19.4	38.8	58.1	77.5	96.9	116.3	135.6	155.0	174.4

Comparing the losses in these two tables clearly shows that the energy losses around a smooth curve at all velocities are significantly less than those at a sharp bend and that as the total angular deflection increases so does the difference between these two values.

The radius of the curve followed by a sewer centre line is described by the equation below:

$$R = L / (2(\tan \Delta/2N) \qquad (15)$$

Where R is radius to sewer centre line in m

L is pipe length along its centreline in m

Δ is total deflection in degrees

N is number of pipe with deflected sockets

The deflection that can be tolerated at the joint in a concrete pipe is small due to the pipe rigidity and tight tolerances. Concrete pipes can, however, be custom made with deflected sockets so that curves can be negotiated. Typically these will allow a deflection of between 2° and 6° with the tolerances being applicable to these deflections. In practice the shortest concrete pipes are approximately 1.22 m. If the angle of deflection for a custom made 900mm diameter pipe is 10° the R/D ratio of 7.7 is unacceptable. If the angle is 5°, the ratio increases to 15.5, which is acceptable.

2.4.2. The Impact of Manholes

A comparison of energy losses excluding those due to transitions at sharp bends (as would be the case if manholes were used) and those due to gradual curves for a velocity of 3.0m/s is presented in Table 4. This clearly shows that the energy losses around a sharp bend (Table 2) are far greater than those around a smooth curve (Table 3).

TABLE 4: COMPARISON OF LOSSES AT BENDS AND CURVES AT VELOCITY OF 3m/s

Configuration	Total Deflection around Bend or Curve in Degrees								
	10°	20°	30°	40°	50°	60°	70°	80°	90°
No of joints	2	4	6	8	10	12	14	16	18
Smooth Curve	7	14	21	28	35	42	49	56	63
Sharp Bend	14	54	115	190	270	345	406	445	459
Difference	7	40	94	162	235	303	357	389	396

Where energy losses are given in mm.

These tables show the energy loss due to the change in direction only. Compensation for these losses should be made by increasing the level drop around the curve by the value of the energy loss. If the change in direction is made at a manhole, the drop in level must compensate for losses due to both the changes in direction and cross sectional shape through the manhole. Losses due to changes in direction are calculated using equations (13) or (14) above. Losses due to cross sectional changes are calculated by applying an energy loss coefficient to the velocity head as given in equation (16).

$$\Delta E = K_T \, v^2/2g \qquad (16)$$

Where ΔE is the energy loss at transition

K_T is the energy loss coefficient for the type of transition

v is the velocity in the downstream section of sewer in m/s

g is the gravitational constant in m/s/s

If a manhole is benched to match the pipe profile below water level there is no energy loss as the flow section is unchanged. However, if the flow shape and area through the manhole are different from that through the sewer (as occurs when benching is overtopped) there will be energy losses and equation (16) is applicable. When manholes are a discontinuity in a sewer the energy loss coefficient will have two components: one for the downstream section of sewer and one for the upstream section of sewer. These individual and the combined coefficients can be determined from equations (17), (18) and (19).

$$K_{d/s} = k\,[1 - (A_p/A_{mh})^2]^2 \qquad (17)$$

Where $K_{d/s}$ is the energy loss coefficient for inlet to downstream section of sewer

K is a coefficient depending on smoothness of transition (0.1 – 0.5)

A_p is the flow area in the downstream section of sewer in m^2

A_{mh} is the flow area in the manhole in m^2

$$K_{u/s} = k\,[1 - (A_p/A_{mh})^2] \qquad (18)$$

Where $K_{u/s}$ is the energy loss coefficient for outlet from upstream section of sewer

k is a coefficient depending on smoothness of transition (0.2 – 1.0)

A_p is the flow area in the upstream section of sewer in m^2

A_{mh} is the flow area in the manhole m^2

$$K_{MH} = K_{d/s} + K_{u/s} \qquad (19)$$

Where K_{MH} is the energy loss coefficient through the manhole

$K_{d/s}$ and $K_{u/s}$ are defined above by equations (17) and (18)

These formulae are applicable to sewers flowing full, which is the case if the manholes are surcharged. Assuming that a sewer flowing into and out of a manhole is flowing just full and that the bottom half of the manhole is benched to match the sewer profile, the application of these equations gives a total loss through the manhole of 0.25 times the velocity head. However, if the flow depth in the manhole is twice the sewer diameter, these equations produce a total loss of 0.98 times the velocity head. This indicates that energy losses at manholes will increase if they are surcharged.

Storm water infiltration under PWWF is considered to account for between 30% and 50% of full flow capacity. This means that above calculations are applicable to PWWF and precautions are needed to prevent surcharging from occurring.

As it is difficult to bench manholes above half of the pipe diameter, one way to deal with surcharging during PWWF is to have the pipes continuous through the manholes and provide access via a hole through the top of the pipe. Such a pipe is called a manhole pipe. If there is a concern about surcharging the hole can be covered with a lid conforming to the pipe profile and fixed to the pipe so that excess pressure is relieved.

For reticulation systems, where small diameter pipes (circa 160 mm) are used, the flow may need to be at critical velocity or faster for self-cleansing. This is not a problem with changes in direction as these can be made at manholes, because they are large enough in comparison to the pipes to radius the benching and accommodate surcharging that may occur. Also, velocities are unlikely to exceed 1.5 or 2.0 m/s and these should not cause any serious problems.

However, for outfall sewers of diameters ≥ 900 mm, supercritical flow and in particular velocities greater than 3.0 m/s, could give rise to serious problems as the pipe and manholes diameters are similar. The manholes cannot buffer surcharging and economically it is not viable to make them large enough to include bends of the required radii. Hence, if there is supercritical flow, sudden changes in direction or obstructions to flow could cause a hydraulic jump and unstable flow conditions resulting surcharging and manholes overtopping.

By way of example, a 900 mm diameter sewer flowing at a depth of 61% of diameter and at 3.0 m/s conveys 1336 litres of water a second or 80 cubic metres a minute. The total energy line will be 1008 mm above the pipe invert; or 108 mm above the top of the pipe. If an hydraulic jump does occur the pipe will suddenly flow full and the velocity will be reduced to 1.46 m/s. The capacity of the sewer will be reduced and the jump will then start moving up the sewer and filling it. There will be greater energy losses at the upstream manholes that could result in surcharging and cause energy losses that become progressively larger at each manhole as the distance from the initial jump increases. If the manholes are 1200 mm in diameter they can only accommodate 1.13 cubic metres per meter of height, which is less than the flow through the sewer, so they could start overtopping within seconds of the hydraulic jump occurring.

2.4.3. Manhole Pipes

When a manhole pipe is used, rather than a structure, such as a conventional manhole with the sewer flowing in one side and out the other, there is no transitional energy loss as the flow section is unchanged.

If the velocity in a sewer is supercritical sudden losses as could occur at a manhole, even if only just surcharged, would cause an hydraulic jump just upstream of the transition. This in turn could result in the upstream section of the sewer flowing full and the surcharging of upstream manholes becoming progressively worse.

The operating advantages of manhole pipes with a lid over access holes are:

- constant flow profile along the whole sewer
- minimum energy losses
- all joints are sealed with rubber rings
- the same corrosion prevention or control measures are taken along the whole sewer
- no one can fall directly into the sewer
- foreign objects cannot be thrown into the sewer

The advantages of using manhole pipes during construction are:

- the sewer can be installed continuously from beginning to end, site conditions permitting
- gaps in the sewer are minimised and possibly eliminated
- inflow of debris into the sewer during construction is minimised
- possible reduction in construction costs and time

When manhole pipes are used in conjunction with deflected socket pipes at changes in direction so that smooth curves are approximated, the sewer performance is further enhanced because energy losses are minimal.

2.4.4. Distance between Access Points

Although the above measures will improve a sewer’s operation by minimising siltation, suitable access must still be provided for cleaning should the need arise.The use of curves and manhole pipes could give rise to problems if the access points are too far apart.

A bucket that is to be pulled through a sewer must be designed so that it can be pulled from both ends and is able to ride over the joints between pipes otherwise it will snag at the smallest imperfections in the pipeline. This means that it must be possible to pull the bucket around a bend from both upstream and downstream directions even if it rides against the pipe wall. However the cables pulling the bucket must not rub against the sewer walls, as they could cause damage.

The maximum angle that can be negotiated by cleaning equipment between access points can be calculated using the pipe and equipment dimensions and the bend geometry. The angular deflection that can be tolerated between the access points will increase as the pipe diameter relative to the bucket width increases and the number of increments in which the bend is made is the same, provided there is the same deflection at each joint and the pipe lengths. This can be tested by drawing the configuration to scale.

3. LOADS ON BURIED PIPES

3.1. Introduction

Loads on buried pipelines are divided into primary and secondary loads. Primary loads can be calculated and include those due to earth fill, traffic and internal pressure. Other primary loads, such as pipe and water mass, can be ignored, except in critical situations.

Secondary loads are variable and unpredictable and difficult to calculate. They can, however cause considerable damage to a pipeline due to differential movement between pipes. It is essential that their potential impact be recognised and that the necessary precautions are taken. Examples of factors that could cause secondary loads are:

- 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.

Where pipelines operate in exposed conditions such as on pipe bridges or above ground, pipes will be subject to thermal stresses and longitudinal movement. Thermal stresses and moisture content changes are caused by temperature differences between the inside and outside of the pipe that alternate cyclically and can cause cracks in pipe walls. This is generally not a problem when the pipe walls are less than 100 mm thick. Longitudinal movement is caused by the expansion and contraction of the pipeline due to temperature changes and can be a problem for any diameter pipeline if adequate measures are not taken. The design of the pipe and pipeline for such conditions is beyond the scope of this manual and should be discussed with a competent manufacturer or specialist consultant.

Although the content of this section covers rigid pipes, such as a concrete a short section on flexible pipes has been added.

3.2. Earth Loads

The calculation of earth loads on buried conduits from first principles is complex. For a thorough understanding, reference should be made to specialist literature and SANS 10102 Parts 1(3) and 2(4). The most important factors for establishing earth loads on buried conduits are:

- the installation method
- fill height over conduit
- backfill density
- trench or external conduit width

To use the tables that follow the loading conditions should be understood. The two basic installation types and corresponding loading conditions are called ‘trench’ and ‘embankment’. These are defined by whether the frictional forces developed 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. This is the mass of soil directly above the conduit assuming there is no friction between this column of material and the columns of soil either side of the conduit. These loading conditions are illustrated in Figure 4.

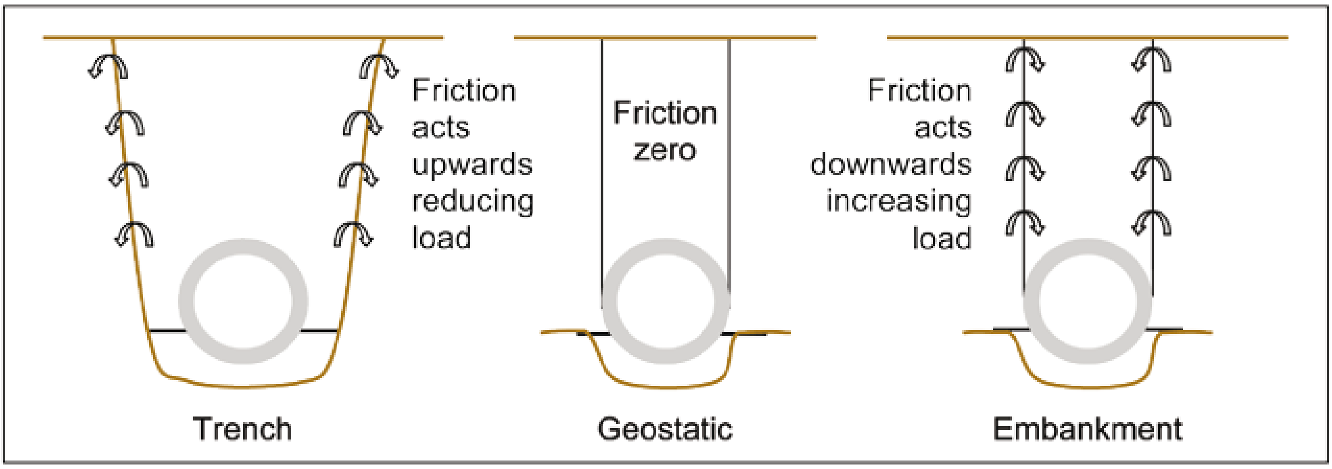


FIGURE 4: COMPARISON OF TRENCH, GEOSTATIC AND EMBANKMENT LOADING

The geostatic load has a value between the trench and embankment load. It is calculated from equation (20), which is the basis of earth loading equations for the other conditions.

$$W_E = \gamma\ H\ B^2 \qquad (20)$$

Where: W_E - load of fill material in kN/m run of sewer

γ - unit load of fill material in kN/m^3

B - trench width on top of conduit, or the outside diameter of pipe in m (trench or embankment condition respectively)

H - is fill height over pipe in m

3.3. Trench condition

Trench condition occurs when the conduit is placed in a trench that has been excavated into the undisturbed soil. Trench installation involves the development of upward frictional forces between the column of earth in the trench and the trench walls which reduce the load that the conduit has to carry. As a result the load on the conduit is less than the mass of the material in the trench above it. The load on the conduit is calculated from equation (21):

$$W_t = C_t\ \gamma\ B_t^2 \qquad (21)$$

Where: W_t - load of fill material in kN/m

γ - unit load of fill material in kN/m^3

B_t - trench width on top of conduit in m

C_t - coefficient that is function of fill material, trench width and fill height

This formula demonstrates the importance of the trench width B_t, which should always be kept to a practical minimum. As the trench width is increased so is the load on the conduit. At a certain stage the trench walls are so far away from the conduit that they no longer help it carry the load. The load on the conduit will then be the same as the embankment load. If the trench width exceeds this value the load will not increase any more. This limiting value of B_t – at which no further load is transmitted to the conduit – is the "transition width". The calculation of the transition width is described in the specialist literature. It is safe to assume that any trench width that gives loads in excess of the embankment condition exceeds the transition width.

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. This maximum load can be calculated from:

$$W_E = 2.63\gamma B_t^2 \text{ in sandy conditions} \qquad (22)$$

$$W_E = 3.84\gamma B_t^2 \text{ in clayey conditions} \qquad (23)$$

Where the symbols are as defined above and in Figure 5.

Trench loading on a circular pipe where the trench widths and nominal pipe diameters are specified is given in Table 5. Trench loading on conduits where the trench widths are specified but the conduit dimensions are not is presented in Table 6.

TABLE 5: TRENCH LOADS ON CIRCULAR PIPE IN KN/M; NON-COHESIVE SOIL

Diameter in mm		Trench width m	Height of backfill above top of pipe in metres										
Inside	Outside		0.6	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0
225	259	0.86	9	15	21	26	30	34	37	40	44	48	50
300	345	0.95	10	17	23	29	34	39	42	46	51	56	59
375	431	1.03	11	18	26	32	38	43	48	52	59	64	68
450	518	1.12	13	20	28	36	42	48	54	58	66	72	77
525	604	1.20	14	22	31	39	47	53	59	64	74	81	87
600	690	1.29	15	23	33	42	51	58	65	71	81	90	97
675	776	1.38	16	25	36	46	55	63	70	77	89	99	107
750	863	1.66	19	31	44	57	69	80	90	99	115	129	141
825	949	1.75	20	32	47	61	73	85	95	105	123	139	152
900	1035	1.84	21	34	50	64	77	90	101	112	131	148	163
1050	1208	2.21	26	42	61	79	96	112	127	141	167	190	210
1200	1380	2.38	28	45	66	86	104	122	138	154	183	209	233
1350	1620	2.62	31	50	73	95	116	136	155	173	207	237	264
1500	1800	2.80	33	53	78	102	125	147	167	187	224	258	288
1650	1980	2.98	35	57	84	109	134	157	180	201	242	278	312
1800	2160	3.36	39	65	95	125	153	180	206	231	279	323	363

Notes

1. For nominal pipe diameters ≤ 1200 mm the external diameter has been taken as 1.15 times the nominal diameter; for larger sizes 1.2 times the nominal diameter.
2. Table 5 is for non-cohesive soil; gravel or sand; unit weight = 20 kN/m^3 and K_μ = 0.19. (K_μ is a parameter based on the frictional properties of the soil.)
3. The tables are based on the trench widths recommended in SANS 1200DB.
4. 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.
5. This procedure is valid only if the soil properties other than unit weight do not change.

TABLE 6: LOADS ON ANY CONDUIT IN KN/M FOR GIVEN TRENCH WIDTHS

Trench Width m	Height of Backfill above top of pipe in metres										
	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
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 that Table 6 is for the same installation conditions and soil properties used in Table 5.

3.4. Embankment condition

In this condition the conduit is installed at ground level and covered with fill material. Generally, the soil surrounding the conduit is homogeneous and the compaction is uniform. 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. Thus the load on the conduit is greater than the mass of the material directly above it. On the other hand, the founding material under the conduit could yield and partly reduce the frictional forces and hence the load.

The load on a conduit is calculated from the formula:

$$W_e = \gamma \, C_e \, B_c^{\,2} \qquad (24)$$

Where W_e - load on pipe in kN/m

γ - unit weight of fill material in kN/m3

B_c- outside diameter of pipe in m

C_e- coefficient which is 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 -height that conduit projects above or below the natural ground level.

The settlement ratio, r_s, is a measure of the settlement of the founding material under the conduit. Values of this parameter for different soil types are given in Table 7.

TABLE 7: VALUES OF SETTLEMENT RATIO

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 5, are defined by the pipe projection relative to the original ground level. Positive projection occurs when the top of the conduit projects above the natural ground level. Zero projection occurs when the top of conduit is level with natural ground and the load on the pipe is the geostatic load. This also applies if the side fill to a sub-trench is compacted to the same density as the undisturbed soil in which the trench has been dug. Negative projection occurs when the top of the conduit is below the natural ground level. As the sub-trench depth increases, so this condition approaches the complete trench condition. That portion of the load in the sub-trench will be less than the geostatic load and the load above the sub-trench will act as a surcharge.

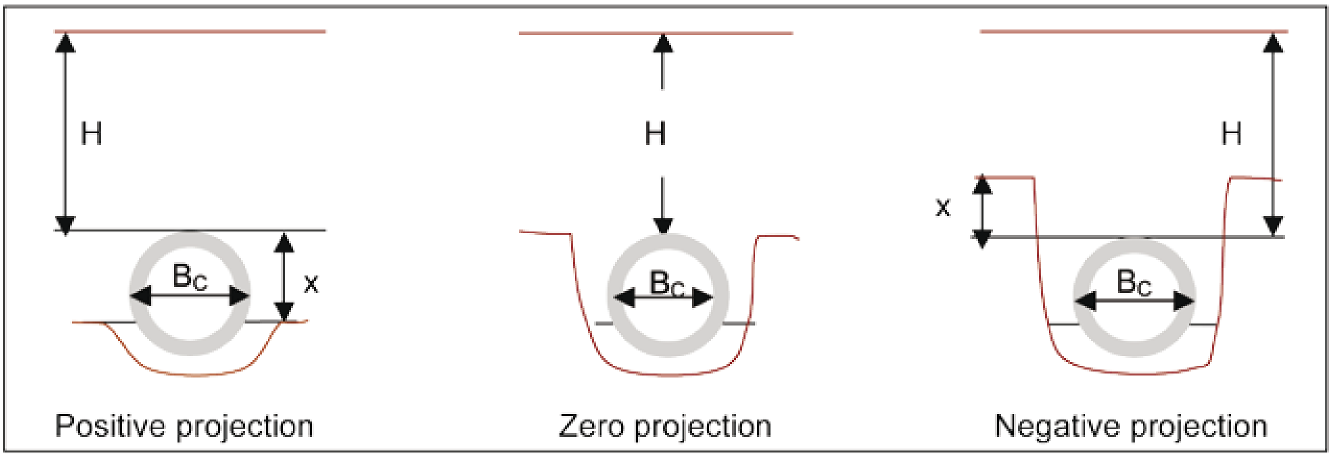


FIGURE 5: TYPES OF EMBANKMENT INSTALLATION

For all practical purposes the earth loading from a positive projecting installation will have a maximum value when the prs ratio has a value of 1.0, and can be calculated from equations (25) and (26) where the relationship between load and fill height is linear.

$W_e = 1.69\ \gamma\ B_c\ H$ in sandy soils (25)

$W_e = 1.54\ \gamma\ B_c\ H$ in clayey soils (26)

Where these symbols are defined above and in figure 5.

As equations (25) and (26) give upper limits, the smaller of the load calculated from them or equations (22) or (23) should be used. Loads from positive projecting embankment loading on circular pipes are presented in Table 8. The un-shaded area to the left is determined from equation (24) using the relevant earth loading coefficient. The shaded area shows where the relationship between fill height and load is linear and is calculated using equation (25).

TABLE 8: POSITIVE PROJECTION LOADING IN KN/M ON ANY BURIED CONDUIT

Diameter mm		Height of backfill above top of pipe in metres										
Inside	Outside	0.6	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0
225	259	5	9	13	17	22	26	31	35	44	52	61
300	345	6	12	17	23	29	35	41	47	58	70	82
375	431	7	14	22	29	36	44	51	58	73	87	102
450	518	8	15	26	35	44	52	61	70	87	105	122
525	604	9	17	30	41	51	61	71	82	102	122	143
600	690	10	18	32	47	58	70	82	93	117	140	163
675	776	11	20	35	52	66	79	92	105	131	157	184
750	863	12	22	37	56	73	87	102	117	146	175	204
825	949	13	23	39	59	80	96	112	128	160	192	224
900	1035	14	25	42	61	85	105	122	140	175	210	245
1050	1208	16	28	46	68	92	121	143	163	204	245	286
1200	1380	18	32	51	74	100	129	163	187	233	280	327
1350	1620	21	37	58	83	111	142	177	216	274	329	383
1500	1800	23	40	64	90	119	151	187	228	304	365	426
1650	1980	25	44	69	97	127	161	199	240	335	402	468
1800	2160	27	47	74	104	136	171	210	252	348	438	511

Notes:

1. Table is for non-cohesive material with unit weight 20 kN/m^3, $K\mu = 0.19$ and $pr_s = 1.0$.
2. Table can be used for other soil densities by multiplying load by actual density /20.
3. Table can be used for different values of pr_s as follows:
   (a) If load is to the left of shaded area, it may be used irrespective of the pr_s value.
   (b) If load falls in shaded area, multiply the value by the factors in Table 9.

TABLE 9: ADJUSTMENT FACTORS FOR SETTLEMENT AND PROJECTION VALUES

pr_s	1.0	0.7	0.5	0.3	0.1
Factor	1.00	0.94	0.90	0.83	0.74

3.5. Jacked Pipeline Installation

When conduits are placed under existing roadways, railways or other areas that are already developed, digging trenches can be extremely disruptive and the indirect costs enormous. An alternative to this is pipe jacking. When a pipe is jacked the soil load above it is reduced by both friction and cohesion that develop between the column of earth directly on top of it and the columns of earth either side of it.

The jacking technique provides a practical and cost-effective way for an outfall to cross existing services by providing a sleeve through which the sewer can be installed at a later stage. When the outfall is greater than 900mm in diameter it is common practice to jack the sewer pipes themselves and thus eliminate the expense of the sleeve. If this is done it is recommended that the section of sewer being jacked is one diameter larger that the sections of sewer being installed using open trench techniques. This allows for the correction of misalignment, if it does occur by casting an invert that is to level without compromising capacity.

The jacking technique involves:

- Excavating a pit at the beginning and end of the proposed line.
- Constructing a launching pad in the entry pit.
- Pushing a jacking shield against the face of the pit.
- Making an excavation slightly larger than the shield just ahead of it by tunnelling through the soil from the protection provided by the shield.
- Pushing conduits into the tunnel as the excavation progresses.
- Grouting the space between the outside of the conduit and the tunnel.

The vertical load on the conduits in a jacked installation is significantly less than that in a trench installation. This load is dependent on the outside dimension of the conduit because the soil above the conduit is undisturbed the load is reduced by both friction and cohesion. Although the calculation of these earth loads is complex, their maximum value is easy to calculate. Once the fill height over a pipe exceeds about 10 times its outside diameter, full arching will take place and no matter how much higher the fill, there will be no further increase in the load. This maximum load can be calculated from:

$$W_E = 2.63\gamma B_c^2 \text{ in sandy conditions} \quad (27)$$

$$W_E = 3.84\gamma B_c^2 \text{ in clayey conditions} \quad (28)$$

Where these symbols are as defined in sections 3.3 and 3.4 above.

The use of these limiting values will give product strengths that are safe but still economical. It should be noted that these equations are the same as equations (22) and (23) with B_t taken as equal to B_c.

3.6. Flexible Pipes

Under certain conditions the use of a flexible pipe may be considered as an alternative to a rigid concrete pipe. Such a comparison must take into account all the relevant factors. Irrespective of the installation type used, the load on a flexible pipe will always be less than the geostatic load calculated by using the outside pipe diameter. This is because flexible pipe deforms under load, settles more than the soil on either side of it, and sheds the load onto this material. The soil loads on flexible pipes can conservatively be calculated from:

$$W_E = \gamma B_c H \quad (29)$$

In practice, if the soil load on a flexible pipe increased, it will deform further, causing the frictional forces between the columns of soil to increase and the maximum load will actually have an upper limit given by this formula. The critical structural parameter for flexible pipes is the composite stiffness of the surrounding embedment material and natural soil that is needed to ensure that the pipes do not deform excessively, rather than the pipe strength. Thus the effort shifts from ensuring that the pipe is strong enough to ensuring that the surrounding soil is strong and stiff enough.

3.7. Traffic Loading

Where a sewer is installed under transportation routes, the axle spacing and loads and wheel spacing, loads and contact areas of the vehicles using them, the type of riding surface and height of fill over the conduits should be determined.

Concrete pipes most frequently subject to severe live loads are those under roads. In this manual 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 100kN 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 30kN/m^2 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.

$$Q_b = 1.25(90 + 12L_s^{1.8}) \qquad (30)$$

Where Q_b - equivalent point load

L_s - effective span of conduit in 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 6 illustrates such a wheel configuration.

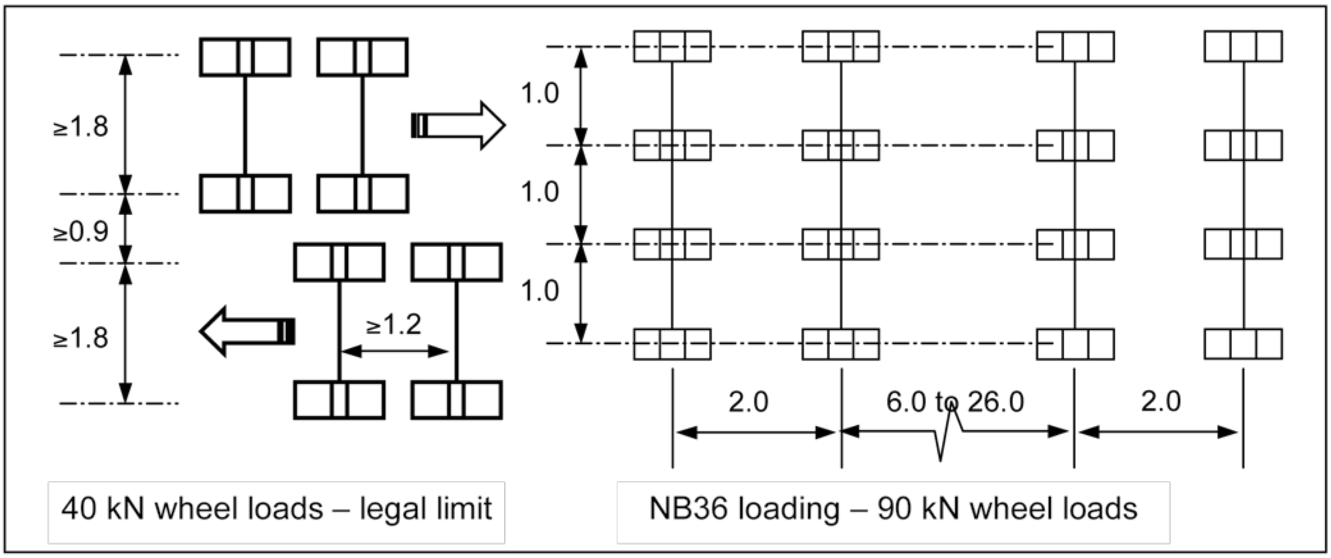


FIGURE 6: WHEEL PATTERN FOR TRAFFIC LOADING ON ROADS

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 40kN and NB36 wheel loads with the configurations shown in Figure 6 are presented in Tables 10 and 11 respectively. The loads presented in Table 10 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 10 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 inadvisable to build sewers with less than 600mm of cover. Refer to the notes below the table when the fill height is less than 600mm or less than half the pipe diameter.

TABLE 10: LOADS IN KN/M ON BURIED CONDUIT FROM GROUP OF 40 KN WHEELS

Diameter mm		Fill height over pipes in 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	1035	24.5	14.3	8.5	5.6	4.0	3.0	2.3	1.8	1.2	0.9	0.7
1 050	1208	28.5	16.7	9.9	6.6	4.7	3.5	2.7	2.1	1.4	1.0	0.8
1 200	1380	32.6	19.1	11.4	7.5	5.3	4.0	3.1	2.5	1.7	1.2	0.9
1 350	1620	38.3	22.4	13.3	8.8	6.3	4.7	3.6	2.9	1.9	1.4	1.0
1 500	1800	42.6	24.9	14.8	9.8	7.0	5.2	4.0	3.2	2.2	1.6	1.2
1 650	1980	46.8	27.4	16.3	10.8	7.7	5.7	4.4	3.5	2.4	1.7	1.6
1 800	2160	51.1	29.9	17.8	11.8	8.4	6.3	4.9	3.9	2.6	1.9	1.4

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: LOADS IN KN/M ON BURIED PIPES FROM NB36 GROUP OF 90KN WHEELS

Diameter mm		Fill height over pipes in m											NB36 pt 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	0.345	26	12	7	4	3	2	1	1	1	1	0	114
375	0.431	31	15	8	5	3	2	2	1	1	1	0	115
450	0.518	35	17	10	6	4	3	2	2	1	1	1	116
525	0.604	39	19	11	7	5	3	2	2	1	1	1	117
600	0.690	43	22	12	8	5	4	3	2	1	1	1	118
750	0.863	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
1 650	1.980	68	46	34	22	15	11	9	7	5	3	3	149
1 800	2.160	69	49	37	24	17	13	10	8	5	4	3	156

The NB36 vehicle travels slowly and thus impact does not need to be considered.

4. CONCRETE PIPE STRENGTHS

4.1. External loads

The required strength for a buried concrete pipe is determined by dividing the installed load by a bedding factor. Factory test loads and reactions are concentrated, but field loads and reactions have a parabolic or radial distribution. It is assumed that in-situ loads are uniformly distributed over a pipe and that reactions are either parabolic or uniform, depending upon the bedding material. A comparison of these loads and reactions is shown in Figure 7.

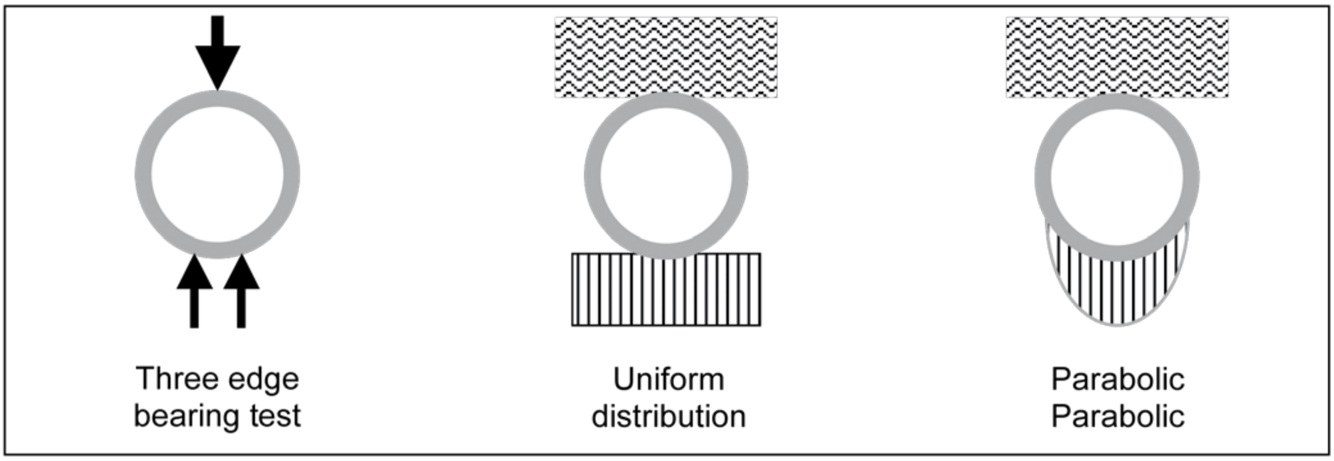


FIGURE 7: FACTORY STRENGTH AS MODEL OF INSTALLED LOAD ON PIPE

Bedding factors are derived for standard bedding classes and are detailed in the Concrete Pipe and Portal Culvert Handbook (13). The bedding factors for a trench installation assume that there is a vertical reaction only and no lateral support to the pipe. For an embankment installation lateral support is considered, hence these bedding factors are somewhat higher than the trench factors. The factors given in Table 12 are adequate for standard installations.

TABLE 12: BEDDING FACTORS FOR CONCRETE PIPE

Bedding details			Installation details		
Class	Material	Angle	Trench	Embankment	Jacking
A	Reinforced concrete	180°	3.4	4.8	n/a
A	Concrete/Soilcrete	180°	2.6	3.9	3.0
B	Granular	180°	2.0	2.4	n/a
B	Shaped sub-grade	120°	1.9	n/a	1.9
C	Granular	60°	1.5	2.0	n/a
D	Granular	0°	1.1	1.2	n/a

Note:

1. The bedding angle refers to the width of bedding support under the pipe
2. Class D bedding should only be used when suitable bedding material is not available.
3. Class A concrete bedding should not be used unless there are special requirements.
4. For zero and negative projection embankment installations use trench bedding factors.
5. For positive projection conditions, where greater accuracy is required the bedding factors can be calculated using the procedure described elsewhere (13, p37-38)
6. Details of the different bedding are covered in section 4.5.

4.2. Internal Pressure

Where a sewer operates under internal pressure there are two conditions that could apply: a gravity sewer that flows full such as a siphon, of where a pressure head is applied as with a rising main. The critical condition for siphons is static head excluding losses due to friction. Dynamic factors that can cause pressure surges above and below the static or working head are the critical condition for rising mains. A sewer that operates as a pressure pipeline is usually a gravity system or a siphon where surges cannot develop. Section 4.3 gives the factors of safety that should be applied to such a section of sewer.

4.3. Safety Factors

In the calculation of the loads and strengths of pipes, maximum and minimum values are used respectively. In addition, the pipe strengths used are determined from a proof load rather than an ultimate load. Hence, if all the relevant factors are considered when determining pipe strength it should not be necessary to apply safety factors. However, where this is not the case, safety factors should be applied. The safety factors to be used in specifying a pipe depend upon:

- field working conditions
- degree and competence of supervision
- height of the fill above the pipe
- whether or not there are corrosive elements in the transported fluid, or groundwater

The choice and application of safety factors is left to the discretion of the designer. For external loads, either each load is considered independently and a factor of safety applied directly to each load, or the factor of safety is applied to the pipe strength. Recommended values are presented in Table 13.

TABLE 13 RECOMMENDED SAFETY FACTORS FOR VERTICAL LOADS

Pipe Application	Factor of safety	
	Reinforced	Non-reinforced
Storm water drainage	1.0	1.3
Sewer pipes without sacrificial layer	1.3	1.7
Pipes laid in corrosive ground condition	1.3	1.7
Sewer pipes with sacrificial layer	1.0	1.3

For pressure pipelines operating under gravity only, a safety factor of 1.5 is normally used. Where additional energy is added and the pressures along the pipeline have been calculated accurately, taking into account surge and water hammer effects, the line is usually divided into pressure zones or reaches. The factor of safety at the lowest section of any zone is usually taken as 1.0, resulting in a higher factor of safety along the other sections.

4.4. Selecting Concrete Pipe Class

Currently there are two South African national standards applicable to the manufacture of concrete pipes:

SANS 676 - Reinforced concrete pressure pipes and

SANS 677 - Concrete non-pressure pipes

Non-pressure pipes are classified by their crushing strength under a vertical knife-edge test-load. There are two crushing load test configurations: the two-edge and the three-edge bearing tests. The three-edge bearing test, shown in Figure 7, is preferred as the pipe is firmly held in place by two bottom bearers before and during the test.

The proof load is defined as the line load that a pipe can sustain without the development of cracks of width greater than 0.25 mm over a distance greater than 300 mm in a two or three edge bearing test. Non-reinforced pipes are not permitted to crack under proof load.

The ultimate load is defined as the line load that the pipe will support in a two or three edge bearing test without collapsing. This is at least 1.25 times the proof load.

The D-load (diameter load) is defined as the standard crushing load strength designation. This is the proof load in kN per metre of pipe length, per metre of nominal pipe diameter. The standard D-load classes with their proof and ultimate loads are presented in Table 14.

TABLE 14: STANDARD D-LOAD CLASSIFICATION FOR CONCRETE PIPES

Pipe Class D-Load	Proof load kN/m	Ultimate load- kN/m
25D	25xD	31.25xD
50D	50xD	62.50xD
75D	75xD	93.75xD
100D	100xD	125.00xD

Example

For a 1050 mm diameter 75D pipe

proof load = 1.05 x 75 = 78.75 kN/m

ultimate load = 1.05 x 93.75 = 98.44 kN/m

Pipes made in accordance with SANS 677 are divided into two types:

- SC pipes for stormwater and culvert applications
- SI pipes for sewer and irrigation applications.

SC pipes are used in applications where there is no internal pressure. A sample (±2%) of pipes is subjected to the proof load test to ensure that they meet this requirement. SI pipes, on the other hand, are used in applications where internal pressure could develop under certain conditions (such as when blockages occur). To ensure that pipes will meet this possible condition and ensure that the joints are watertight, a sample of pipes is hydrostatically tested to a pressure of 140 kPa in addition to the crushing strength test.

4.4.1. External load

The relationship between the factory test load and installed field load is given by the equation developed by Marston and Spangler as follows:

$$W_T = [(W_e/B_{Fe}) + (W_t/B_{Ft})]\ FS \tag{31}$$

Where W_T - required proof load for 0.25 mm crack (kN/m)

W_e - external soil load (kN/m) as calculated or using Tables 5, 6 or 8

W_t - external traffic load (kN/m) as calculated or using Tables 10 or 11

B_{Fe} – bedding factor for soil loads from Table 12

B_{Ft} – bedding factor for traffic loads as discussed below

FS - factor of safety as given in Table 13

Sewers are usually placed with at least 600 mm of cover over them, except in the case of larger diameter pipes where this cover should be at least half the pipe diameter. Under these circumstances any traffic loads are distributed over the whole outside pipe diameter. The formula then reduces to:

$$W_T = (W_l/B_F)\ FS \tag{32}$$

Where W_l – total external load in kN/m

B_F – bedding factor from Table 12

The other symbols are defined earlier.

FS has a value of 1.0 and 1.3 for reinforced and non-reinforced pipes, respectively.

The pipe class is selected so that:

$$S \geq W_T / ND \tag{33}$$

Where S - proof load of a standard D-load class pipe (kN/m)

ND - is nominal pipe diameter in m

Pipes installed with less cover must have 180° of bedding support under them and the appropriate live load bedding factor is given in Table 12 when the angle at which the live load is distributed over the pipes is equal to the bedding angle in degrees.

4.4.2. Internal pressure

The selection of the pressure class is made as follows:

$$t = p \times FS \qquad (34)$$

Where t - required test pressure (kPa)

p - design pressure in pipeline

FS- factor of safety

The pipe class is selected so that

$$T \geq t \qquad (35)$$

Where T - test pressure of standard pressure class pipe (kPa)

The standard pressure classes for concrete pipes given in SANS 676 are T2, T4, T6, T8 and T10, where the T designation refers to the factory test pressure in bar. (A bar is equivalent to 100 kPa or 10 m of water head.)

4.4.3. Combined Internal Pressure and External Load

Where pipes will be subject to combined external load and internal pressure, the following formula is used for pipe selection:

$$T = t / (1-(W_T /S)^2) \qquad (36)$$

When selecting a pipe for these conditions, a balance between internal test pressure, T and the external crushing load, S, should be sought. A pipe should not be selected to withstand very high pressure and a very low vertical load or vice versa, as this is uneconomic.

The maximum internal pressure to which a sewer pipe can be subjected will occur if there is a blockage and the pressure accumulates until it is relieved at an upstream manhole. Hence, for almost all applications the SI factory hydrostatic test to a pressure of 140kPa is adequate.

4.5. Pipe Bedding and Installation Details

Details of an installed pipe and the surrounding materials are illustrated in Figure 8.

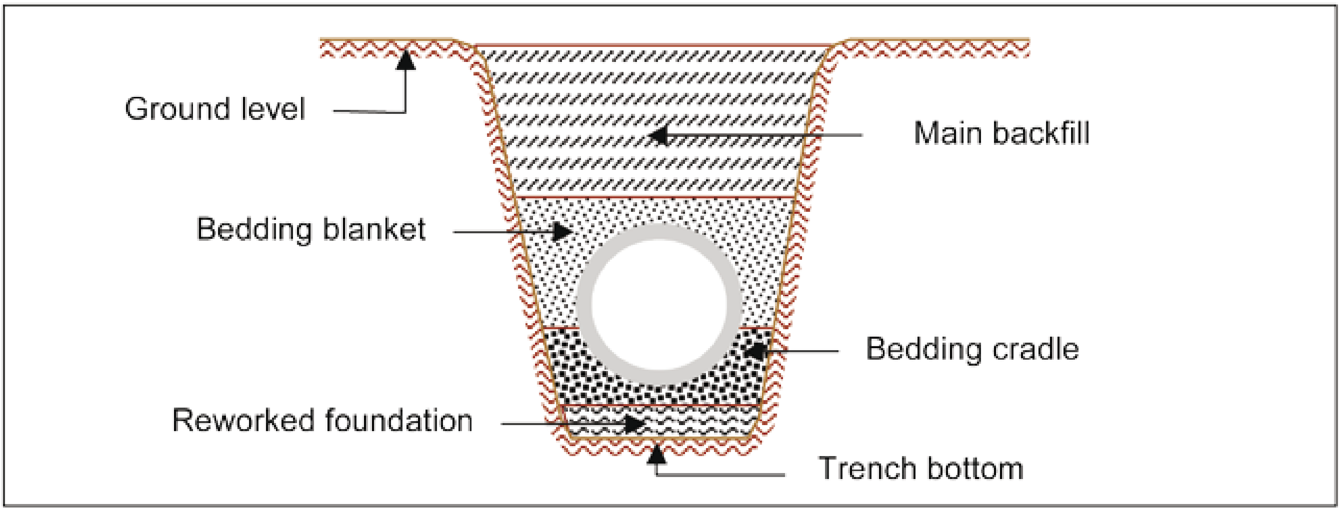


FIGURE 8: PIPE BEDDING AND INSTALLATION TERMINOLOGY

The trench bottom is not the pipe bedding; it is the foundation for the pipe bedding. If the bearing capacity of the trench bottom is inadequate to support the bedding or it is uneven it must be reworked and compacted to provide a level surface with the required bearing capacity. If this cannot be achieved with the insitu material then material must be imported, placed and compacted to provide adequate support.

The bedding cradle enhances the load carrying capacity of the pipe and cushions it from the trench floor. When selecting materials for the bedding cradle the designer must consider the interface between this material and the surrounding natural material. The bedding cradle should consist of cohesion-less granular material that is free draining with a grading and particle size that will not allow the surrounding material to be washed into it. If there is the possibility of this happening the bedding and the pipe should be wrapped with a geofabric to prevent this. The ingress of fine material into the bedding cradle will result in a loss of support and reduce the load carrying capacity of the pipe

The bedding blanket cushions the pipe from the main backfill. It should consist of a fine material free of organic matter and clay lumps. It must be easily compactable hence a cohesive material with a PI in excess of ± 18 would be unsuitable.

Detailed information concerning the different bedding classes and their variations is provided in the Concrete Pipe and Portal Culvert Handbook (13). The difference between the Class B, C and D beddings is in the width of support from the cradle and its thickness under the pipe.

As a reference, the different variations to the class B trench bedding are detailed in Figure 9. The bedding angle of the standard class B granular bedding is 180° and pressure distribution under the pipe is assumed to be parabolic. In the case of the fully encased case a bedding factor of 2.2 can be used. The shaped sub-grade has a bedding angle of 90°. It can be difficult to construct and is not recommended. The selection, placement and compaction of the granular material must be carried out so that this assumptions are not compromised.

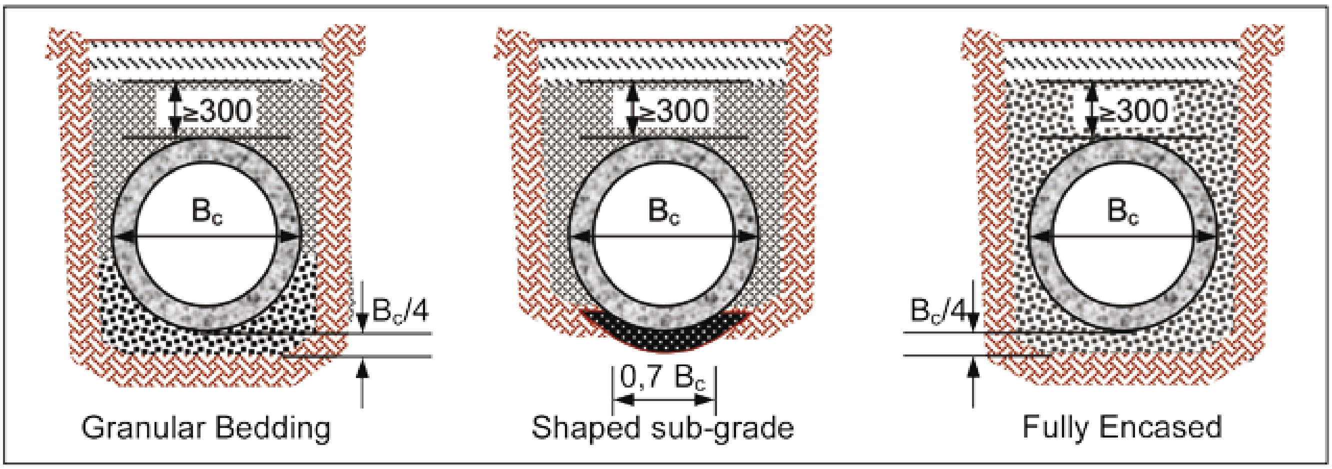


FIGURE 9: CLASS B TRENCH BEDDINGS

4.6. Jacking Conditions

When pipes are jacked the excavation is slightly larger than the external diameter of the pipe. However, the process of installing a pipe ensures that positive contact is obtained around the bottom portion of the pipe and that ideal bedding conditions are obtained. If the pipe carries all or part of the vertical earth load it is appropriate to use trench bedding factors. These will depend on the width of contact between the outside of the pipe and the material through which the pipe is jacked. As this contact will usually be over at least 120°, a bedding factor of 1.9 can be used.

When determining the bedding factor, the behaviour of the in-situ material after the jacking has been completed the post installation treatment given to the void between the pipe and the excavation should be considered. If this is grouted, a bedding factor of 3.0 can be used.

5. CORROSION PREDICTION

5.1. Background

Concrete is the most frequently used material for the manufacture of outfall sewers. Under certain conditions concrete sewers may be subject to corrosion from sulphuric acid (H_2SO_4) formed as a result of bacterial action. The first indication of this corrosion is a white efflorescence above the water line that appears several months after the sewer has been put into operation. Thereafter deterioration may be rapid with the concrete surface becoming soft and putty-like and the aggregate falling out.

5.2. Corrosion Mechanism

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. These are illustrated in Figure 10 below.

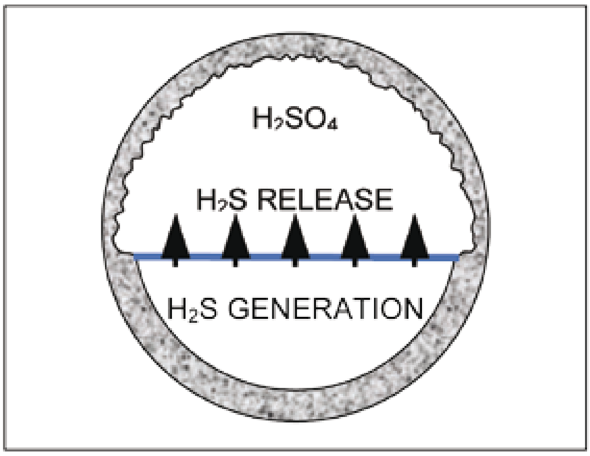


FIGURE 10: FACTORS CONTRIBUTING TO CORROSION MECHANISM

The most important factors contributing to H_2S generation in the effluent are:

- 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:

- 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:

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

If there is insufficient oxygen in the effluent (less than 1mg/lt) the bacteria that live in the slimes layer on the sewer walls strip the oxygen from the sulphates in the effluent to form sulphides. The first set of factors influences the rate at which this occurs. When there is an imbalance of H_2S in the sewage and the sewer atmosphere this gas will come out of solution until there is equilibrium between the gas concentration in the sewage and the sewer atmosphere. The second set of factors influences this. The H_2S released into the sewer atmosphere is absorbed into the moisture on the sewer walls and is oxidised by another set of bacteria to H_2SO_4 under the influence of the third set of factors.

If the pipe material is alkaline or has an alkaline component, this will react with the acid producing deterioration. This happens in concrete pipes where the acid formed attacks the alkaline cement in the concrete above the water line. If an inert aggregate is used there is aggregate fallout when the binder corrodes. This exposes more of the binder that in turn is corroded by the acid. The deterioration of the pipe wall can be rapid. If concrete is made using an alkaline calcareous aggregate, the acid attack is spread over both binder and aggregate, the aggregate fallout problem is minimised and the rate at which the sewer wall deteriorates is reduced.

5.3. Corrosion Prediction Theory

Pomeroy and Kienow (6) developed a quantitative method for predicting the rate of sulphide generation and the resultant rate of concrete corrosion. This later became know as the Life Factor Method (LFM). In 1984 the American Concrete Pipe Association (ACPA) published the "Design Manual Sulfide and Corrosion Prediction and Control" (7). This quantified the LFM by providing equations for predicting corrosion in concrete sewers based on the biological composition of the effluent, the system hydraulics and the alkalinity of the concrete used. The final output is the required cover to pipe reinforcement. In South Africa the application of LFM has been slightly different. The cover for reinforcement has been divided into two components: the "sacrificial layer", which copes with the corrosion and is determined from the LFM, and the standard cover that ensures that the pipe will remain serviceable when the "sacrificial layer" has been corroded away.

5.4. Sulphide Generation

Pomeroy and Parkhurst (8) assumed that the concentration of nutrients supporting sulphate-reducing bacteria living in the slimes layer is proportional to the biochemical oxygen demand (BOD) of most municipal wastewaters. This, in turn, is dependent on the effluent temperature that varies during the year and influences biological factors leading to sulphide generation and release. The biodegradable material in the effluent provides the energy that converts the sulphate to sulphide. The quantity available for doing this is measured by the 5 day biochemical oxygen demand test (BOD_5) (7, p2-3). Sulphide generation may occur at temperatures above 15°C (2, p2-3). As the designer is interested in the cumulative annual effect, sulphide generation is determined from the effective biochemical oxygen demand.

$$[EBOD] = [BOD_5] \times 1.07\ (T-20) \qquad (7, p2-4) \qquad (37)$$

Where [EBOD] - effective $[BOD_5]$ in mg/l

$[BOD_5]$ – composite average BOD_5 in mg/l

T – wastewater temperature in °C

Sewers that feed into wastewater treatment plants should have the wastewater analysis examined so that EBOD values for any period during the year can be estimated from the recorded BOD_5 values and temperatures. If there were insufficient data to calculate the average annual sulphide concentration this can be determined by using correction factors to determine the climactic effective BOD. The equation for calculating this (38):

$$[EBOD]_c = [BOD_5]_c \times 1.07^{(T_c-20)} \qquad (2, p2-5) \qquad (38)$$

Where $[EBOD]_c$ – climactic effective $[BOD_5]$ in mg/l

$[BOD_5]_c$ – climactic composite average BOD_5 in mg/l

T_c – climatic wastewater temperature in °C

The climactic conditions are defined as the average $[BOD_5]$ values during the warmest six-hour flow period of the day in the warmest three months of the year. These climactic values can be approximated as 1.25 times a flow-proportioned 24 hour composite BOD. Wherever possible these values should be confirmed by actual measurements. In South Africa, the chemical oxygen demand (COD) test is usually done in lieu of the BOD test, as it is much quicker. The relationship between these two parameters varies between sewer systems, so in critical situations it should be determined. The COD value is usually about double the BOD value.

When a sewer flows full, as in the case of a force main or siphon, there is no air/water interface, so there is no release of the H_2S generated and it accumulates in the effluent. The accumulation of gas generated in such a closed system is calculated using the equation:

$$S = M\, t\, [EBOD]\, (4/D + 1.57) \qquad (2, p21) \qquad (39)$$

Where S - sulphide accumulation in effluent in mg/l in time t

M - sulphide flux coefficient in a full pipe m/hr

t - flow time in sewer reach in hr

D - internal pipe diameter pipe in m

The downstream sulphide concentration is calculated from:

$$S_2 = S_1 + M\, (t_2 - t_1)\, [EBOD]\, (4/D + 1.57) \qquad (2, p21) \qquad (40)$$

Where S_2 - sulphide concentration at time t_2 in effluent in mg/l

S_1- sulphide concentration at time t_1 in effluent in mg/l

The other terms are defined above.

However, when a sewer flows partly full, as is usually the situation there is an air/water interface, effluent entrains air and releases some of the gases generated. This means that the sulphide accumulation in the effluent is slower than it would be in an equivalent system with the pipe flowing completely full. Assuming there is little or no dissolved oxygen in the sewage, sulphide [S] generation can be determined from:

$$d[S]/dt = M' (EBOD/R) - m\, ([S]\, (sv)^{3/8}/d_m) \qquad (2, p20) \qquad (41)$$

Where d[S]/dt – rate of change of total sulphide in mg/l

M' – effective sulphide flux coefficient for H_2S generated in slimes layer when there is open channel flow in a gravity system in m/hr

EBOD – effective BOD = BOD x $1.07^{(T-20)}$ in mg/l

T – wastewater temperature in °C

R – hydraulic radius of flow area in m

m – sulphide loss coefficient due to oxidation and escape into atmosphere

[S] – total sulphide concentration in mg/l

d_m – mean hydraulic depth = flow area / flow width in m

v – mean flow velocity in m/s

s – slope of energy grade line in m/m

There will be a theoretical upper limit to the sulphide concentration in the effluent when the losses equal the [S] generation and this will occur when:

$$[S]_{lim} = (M'/m)\, EBOD\, (sv)^{-3/8}\, (P/W) \qquad (2, p20) \qquad (42)$$

Where $[S]_{lim}$– upper limit for H_2S generation in an open channel gravity system in mg/l

P – wetted perimeter in m

W – flow width in m

The limiting value of [S] is approached asymptotically in a sewer flowing as an open channel gravity system and will not be reached under these conditions. However, when a sewer flowing as an open channel gravity system receives the discharge from a closed system, the [S] concentration may exceed the limiting value and the H_2S will be stripped rapidly from the effluent to re-establish equilibrium between gas concentrations in the sewer atmosphere and effluent. The downstream sulphide concentration is calculated from:

$$S_2 = S_{lim}-(S_{lim}-S_1)/(\log^{-1}[m(sv)^{-3/8}t/(2.31dm)] \qquad (2, p20) \qquad (43)$$

Where S_2 – downstream sulphide concentration in mg/l

S_1 – upstream sulphide concentration in mg/l

Several sulphur compounds are formed in sewage as a result of biological activity, but only gaseous H_2S that can escape, and when it does, it causes odour and corrosion problems. For a corrosion analysis, it is therefore necessary to determine the proportion of [S] formed that is dissolved and the proportion of this dissolved sulphide [DS] that is H_2S.

The proportion of [S] that is [DS] is variable and dependent upon effluent pH and its metals content. This relationship has to be measured as it cannot be calculated and is peculiar to the situation being analysed. The ACPA manual (7, p3-11) suggests that for design purposes, in the absence of actual values, that a conservative estimate of 0.2 mg/l should be used for the insoluble sulphide in the effluent. The proportion of [DS] that is H_2S is directly related to the pH of the effluent. At pH 9.0 it is 0%; at pH 8.0 it is 10%; at pH 7.0 it is 50%; at pH 6.0 it is 90% and at pH 5.0 it is 100%.

5.5. Hydrogen Sulphide Release

This only occurs from a sewer that is flowing partly full when there is no equilibrium between the concentrations of H_2S in the sewer atmosphere and effluent. The H_2S concentration in the sewer atmosphere is invariably much lower than its equilibrium concentration in the effluent so the release of gas is from the effluent to the sewer atmosphere.

Under typical sewer conditions, once the [DS] in the effluent has been determined as described above, the rate of H_2S release, called the H_2S flux, can be calculated as follows:

$$\varphi_{sf} = 0,69\ (sv)^{3/8} J\ [DS] \qquad (2, p15) \qquad (44)$$

Where φ_{sf} - H_2S flux from stream surface, g/m^2/h

s - energy gradient of wastewater stream, m/m

v - stream velocity, m/s

J - fraction of DS present as H_2S, that is a function of pH

[DS]-average annual dissolved sulphide concentration in wastewater, mg/l

(0,2 to 0,3 mg/l less than the total sulphide concentration)

The absorption of this H_2S into the moisture layer on the wall of the sewer is determined by a variation of equation 44:

$$\varphi_{sw} = 0,69\ (sv)^{3/8}\ J\ [DS]\ (W/P') \qquad (2, p16) \qquad (45)$$

Where φ_{sw} - H_2S flux to the pipe wall, g/m^2/h

W/P'- ratio of stream width to pipe wall perimeter above water surface.

This assumes that all the H_2S released is absorbed into the moisture layer.

5.6. Acid Formation and Concrete Corrosion

Concrete corrosion rates are determined from the rate at which the H_2S flux to the pipe wall is oxidised to H_2SO_4. According to the EPA Manual (2, p23), "34 g of H_2S are required to produce sufficient H_2SO_4 to neutralise 100 g of alkalinity expressed as calcium carbonate ($CaCO_3$) equivalent". If all the φsw is oxidised, the annual corrosion rate for the concrete is predicted as follows:

$$C_{avg} = (11.5k/A)\ \varphi_{sw} \qquad (2, p23) \qquad (46)$$

Where C_{avg} - average corrosion rate, mm/year

K - efficiency coefficient for acid reaction based on estimated fraction of acid remaining on sewer wall. May be as low as 0,3 and will approach 1,0 for a complete acid reaction.

A - Alkalinity of concrete expressed ascalcium carbonate ($CaCO_3$) equivalent; It varies from ± 0,16 for siliceous aggregate concrete to ± 0,9 for calcareous aggregate concrete; 0,4 for mortar linings.

11,5 - converts φ_{sw} in g/m^2/h, into Cavg, in mm/year

When equation (47) is used, together with the design life, to determine the required additional concrete cover over reinforcement to ensure that a sewer is serviceable at the end of this period, the Life Factor Method (LFM) formula is used as follows:

$$Az = 11.5\ k\ \varphi_{sw}\ L \qquad (7, p4\text{-}4) \qquad (47)$$

Where z - additional concrete cover required over reinforcement in (mm)

L - required design life of sewer in years

The term Az, called the Life Factor, is used for comparing different concrete mixes. The left hand side of the equation describes the pipe material properties in terms of additional cover over reinforcement and alkalinity and the right hand side describes the conditions within the sewer in terms of effluent properties, flow characteristics, sewer atmosphere and the required service.

There are three options for preventing or minimising the corrosion in concrete sewers:

- preventing acid formation;
- modifying the concrete; and
- protecting the concrete.

Acid formation can be prevented or minimized by adjusting the hydraulic design of the sewer. If self cleansing gradients at all flow levels can be maintained throughout the whole system then sufficient oxygen (> 1.0mg/lt) should be entrained in the effluent to prevent the formation of sulphides. However, due to physical constraints this is not always possible and some corrosion should be anticipated. In most sewers, modifying the concrete by changing the concrete components and/or providing additional cover over reinforcement is the most cost-effective option. Protecting concrete by using an inert lining or coating is effective, but only economically justifiable when severe corrosion is predicted.

6. CORROSION CONTROL

6.1. Developments in South Africa

Since the 1960's most concrete sewers installed in South Africa were made using calcareous aggregates, usually dolomitic (DOL), and the sacrificial layer principle. This solution followed the recommendations of a 1959 publication (9) published by the Council for Scientific and Industrial Research (CSIR). The state of the sewers in Johannesburg, where the first calcareous pipes were laid in 1960, indicates that this results in a considerable increase in a sewer's life. There have been few reports of serious problems in sewers made using this approach and where they have occurred they can be explained and with the present know how could have been prevented.

However, concern has been expressed by local authorities and consultants that the "dolomitic aggregate" solution might not be adequate for certain sewers in the long-term. This was substantiated by applying the method (developed by Pomeroy and Parkhurst (8) in the USA that quantifies the biogenic corrosion of concrete) to several sewers where severe corrosion is anticipated.

When the CSIR undertook a literature search during the 1980s, no reference could be found to field trials that calibrated the actual performance of various sewer materials (10). Following several meetings of interested parties, a steering committee was formed and a decision taken to include an experimental section, with a bypass, as part of a sewer being installed at Virginia in the Free State. The LFM indicated that the conditions anticipated in this sewer were so corrosive that the traditional solution would be unsuitable and a cementitious pipe would require an inert lining or coating. There have been three phases in the monitoring of material performance and conditions in this sewer as follows.

Phase One (run by the CSIR) monitored conditions in the sewer and the performance of traditional sewer pipe materials, including their responses to pure acid attack, in a laboratory.

Phase Two (run by the UCT) continued monitoring sewer conditions and investigated ways of simulating these conditions in a laboratory.

Based on the five year findings, the possibility of using a host pipe made of one type of concrete to provide strength and an additional layer of another concrete to cope with the corrosion, was investigated. An effective technique for manufacturing such a pipe was developed and since 1997 has been used in many of the major outfall sewers in South Africa. The most commonly used combination of materials is PC/SIL concrete in the host pipe and CAC/DOL concrete for corrosion control layer.

Phase Three (run by UCT and an independent consultant) continues the second phase, measuring the actual corrosion that occurred during the first two phase, supervising the rehabilitation of the experimental section, and measuring the actual corrosion on various new materials to be calibrated for use in the LFM.

At the time of writing, the experimental section of sewer had been rehabilitated and the actual corrosion on the samples installed during phase one measured (11) as given in Table 15 below. From this table it can be seen that average corrosion rates measured after 14 years for all the materials were greater than the estimates made following earlier inspections. As these measurements were taken using samples removed from the sewer actual wall, wall thicknesses were determined to calculate actual corrosion rates.

Until recently, the LFM had only been applied to Portland cement (PC) concretes. The corrosion rates measured in this experimental section of sewer meant that the LFM could now be applied to alternative concretes that have effective alkalinity values exceeding unity. An inert material such as HDPE that is used as a lining can be taken as having an apparent infinite 'alkalinity'.

TABLE 15: MEASURED & ESTIMATED CORROSION RATES & MATERIAL FACTORS (11)

Cement/ Aggregate	5 year estimate		12 year estimate		14 year measured		Material factor***
	Total mm	Average mm/year	Total mm	Average mm/year	Total mm	Average mm/year	
PC/SIL	>30	>6,0	>64	>6,0	> 105	> 7.5	1.000
PC/DOL	10 – 15	2 – 3	20 – 30	1,7 – 2,5	43	3.1	0.410
CAC/SIL	5 – 10	1 – 2	10 – 15	0,8 – 1,2	26	1.9	0.250
FC	10 - 12	2 +	20 - 25	1,7 – 2,1			0.270
CAC/DOL *	3,0	0,6	7,2	0,6	8,4	0,6	0.085
CAC/ALM **							0.025

*Values estimated on the basis of other materials and performance of UCT samples in sewer

**Much less than CAC/DOL – no mass loss 17 months in sewer and pH on surface >6,4

***Average of maximum loss at side divided by corresponding value for PC/SIL.

The LFM can thus be used to calculate the required corrosion control layer thickness by incorporating a material factor, MF, that is equal to the ratio between the corrosion rate for the alternative material and a standard concrete made from PC and siliceous aggregate(SIL), for which an ‘A’ value of 0.16 is assumed. The additional materials presented in this table are calcium aluminate cement (CAC), fibre reinforced cement (FC) and aluminate aggregate (ALM). By applying the LFM as described in equation (47) to a particular sewer and assuming an ‘A’ value of 0.16 for a PC/SIL concrete the required corrosion control layer thickness can be established. The annual corrosion rate and additional cover over reinforcement for another concrete are calculated by multiplying the PC/SIL values by the appropriate material factor.

$$(C_{avg})_o = M_F\ (k/A)\ \varphi_{sw} \qquad (48)$$

Where $(C_{avg})_o$ – average annual corrosion rate for alternative material

M_F - material factor for chosen alternative material given in Table 14

$$z_o = M_F\ (k/A)\ \varphi_{sw}\ L \qquad (49)$$

Where z_o – total additional cover required for alternative material

The application of these equations for determining the most cost-effective pipe material is described in the following section.

6.2. Pipe Material Choice for Sewers

There are several concrete pipe alternatives for sewers:

- host pipe and sacrificial layer made from PC/SIL
- host pipe and sacrificial layer made from PC/DOL
- host pipe made from PC/SIL and corrosion control layer from CAC/SIL
- host pipe made from PC/DOL or PC/SIL and corrosion control layer from CAC/DOL
- host pipe made from PC/DOL or PC/SIL and an HDPE lining cast in.

Although a lining of CAC/SIL is technically sound, it is not cost-effective unless it is very expensive to transport dolomitic aggregate to the manufacturing plant.

A distinction has been made between the conventional sacrificial layer, where the whole pipe is made using the same material and the corrosion control layer where two different materials are used. The term ‘corrosion control layer’ is inclusive, as it also refers to an inert lining that is cast into concrete. The relative corrosion rates of these materials are presented in Table 16. By applying the LFM and MF as described above, a technically sound solution that is also the most cost effective for the sewer being considered can be selected.

Pipe size. As the primary function of a sewer is to convey wastewater the first item that should be addressed is the pipe size required. Ideally this should be based on two limiting values related to velocity:

- a minimum value of 0,7m/s at the lowest flow to ensure self-cleansing and
- a maximum value of 0,8 times the critical velocity to prevent excessive turbulence.

These parameters are applicable to the design flows excluding surcharging due to infiltration when the sewer is flowing at a depth that is 61% of diameter. For pipes smaller than 900mm in diameter this upper limit may be impractical and the criteria for selecting the pipe diameter should be changed to ensure that the total energy line is contained within the sewer.

Corrosion control layer. The internal diameter (ID) and hydraulic properties obtained from these calculations should be used in combination with the effluent properties to predict the potential corrosion for the required design life for a PC/SIL concrete. The relative corrosion rates of other concretes should be calculated based on the details presented in Table 15 above. The additional cover with an allowance of 3 to 5mm for an interface, if the corrosion control layer and host pipe are made from different concretes, should be added to the required internal diameter to calculate the host pipe internal diameter.

Pipe strength. A preliminary assessment should be conducted of the pipe class that will be required to deal with the worst-case scenario for the installation conditions. The outside diameter (OD) of the pipe will generally be 1.2 times the indicated host pipe ID.

Mould size. The manufacturers brochures should be consulted to determine the nearest actual external diameter to provide at least the external diameter indicated by these calculations. This should be done for each of the solutions being evaluated as when severe corrosion is predicted there will be a significant difference between the minimum required host pipe outside diameters and this could mean that the pipes using a different corrosion control measures should be made in moulds of different sizes. This is illustrated in the example that follows.

Check using actual dimensions. Once the mould ODs for the alternatives are established, the following exercise should be repeated for each pipe in reverse order:

- for required OD, determine the pipe strength required for the installed conditions
- add required lining thickness to the host pipe ID to determine the actual pipe ID
- check hydraulics of the sewer using the actual ID.

The designer is now in a position to obtain budget prices from suppliers so the alternatives can be compared on an economic basis.

Example: determine the most cost-effective pipe with an actual ID of 900mm for a range of Az values between 5 and 20. Assume that the required pipe class needs a wall of 0.1 x ID.

TABLE 16: COMPARISON OF PIPE LININGS FOR A RANGE OF AZ VALUES

Material	PC/SIL			PC/DOL			CAC/DOL		
Az value	5	10	20	5	10	20	5	10	20
Pipe ID- mm	900	900	900	900	900	900	900	900	900
Corrosion layer-mm	30	60	125	12	24	50	5	9	15
Host ID-mm	960	1020	1150	924	948	1000	910	918	930
Pipe OD-mm	1152	1224	1380	1108	1138	1200	1092	1102	1116
Host pipe - kg	822	928	1179	757	803	892	738	753	771
Corrosion L - kg	226	467	1038	89	178	385	37	66	111
Total - kg	1048	1395	2217	846	981	1277	775	819	882
% Host price	145	193	307	117	136	177	123	132	153

Table 16 above illustrates the impact of corrosion potential on the cost-effectiveness of materials commonly used for corrosion control in South Africa. This relationship can be obtained for other locations by using the relevant costs. As corrosion potential increases, solutions that are more costly to produce become more cost-effective because less material is used.

- If there is any corrosion potential at all the PC/SIL solution will be the most costly.
- For low Az values (<5 to 10) the PC/DOL solution where the host pipe and sacrificial layer is made from the same material is most cost effective.
- Where the corrosion potential is greater (15 < Az <30) the CAC/DOL corrosion control layer and a host pipe of a standard concrete will be the most cost-effective.
- Where corrosion potential is severe (Az >30) an HDPE lining cast into the host pipe will probably be the most cost-effective solution.

It should be noted that the costs used in this exercise are estimates and that to make an accurate comparison for a project the actual pipe prices should be obtained from the suppliers.

The above example indicates that all sewer pipes and manholes should be manufactured using calcareous aggregates even if no corrosion is expected. The concrete used should contain not more than 25% insolubles when tested in hydrochloric acid (Details of the test method are given in SANS 677 (12)). In some parts of South Africa aggregates are only available with insolubility levels of 12% to 18%. If available, the lowest practical level should be specified.

The standard sacrificial layer thicknesses for OPC/DOL pipes used in South Africa are 13 mm and 19 mm for pipes up to 1050 mm and for larger diameters respectively. If the corrosion analysis indicates that these thicknesses are inadequate and a more costly material cannot be justified then a thicker sacrificial layer should be specified. To ensure that the hydraulic requirements will be met the minimum internal diameter and the sacrificial layer thickness should be specified. When the corrosion control layer and host pipe are made from different concretes an allowance should be made for the interface between the two concretes. Under these circumstances the standard design values for the corrosion control layers should be assumed to have a minimum of 10 and 15mm instead of the nominal values of 13 and 19mm used for sacrificial layers.

6.3. Corrosion Control Layer Thickness and Crack Widths

Where an increased sacrificial/corrosion control layer thickness is specified the allowable crack width should be increased in proportion to the increase in concrete cover over steel (12). The allowable crack width can be calculated using formula (50):

$$r = q\,(t-x) / (t-x-C_2) \tag{50}$$

Where r – allowable crack width for a sacrificial layer thickness of C_2 in mm

q – allowable crack width for a pipe with standard cover to steel in mm

t – total wall thickness of pipe in mm

x – distance from the outside surface of pipe to the neutral axis in mm

$$C_2 = C - C_1 \tag{51}$$

Where C – total concrete cover to inner steel reinforcement cage in mm

C_1 – standard specified concrete cover to inner steel reinforcement cage in mm

The neutral axis of the pipe can be assumed to be half the host pipe wall thickness. The relationship between these symbols is illustrated in Figure 11 below

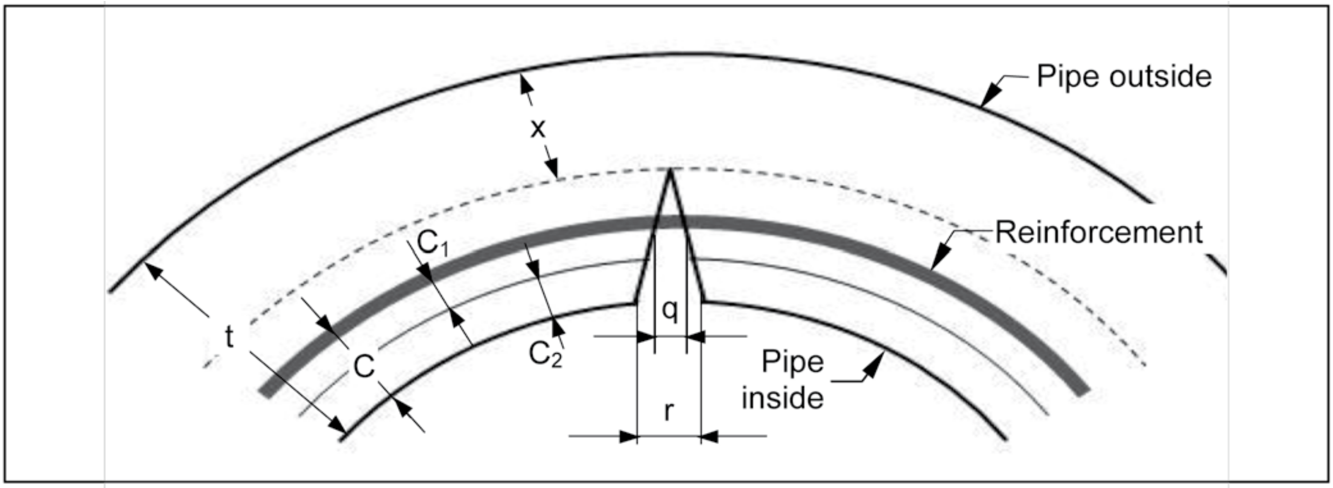


FIGURE 11: RELATIONSHIP BETWEEN CRACK WIDTH AND SACRIFICIAL LAYER

Example: If a 900 mm diameter concrete pipe with a wall of 93 mm has a sacrificial layer of 20 mm, what is the allowable crack width at proof load? Standard cover steel is 10 mm.

Neutral axis, x = 93/2 = 46.5 mm

$C = C_1 + C_2 = 10 + 20 = 30$ mm

r = 0.25(113 – 46.5)/(113 – 46.5 – 20) = 0.36 mm

There are two practical factors to consider when corrosion control layers are thicker than standard specified, layers: firstly, if the layer is thicker than one third of the pipe wall the reinforcement will be close to the wall centreline and will be ineffective in controlling cracks. Secondly, if the layer thickness is more than twice the standard cover over reinforcement the crack widths accepted when equation (50) is applied could be excessive and allow aggressive elements to enter the cracks thus moving the corrosion front closer to the reinforcement.

The ACPA Handbook (1, p6-17, p6-18) states that no pipes with cracks of up to 0.5 mm have been recorded when cover over reinforcement is 25 mm. As this is not substantiated by scientific study it is recommended that crack widths in sewer pipes be limited to 0.4 mm even if equation (50) indicates a larger value.

6.4. Remarks on CAC/DOL Linings

The use of CAC/DOL concrete as a corrosion control layer for concrete sewers has been based on information obtained from the testing of various cementitious materials in an aggressive real-life situation. This technique has been used in South Africa since 1997, and has proved to be both technically sound and cost-effective for sewers where conditions are aggressive and the cost of a cast in HDPE lining cannot be justified. This is because the material is about four times as effective in dealing with corrosion as the PC/DOL solution and at least ten times as effective as standard PC/SIL concrete. The same is probably applicable in any other developing country were material expenses are the major cost associated with the manufacture of sewers.

A further factor favouring the use of the CAC/DOL corrosion control layer is that when it is economically justified the total cover over reinforcement will be significantly less than that for PC/DOL, eliminating the need to make adjustments for excessive crack widths and make mould size adjustments.

6.5. Notes on HDPE Linings

The technique of lining concrete sewers with an inert lining was first used in the USA about 60 years ago. This material consists of a 1.5 mm thick uPVC sheet with longitudinal ‘T’ shaped strips anchoring it into the concrete. According to this material supplier there have been no reports of this lining becoming detached from precast pipes. The HDPE lining used in South Africa uses uniformly spaced anchors and is an improvement on the original system. It has been used for many of the large outfall sewers in this country since 1991.

This HDPE lining offers the sewer owner several additional advantages that are not always appreciated. Its surface is smoother than that of concrete, and thus a pipe with a smaller internal diameter can be used. Alternatively, the same diameter can be used at a flatter gradient. The long-term pullout strength of the lining anchors is able to cope with a ground water pressure in excess of 10m. If necessary the HDPE lining can be welded together at the joints by applying a bead of HDPE to make the lining continuous and create a water- and gas-tight joint. The abrasion resistance of the HDPE is actually greater than that of concrete.

Concrete pipe with a cast in HDPE lining has all the advantages of a strong rigid pipe that keeps its shape as well as those of a plastic pipe that is inert to acid attack. It is the best pipe for large diameter gravity pipelines in almost any condition.

6.6. Design and Detailing Considerations

Few of the many publications on sewer corrosion have been written in a South African context and as far as can be established, none have quantified the corrosion rates of non-PC concretes as described above. They do, however, address the issues of hydraulic design and detailing, and the following points should be considered:

- The longer the sewage stays in a sewer the greater the chance that it will turn septic and the rate of sulphide generation increases(?). Where practical, retention times should be kept to a minimum (less than an hour).
- Slow flows inhibit the absorption of oxygen into fresh sewage causing an increase in sulphide generation. In addition, slow flows could result in thicker slimes layers and silt build-up that in turn increase H_2S generation. Minimum flow velocities at the minimum discharges should be maintained.
- Moisture condensation on sewer and manhole surfaces provides a habitat for bacteria that produce H_2SO_4. However, reducing moisture condensation is not always possible.
- Junctions between sewers with different velocities obstruct slow flow causing long retention times. When sewers are joined upstream, gradients should be adjusted so that entry velocities and energy lines correspond.

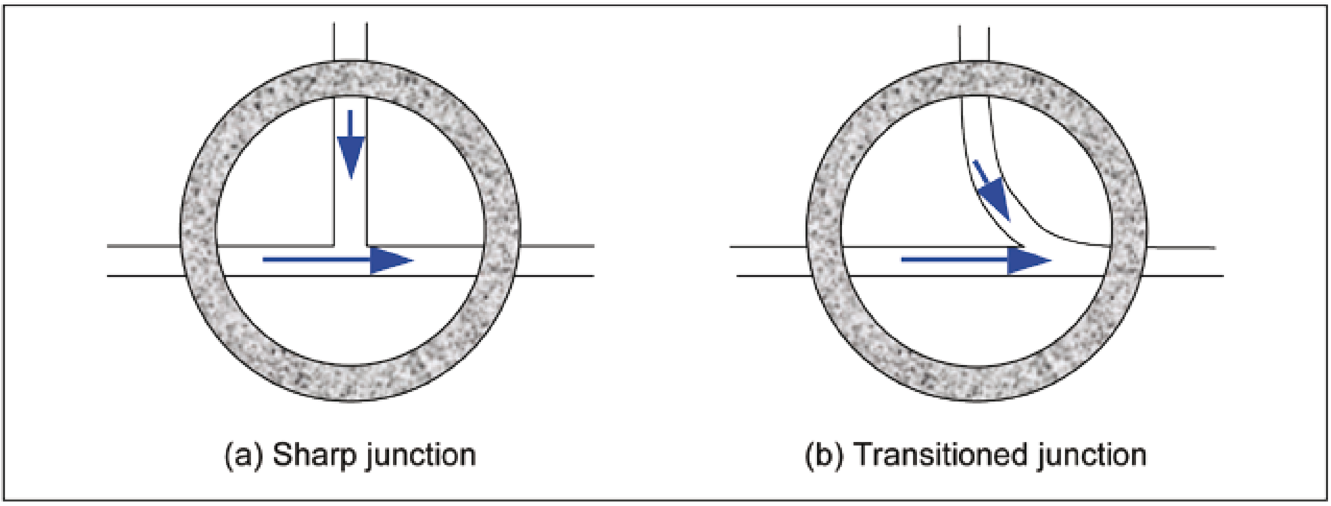


FIGURE 12: CONNECTION BETWEEN COLLECTOR AND LATERAL

- If a fast flowing lateral pipe discharges into a collector at the same invert level at a manhole with benching as shown in Figure 12(a), the flow in the collector may be obstructed causing long retention times upstream of the junction. The fast flows should enter above, and in the direction of the collector sewer, not at the same level and at as small an angle as possible, as shown in Figure 12(b).
- Manholes generally do not have a circular cross-section hence there are energy losses when a sewer passes though a manhole. They should be benched to minimize these losses. Where possible, manholes should be built over pipes that have holes cut into them so that the flow profile through the sewer remains constant.
- Most junctions are made at manholes. Energy losses and turbulence are associated with the release of H_2S and local corrosion. The inverts of such manholes should be carefully benched with smooth transitions to minimise energy losses and turbulence.
- Bends are also generally made at manholes. This results in energy losses and with the same consequences as at junctions. Whenever possible, sharp bends at manholes should be avoided by using pipes with deflected joints so that the sewer follows a curve. A manhole can then be placed on a straight section of sewer at the beginning or end of the curve.
- Supercritical flow should be avoided in sewers as obstructions can result in hydraulic jumps and risk surcharging manholes. If supercritical flow cannot be avoided, it may be necessary to increase the pipe diameter to ensure that if a hydraulic jump does occur, it will be contained within the sewer.

The rate of H_2S generation in rising mains and siphons is much greater than that in sewers flowing partly full: because the slimes layer extends around the full pipe circumference, none of the gas generated escapes and there is no oxygen enrichment of the sewage. Severe corrosion can occur in downstream sections of sewers especially when sewage retention time exceeds an hour. When the sewage discharges into the gravity-operated section of sewer the accumulated H_2S is liberated and can cause severe local corrosion. Procedures for minimising retention times and the resultant corrosion include:

- using the smallest practical pipe diameter for the full flowing section of sewer
- making the section as short as possible
- operating pumps frequently, particularly in the early years of the system where low flows could result in the sewage upstream of the full flowing section becoming septic.

Sewage with a high BOD usually results in a high sulphide content, which could result in the corrosion of structures at the purification works. Measures that can be taken to reduce this include:

- if the BOD is very high (greater than 1 000 mg/l) pre-treat the sewage
- lay the feed line to the dosing tank below the hydraulic gradient to exclude oxygen
- in special cases add hydrated lime to increase the sewage pH or ventilate the outfall using a forced draught.

Careful hydraulic design and attention to detail reduces sewer corrosion. However, they cannot eliminate the problems that could arise if the corrosion potential is severe and has not been identified by doing the necessary corrosion analysis. The above considerations should be used in combination with an application of the LFM when designing and detailing sewers, not as a substitute for such an analysis.

7. FIELD TESTING

7.1. General Comments

Pipelines consist of pipes and joints. Concrete pipes used for sewers and low-pressure pipelines are factory tested hydrostatically and under load before delivery to site to ensure that they will meet structural requirements. As pipes are jointed on site they should be tested on site to ensure that the pipeline jointing meets the operating requirements.

Concrete heals autogenously, and therefore hairline cracks or damp spots should not be cause for rejection as this type of leakage will stop within days of the pipe surface being exposed to a moist environment. However, leaking joints need to be repaired because infiltration and exfiltration cause progressive problems for the sewer such as the risk of relative settlement, cavities in the surrounding soil and the development of sink holes.

7.2. Types of Pipe Joint

The function of joints is to provide flexibility and sealing for pipelines. Joints are designed to cope with movements that occur as a result of secondary forces within the soil mass. They are designed to seal as well as tolerate movements in three directions, longitudinal movement, deflection or radial movement, and relative settlement or displacement relative to adjacent pipes. In addition, joints must accommodate tolerances of concrete surfaces, seal dimensions and laying procedures.

Figure 13 shows four types of pipe joints: butt (or plain-ended), interlocking (or Ogee), spigot and socket, and in-the-wall joints. These are used for different purposes, depending on the amount of movement anticipated and the importance of the pipeline being sealed.

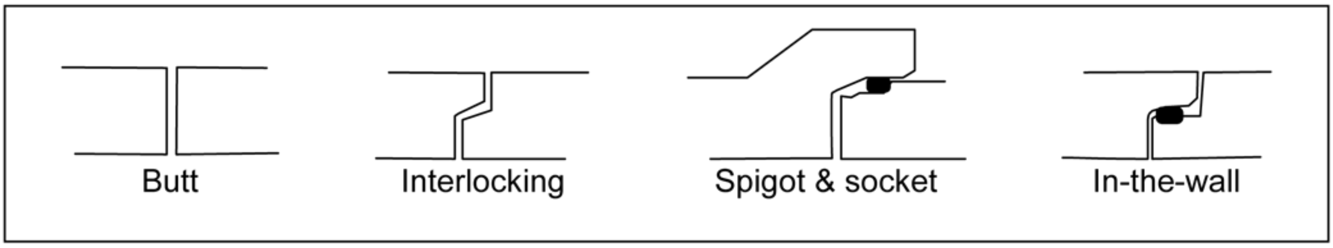


FIGURE 13: SECTION THROUGH DIFFERENT CONCRETE PIPE JOINT TYPES

Butt-ended and interlocking pipe joints do not prevent infiltration and exfiltration of water, so are only used for storm water drainage and culvert pipes. Butt-ended pipes are seldom used as they do not have any means of self-centring. Joints for sewers or pressure pipelines should include a seal in the form of either a rolling or confined ring. The rubber ring enables the joint to be deflected as shown in Figure 14 so that pipes can be laid around curves and still remain watertight.

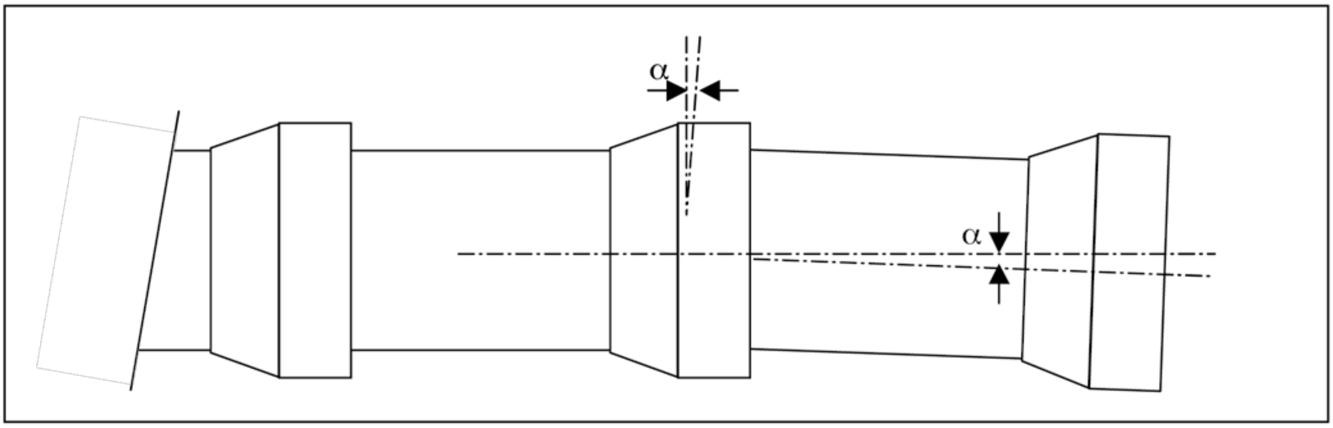


FIGURE 14: ANGULAR DEFLECTION OF SPIGOT AND SOCKET JOINT PIPES

Spigot and socket joints are most commonly used for sewers less than 1500mm in diameter. In the case of larger pipes the wall is so thick that a rubber ring can be accommodated within the wall thickness. This is called an “in-the-wall joint”. In these cases joint nibs or grooves on the jointing surfaces confine the seal. Because the seal remains in a fixed position and the socket slides over it, this type of joint is sometimes called a “confined” or “sliding rubber ring joint”. Particular attention should be paid to lubricating joints. The amount of movement that can be tolerated at a joint will depend on the pipe size and the manufacturer’s details.

The advantage of the in-the-wall joint is that the outside diameter of the pipe remains constant, making the pipe ideal for jacking. Jacking pipes of 900mm diameters and larger use this joint type. However, this type of joint is seldom used for pipes of less than 1500mm diameter for sewers. The joint is designed to cope with the same conditions as the spigot and socket joint, but as it is shorter, the amount of movement that it can tolerate is less. They are designed to cope with a deflection of 0.5 degrees so that curves can be negotiated. For details of how these pipes should be jointed and the deflections that can be tolerated, refer to the pipe supplier.

The radius of the curve that can be negotiated is dependent on the permitted angular deflection for each pipe size. Typical deflections and curve radii for pipes with spigot and socket joints are presented in Table 17. The details for a specific project should be discussed with the pipe manufacturer concerned.

TABLE 17: ANGULAR DEFLECTIONS AND CURVE RADII

Nominal Pipe Diameter - mm	Permissible Degrees	Minimum Radius - m	Notes
300 - 375	2.00	70	The radius of curve that can be negotiated is directly proportional to the pipe’s effective length. Values in this table were calculated using an effective pipe length of 2.44 m.
450 - 600	1.50	93	
675 - 900	1.00	140	
1 050 - 1 200	0.75	186	
1 350 – 1 800	0.50	280	

If a different length is used the radius from the table should be corrected by the ratio between the two lengths. Where sharper curves are required, pipes with deflected spigots or sockets, or radius pipes can be produced. Details should be discussed with the manufacturer.

7.3. Water Testing on Site

Apart from visual inspection, the only field-testing needed on a concrete pipeline is for water-tightness. The water test is carried out as follows (13, p56):

- Close the section of pipeline to be tested with bulkheads or plugs. As these will be subject to considerable forces they should be designed and installed to ensure that they can withstand these forces with an adequate safety factor.
- Open air valves and slowly fill the section with water ensuring that all the air escapes.
- Keep test section under a slight pressure for 3 to 5 days to allow pipes to absorb water (if pipes were exposed for more than a month additional time may be needed for this).
- During this period check the sealed ends and joints for leaks and the rate at which water has to be added to maintain the pressure.
- When the rate of adding water stabilises increase the pressure to the required value.

SABS 1200-LD (14, p9) prescribes that sewers should be tested with a water head of not less than 1.2m and not more than 6.0m and that the loss allowance is not more than 6 litres/100mm of diameter/100m/hour.

The full-scale water testing of large diameter sewers is difficult and costly. When available, joint testing equipment that applies water pressure to one joint at a time can be used. This equipment must be used with care, as the joint that has already been factory tested: hence the pressures used are not the same as those for which the pipeline is rated.

When a sewer can accommodate human entry (≥1200 mm in diameter) and is below the water table, it can be physically inspected to check for leaks.

7.4. Air Testing on Site

Air testing of concrete sewers is an effective way of identifying isolated leaking sections such as poor joints or damaged pipes. As air and water have different properties this test is not an indicator of the water-tightness of the pipe wall. This test can therefore be used as an acceptance test but not as justification for rejection. If there is a dispute the final acceptance or rejection of a sewer should be based on a water test.

This test is conducted in a similar way to the water test. However, as its purpose is to find isolated problems the air pressure inside the section being tested is only just above atmospheric pressure. The procedure followed follows (13, p57):

- Seal the ends of the section to be tested with bulkheads or plugs, making sure that the safety factor of blow out to test pressure is at least 2.
- One of the bulkheads is fitted with connections to an air source, a pressure release valve and a pressure gauge or monometer.
- Air is added to the test section to increase the internal pressure to a prescribed amount above atmospheric pressure. This must allow sufficient time for this to stabilise, as there may be differences between the air and pipe wall temperatures.
- Once the air pressure within the test section has stabilised the air supply is stopped and the time in seconds that it takes for a given pressure drop is measured. The rate of air loss is then calculated.

The sewer is then inspected to determine whether joints or damaged sections are leaking. These leaks can usually be identified by the sound of escaping air. If no localised leaks are identified and the rate of pressure drop is unacceptable the exposed sewer is sprayed with soapy water to locate problem areas. If bubbles form on sections of exposed pipeline this is probably because the pipes have dried out during prolonged periods of exposure (in excess of 6 weeks). When these pipes are exposed to the moist sewer atmosphere the concrete will absorb moisture and the microstructure seal.

Section 7 of SABS 1200-LD (14, p9) prescribes procedures and pressures that should be used for the air testing of sewers as follows:

- apply an initial pressure of 3.75kPa (375mm of water)
- once the pressure stabilises, reduce it to 2.5kPa (250mm of water)
- switch off the air source and measure how long it takes for the pressure to drop to 1.25kPa (125mm of water)
- the minimum acceptable time for this drop to take place is 2 minutes/100mm diameter.

Leaking joints or damaged sections of pipe must be repaired using means that are approved by the project engineer. Whenever possible, defects should be repaired with the pipes in place. Only when pipes have been incorrectly installed or there has been damage due to soil movement, should pipe replacement be considered. If this is necessary it must be done from manhole to manhole so that a whole reach is redone at once and the possibility of relative settlement between sections of sewer eliminated.

Handling damage

8. CONDITION OF EXISTING SEWERS

8.1. Aging of Sewers

Sewers can deteriorate with time and fail to serve their function and should be inspected periodically to assess their condition. When necessary, they should be rehabilitated to ensure that their operation is sustainable.

Until recently infrastructure design was based on a 40 year life on the assumption that at the end of this period the infrastructure would be replaced or rehabilitated. Many sewers in South Africa's older cities are older than 40 years and have not been rehabilitated or replaced. Thus it is not surprising that there are periodic failures. The combination of the failure to replace malfunctioning sewers, the growing urban population and increasing industrialisation means that the best way of meeting demands is supplementing the construction of new sewers by the rehabilitation of the old ones.

The total energy line for a sewer is almost always well below the ground surface, and therefore when leaks occur they are not detected. This means that the effluent and ground water may be exchanged. During high flows the effluent may flow out of the sewer and cause groundwater pollution and during low flows groundwater may enter the sewer and overload it. Surrounding material is then transported into the sewer, resulting in increased sediment in the effluent and the formation of cavities around the sewer due to the loss of bedding support.

Sewer leaks may go unnoticed for years – until cavities that have formed around defects surface as sinkholes. This is extremely dangerous for the public and may cause damage to property. It also indicates that progressive failure of the sewer has probably started and there may be no alternative but to replace it. When the sewer has been correctly installed there should still be a sound bedding foundation and an acceptable gradient, and on-line trenchless replacement techniques can be used.

8.2. Pipe Materials

The choice of pipe material is the most significant contributor to the life of any pipeline, yet it probably receives less attention than any other factor. For a pipeline to remain durable the pipe material used must be able to handle aggressive chemical elements in the effluent, sewer atmosphere or in the ground water and soil surrounding the pipeline as well as the physical properties of the effluent such as the abrasive effects of materials carried and temperature. Under certain conditions the combined impact of chemical and physical factors can accelerate the deterioration of the pipe material. The material choice must be based on an analysis of the various components in and around the pipeline. The term "pipe soil system" is also applicable to durability considerations.

All these factors need to be considered when selecting pipe materials. Although reinforced concrete is ideal material for outfall sewers it deteriorates under certain conditions, which should be identified during the planning and design stage of a project so that it can be modified or protected. Irrespective of the pipe material and the site conditions, it is essential that the pipeline is correctly installed.

8.3. Condition Assessment

Trenchless rehabilitation techniques are frequently used in urban areas where old sewers are malfunctioning and where the conditions below the surface and in the sewer are not known. Before taking action it is essential to establish these conditions and identify the actual problems, their location, severity and causes.

Condition assessment involves gathering and processing the visual data from inside a pipeline using a CCTV camera and categorising faults. More broadly, it involves gathering data from as many sources as possible, including:

- hydraulic performance (size)
- water tightness (joints)
- all aspects of the pipe soil system (structure)
- effluent composition (durability).

This information is used to determine the pipeline performance as well as the condition of the pipes, the joints and the soil around them both qualitatively and quantitatively.

8.4. CCTV Camera Inspections

The defects that may occur inside a pipeline can be broadly grouped into four categories that correspond to the performance criteria that are compromised. Sufficient information must be gathered to identify the problems, categorise them and describe them in terms of their location, severity and causes.

The first two items can be obtained from a CCTV inspection of the sewer's internal bore. The camera can be used in non-man entry pipes; the inspection is rapid and is done at low flow. If necessary the pipeline can be plugged to allow the inspection. Data from the whole section are captured and can be digitised. This means that after data processing a visual inspection can be done to identify the critical sections of a sewer.

Currently the data captured by this equipment only provide a visual representation. However, equipment is being developed so that two additional sets of data can be gathered from within the pipeline namely a continuous record of the pipeline's internal dimensions by using laser profiling and the presence of voids and changes in material outside the pipeline by using in-pipe-radar.

This combination of inspection techniques will be useful for determining the condition of operating pipelines and the installation quality of new ones.

8.5. Physical Measurement of Corrosion

The CCTV inspections and laser profiling provide valuable information using inexpensive means, but still do not provide a complete picture and it is, therefore, necessary to do periodic physical measurements. This involves the identification of sections of sewers where severe corrosion is anticipated which can be easily exposed from the surface so that the investigator can do a live inspection and take material samples. Such a window is illustrated in Figure 15.

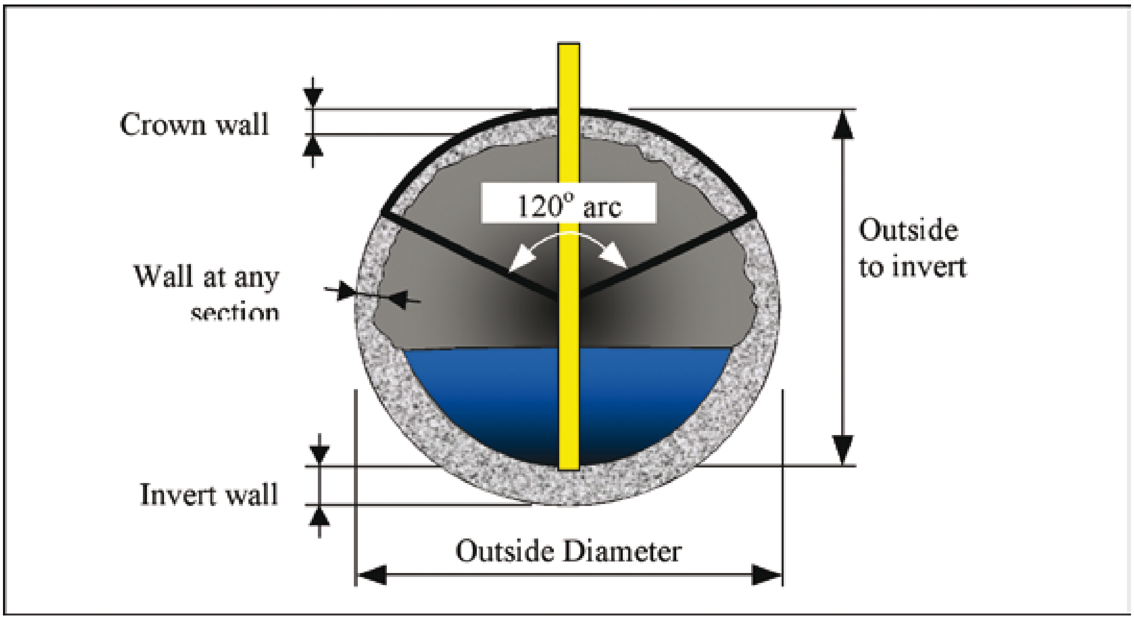


FIGURE 15: ILLUSTRATION OF AN INSPECTION WINDOW

This type of inspection can be carried out concurrently with a CCTV inspection and the respective results calibrated to provide a comprehensive impression of the sewer condition.

Although the LFM and the additional factors described above endeavour to model actual conditions, the data obtained from these models should be confirmed empirically. By comparing the corrosion rates from the theoretical analysis with the physical measurements of corrosion in an existing sewer and the output from a CCTV camera survey the model can be verified or adjusted.

8.6. Comparing Life Factor Model, CCTV and Physical Inspection Data

A diagram illustrating the interaction between the various sources of data obtained from a sewer is presented in Figure 16. If the corrosion rates determined from the theoretical analysis and the physical inspection correlate and the CCTV survey indicates that the conditions are consistent along the sewer the response to the sewer condition will be straightforward. The rehabilitation procedure depends on technical issues related to the severity and cause of the problems, the size of the project and availability of technology.

If the corrosion rates used in this comparison do not agree, the reasons for the disparity should be investigated. In most such cases effluent being conveyed is domestic and the measured corrosion is greater than that predicted. This indicates that the dissolved sulphide content is higher than that calculated by an analysis of the sewer-section hydraulics and that it is necessary to extend the analysis to include the upper reaches of the sewer. This usually indicates that an upstream rising main or siphon with a long retention time is the source of the problem..

When retention times in a pipe flowing full (as in a rising main or siphon) exceed an hour the rate of sulphide generation could be high enough to cause a downstream corrosion problem. Temperatures in excess of 20°C and pH values lower than 7 also increase the corrosion potential.

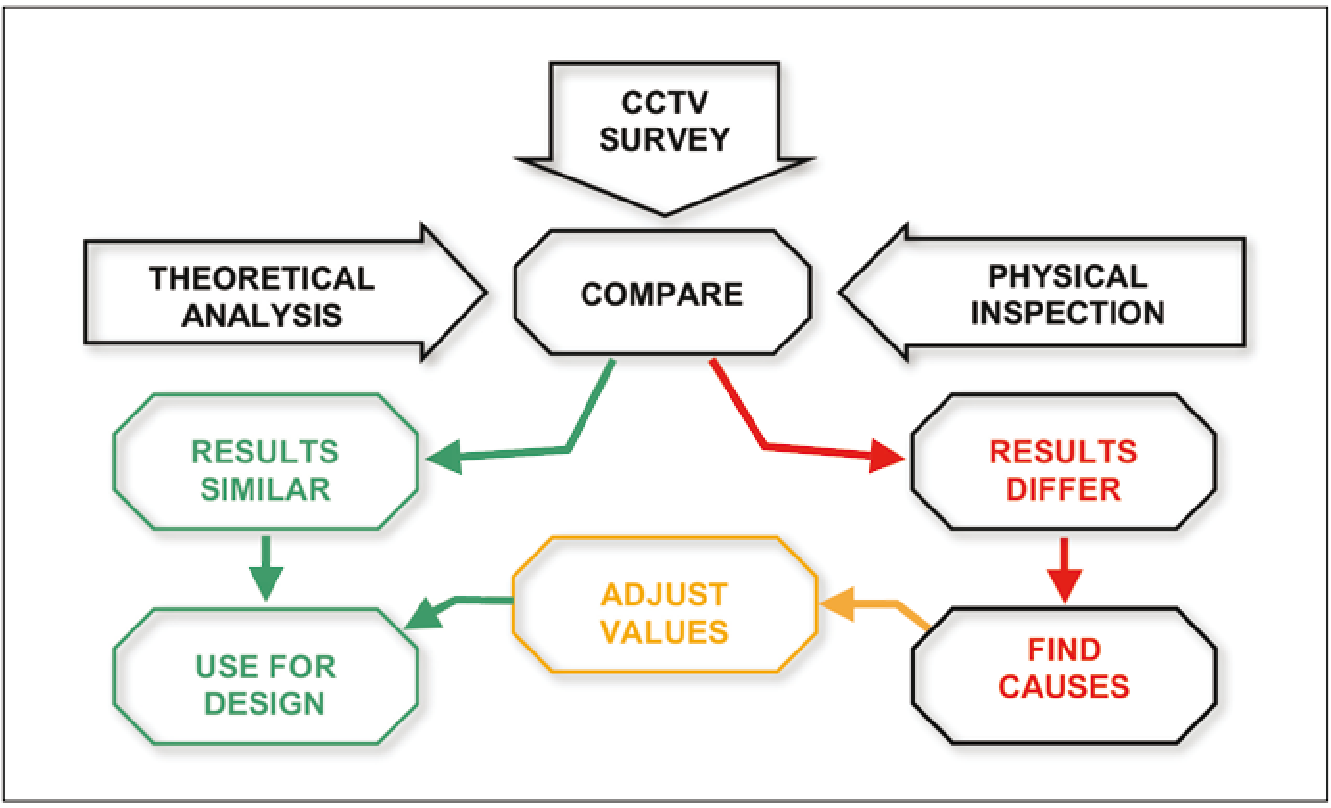


FIGURE 16: COMPLEMENTING LFM WITH CONDITION ASSESSMENT

When the effluent is industrial or has a high industrial content the pH could be so low that sulphide-producing bacteria in the slimes layer and below effluent level are killed off. Under these circumstances the corrosion above the effluent level may be less than expected or nonexistent, but the low pH content of the effluent (especially if it is consistently less than 5.5) could result in the corrosion of the sewer invert. This problem may not be identified via a CCTV inspection as it is below the effluent surface, although the physical inspection should identify it.

The procedure described above is applicable to existing sewers. Most new sewers are constructed in areas that already have sewers installed and therefore the conditions and corrosion rates can be compared with the theoretical model and design alterations made.

9. CONCLUDING REMARKS

As sewers are usually buried gravity pipelines that flow partly full at a depth below other services, their deterioration goes unnoticed until there is a serious failure and at times a disaster. Although concrete pipe is the ideal material for large diameter outfall sewers, under certain conditions they may be subject to attack from biogenically generated sulphuric acid above the water level. In South Africa there have been very few such events since the introduction of the dolomitic aggregate sacrificial layer in the early 1950s.

When new concrete sewers are designed, this corrosion can be predicted by using the LFM. By applying material and turbulence factors to the result, the pipe material or lining can be selected. If possible, the corrosion rates from this analysis should be compared to the corrosion rates obtained from the physical measurement of losses on existing sewers servicing the same catchment area under similar conditions. Differences between these values should be investigated and the causes identified. Should these be applicable to a new sewer it should be designed accordingly.

As the operating conditions in a sewer change, for different reasons, the pipe material of an existing sewer may become unsuitable resulting in deterioration and failure. It is therefore essential that existing sewers are periodically inspected and their condition assessed so that maintenance and rehabilitation can be implemented.

The purpose of this publication has been to present the latest developments in the field of pipe material choice to control the corrosion problem in outfall sewers and to emphasize the importance of assessing the operating conditions during the investigative and design stages of projects involving both the design of new sewers and the rehabilitation of old ones

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