CONCRETE PIPE AND PORTAL CULVERT HANDBOOK

CONCRETE MANUFACTURERS ASSOCIATION
Quality cast in concrete

PIPES, INFRASTRUCTURAL PRODUCTS AND ENGINEERING SOLUTIONS DIVISION

PREFACE TO THE SIXTH EDITION

Concrete pipes and portal culverts are the most frequently used and accepted products for stormwater drainage, culverts, outfall sewers and many other applications. To meet these needs South Africa's concrete pipe industry has grown tremendously over the past eighty years.

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

This handbook is intended to cover all aspects of concrete pipe and portal culvert selection, specification, and testing. As a handbook it does not attempt to replace textbooks or codes, but rather to complement them by providing the information needed for quick site decisions and guidance for designers to ensure that all aspects of product use are considered. A companion publication 'The Concrete Pipe and Portal Culvert Installation Manual' deals with product installation.

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

The Concrete Pipe, Infrastructural Products and Engineering Solutions (PIPES) Division of the Concrete Manufacturers Association has had this handbook prepared for the guidance of specifying bodies, consultants and contracting organisations using concrete pipes and portal culverts manufactured in accordance with the relevant SANS (SABS) standards. The Division expresses appreciation to A.R. Dutton & Partners for the preparation of the original Concrete Pipe Handbook to which has now been revised and amended several times to include developments.

Although every effort has been made to ensure the accuracy of the information given, it is not possible for the CMA to accept responsibility for the work prepared on the basis of this Manual

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TABLE OF CONTENTS

1. INTRODUCTION 2
1.1. OBJECTIVE 2
1.2. SCOPE 2
2. PRODUCT CLASSIFICATION 2
2.1. STANDARDS 2
2.2. CONCRETE PIPES 3
2.3. PORTAL CULVERTS 5
2.4. MANHOLES 7
3. HYDRAULICS 7
3.1. CONDUIT CLASSIFICATION 7
3.2. HYDRAULIC LENGTH 8
3.3. PRESSURE PIPELINES 9
3.4. SEWERS AND STORMWATER OUTFALLS 10
3.5. HYDRAULICS OF STORMWATER CULVERTS 11
3.6. POROUS PIPES 18
4. LOADS ON BURIED PIPELINES 20
4.1. INTRODUCTION 20
4.2. EARTH LOADS 20
4.3. TRAFFIC LOADING 28
5. CONCRETE PIPE STRENGTHS 30
5.1. EXTERNAL LOADS 30
5.2. INTERNAL PRESSURE 31
5.3. SAFETY FACTORS 32
5.4. SELECTION OF THE CONCRETE PIPE CLASS 32
6. BEDDING 35
6.1. GENERAL 35
6.2. TRENCH AND NEGATIVE PROJECTION INSTALLATIONS 36
6.3. POSITIVE PROJECTION INSTALLATIONS 39
6.4. SOILCRETE BEDDING 41
6.5. JACKING CONDITIONS 42
7. PIPE JOINTING 42
7.1. JOINT TYPES 42
7.2. BUTT AND INTERLOCKING JOINT PIPES 42
7.3. SPIGOT AND SOCKET JOINTS 43
7.4. IN-THE-WALL JOINTS 44
8. FLOATATION 44
8.1. GENERAL 44
8.2. FLOATATION BEFORE BACKFILLING 45
8.3. FLOATATION AFTER BACKFILLING 45
9. SEWER CORROSION 45
9.1. CORROSION MECHANISM 45
9.2. CORROSION PREDICTION AND CONTROL 47
9.3. DEVELOPMENTS IN SOUTH AFRICA 48
9.4. DESIGN AND DETAIL CONSIDERATIONS 50
9.5. PIPE MATERIAL CHOICE FOR SEWERS 51
9.6. SACRIFICIAL THICKNESS AND ALLOWABLE CRACK WIDTHS 54
10. PORTAL CULVERTS 55
10.1. GENERAL 55
10.2. DETERMINING PORTAL CULVERT STRENGTHS 55
10.3. PORTAL BASE SLABS 57
11. CONCLUDING COMMENTS 58
BIBLIOGRAPHY 59

1. INTRODUCTION

1.1. OBJECTIVE

The purpose of this handbook is to give the users, designers, specifiers and installers of precast concrete pipe and portal culverts the basic guidelines for the correct use, selection and specification of these products. A companion publication “The Concrete Pipe and Portal Culvert Installation Manual” gives details of how these products should be installed.

1.2. SCOPE

The content of this handbook covers the pre-construction activities associated with precast concrete pipe and portal culverts, namely those undertaken by the designer of the project. Descriptions are given of the basic theory needed for determining:

- product size
- product strength
- product durability
- special product features

The basic formulae, diagrams and tables support this. This information is adequate for most product applications. However, the theory given is by no means rigorous. The reader is advised to consult the relevant textbooks or codes, should a detailed analysis be required. A list of useful publications is given at the end of this handbook.

2. PRODUCT CLASSIFICATION

2.1. STANDARDS

There are three groups of standards which are applicable to precast concrete pipe and portal culverts, namely:

- Codes of practice detailing how product size, strength and durability are selected.
- Product standards that prescribe what product requirements have to be met.
- Construction standards that prescribe how products should be installed.

The South African Bureau of Standards (SABS) has been restructured. The division dealing with the production of standards is Standards South Africa (StanSA). All the previously designated SABS standards are to be renamed as South African National Standards (SANS) and will retain their numbers. This document uses the latter.

The division dealing with the issuing of manufacturing permits and the auditing of production facilities is Global Conformity Services (GCS). The products covered by this publication comply with the requirements of relevant (SANS) document. These are performance specifications that detail the properties of the finished products needed to ensure that they are suitable for their required application. All these standards have the same basic layout, namely:

- Scope
- Normative references
- Definitions
- Materials used
- Requirements to be met
- Sampling and compliance
- Inspection and test methods
- Marking
- Normative and informative annexures.

Most factories operated by the PIPES Division member companies have approved quality management systems to ensure that products comply with the relevant SANS specifications. In addition to this GCS, does frequent audits to check that standards are being maintained. These standards are periodically reviewed to ensure that marketplace requirements are met.

2.2. CONCRETE PIPES

2.2.1. Standards

Currently there are two South African national standards applicable to concrete pipe:

SANS 676 - Reinforced concrete pressure pipes

SANS 677 - Concrete non-pressure pipes

The code of practice for the selection of pipe strength is:

SANS 10102 - Part 1: Selection of pipes for buried pipelines: General provisions

- Part 2: Selection of pipes for buried pipelines: Rigid pipes

There are no standards for determining the size or durability of concrete pipe. If the reader requires more detail than given in this publication, reference should be made to the appropriate literature, some of which is detailed at the end of this publication.

The standards for the installation of concrete pipe are included as sections in SANS 1200 Standardized specification for civil engineering construction. These sections are:

SANS 1200 DB - Earthworks (pipe trenches)

SANS 1200 L - Medium pressure pipe lines

SANS 1200 LB - Bedding (pipes)

SANS 1200 LD - Sewers

SANS 1200 LE – Storm water drainage

SANS 1200 LG - Pipe jacking

2.2.2. Pipe classes

Non-pressure pipe

Pipes are classified in terms of their crushing strength when subjected to a vertical knife-edge test-load. The two alternative crushing load test configurations are shown in Figure 1 (a) & (b).

(a) Two edge bearing test (b) Three edge bearing test

Figure 1: Crushing Load Test Configurations for Concrete Pipe

The three edge-bearing test is preferred as the pipe is firmly held in place by the bottom two bearers before and during the test. With the two-edge bearing test there is the danger that the pipe could slip out of the testing apparatus or might not be perfectly square when tested.

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

The ultimate load is defined as the maximum line load that the pipe will support in a two or three edge-bearing test and shall be at least 1.25 times the proof load.

The standard crushing load strength designation is the D-load (diameter load). This is the proof load in kilonewtons per metre of pipe length, per metre of nominal pipe diameter. The standard D-load classes with their proof and ultimate loads are given in Table 1.

TABLE 1: STANDARD D-LOAD CLASSIFICATION FOR CONCRETE PIPES

Pipe Class D-Load	Proof load kN/m	Ultimate load-kN/m	Example
25D	25xD	31.25xD	For a 1050 mm diameter 75D pipe
50D	50xD	62.50xD	proof load = 1.05 x 75 = 78.75 kN/m
75D	75xD	93.75xD	ultimate load = 1.05 x 93.75 = 98.44
100D	100xD	125.00xD	kN/m

Pipes made in accordance to 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 small sample (±2%) of pipes is subjected to the crushing strength test to prove that they meet the strength required. SI Pipes, on the other hand, are used in applications where there could be internal pressure under certain conditions (as when blockages occur). To ensure that the pipes will meet this possible condition and ensure that the joints are watertight, a small sample of pipes is hydrostatically tested to a pressure of 140 kilopascals in addition to the crushing strength test.

Table 2 gives proof loads of the preferred nominal diameters given in SANS 676 and 677.

TABLE 2: PREFERRED CONCRETE PIPE DIAMETERS AND PROOF LOADS

Nominal Pipe Diameter-mm	D Loads in Kilonewtons/m			
	25D	50D	75D	100D
300	-	15.0	22.5	30.0
375	-	18.8	28.1	37.5
450	-	22.5	33.8	45.0
525	13.1	26.3	39.4	52.5
600	15.0	30.0	45.0	60.0
675	16.9	33.8	50.6	67.5
750	18.3	37.5	56.3	75.0
825	20.6	41.3	62.0	82.5
900	22.5	45.0	67.5	90.0
1 050	26.3	52.5	78.8	105.0
1 200	30.0	60.0	90.0	120.0
1 350	33.8	67.5	101.3	135.0
1 500	37.5	75.0	112.5	150.0
1 800	45.0	90.0	135.0	180.0

Notes

1) Pipes with diameters smaller than 300 mm, or larger than 1 800 mm are made at some factories.

2) Strengths greater than 100D can be produced to order.

3) Most pipes are made in moulds with fixed outside diameters. The designer should check minimum the internal diameters to ensure that requirements are met.

Pressure pipe

Pressure pipes are classified in terms of their hydraulic strength when subject to an internal pressure test under factory conditions.

Hydraulic strength is defined as the internal pressure in bar that the pipe can withstand for at least 2 minutes without showing any sign of leakage. The standard hydraulic strength designation is the test (T) pressure. The SANS 676 pressure classes are given in Table 3.

TABLE 3: STANDARD PRESSURE CLASSES FOR PIPE

Pipe class	Test pressure		Notes
	Bars	Kilopascals	
T2	2	200	Most concrete pipes used for pressure applications are special purpose pipes which are subjected to both internal pressure and external load should be specified in terms of both the pressure class and D-Load class.
T4	4	400	
T6	6	600	
T8	8	800	
T10	10	1 000	

Special-purpose pipe

Many pressure pipelines are installed at a nominal fill and where they are not subject to traffic loads. Under these circumstances the hydraulic strength designation, given in Table 3, is adequate.

However, when a pipeline is subject to the simultaneous application of internal pressure and external load, the pipes will need to sustain a higher hydraulic pressure and crushing strength than when service loads are applied separately.

Under these conditions the pipes will be classified as special-purpose pipes and the required hydraulic test pressure and crushing strength to meet the required installed conditions will have to be calculated. These pipes must be specified in terms of both their D-load and T-pressure values.

2.3. PORTAL CULVERTS

2.3.1. Standards

The standard for precast concrete culverts is SANS 986, precast reinforced concrete culverts.

There is no National code of practice for the selection of portal culvert size or strength. However, the biggest single group of users, the national and provincial road authorities, require that portal culverts under their roads meet the structural requirements of TMH7, the Code of Practice for the Design of Highway Bridges and Culverts in South Africa. The local authorities generally adhere to the requirements of this code. This document also gives guidelines for product durability.

If more detail than provided in this document is required, reference should be made to the appropriate literature, some of which is listed at the end of this publication.

The standards for the installation of precast portal culverts are included in sections 1200DB and 1200LE of the SANS 1200 series.

2.3.2. Portal Culvert Classes

Precast portal culverts are classified in terms of their crushing strength, when subjected to a combination of loading cases involving vertical and horizontal knife-edge test-loads under factory conditions. The proof and ultimate loads are defined in the same way as for

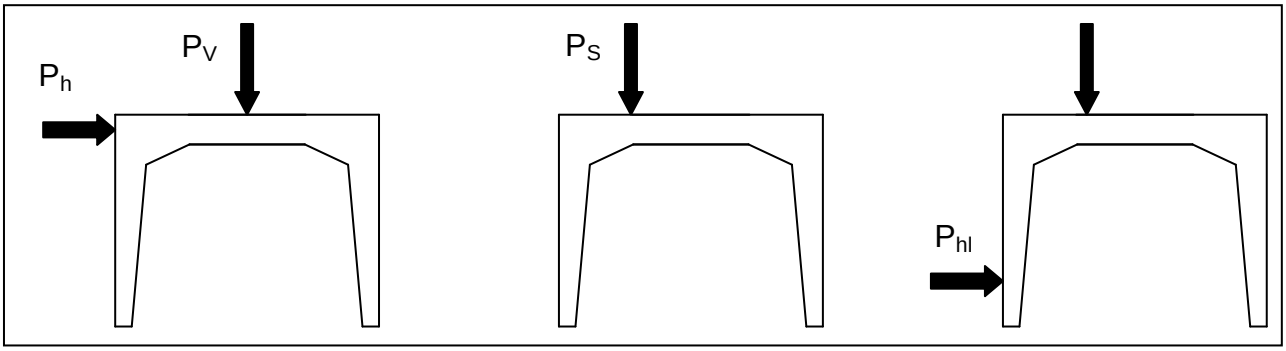
pipes with the ultimate loads being 1.25 times the proof loads for the particular loading configurations.

The standard crushing strength designation used is the S-load. (Span-crushing load) This is the vertical component of the proof load in Kilonewtons that a 1metre length of culvert will withstand, divided by the nominal span of the portal culvert in metres.

There are three different loading configurations that are applied to precast portal culverts to model the installed conditions, namely:

- Deck bending moment and sway
- Deck shear
- Inner leg bending moment and shear

These configurations are shown respectively in Figure 2(a), (b) and (c) below and the standard S-load classes with their proof load requirements are given in Table 4.



(a) Deck bending moment and sway (b) Deck shear (c) Inner leg bending moment &shear

Figure 2: Load Test Configurations for Precast Portal Culverts

TABLE 4: STANDARD S-LOAD CLASSIFICATION FOR PORTAL CULVERTS

Culvert class S-Load	Proof loads - kN/m of length		Leg Proof loads - kN/m of length	
	Vertical	Horizontal	Height > S/2	Height = S
75S	75 x S	30	0.4 x 75 x S	0.60 x 75 x S
100S	100 x S	30	0.3 x 100 x S	0.50 x 100 x S
125S	125 x S	30	0.2 x 125 x S	0.45 x 125 x S
150S	150 x S	30	0.2 x 150 x S	0.43 x 150 x S
175S	175 x S	30	0.2 x 175 x S	0.40 x 175 x S
200S	200 x S	30	0.2 x 200 x S	0.40 x 200 x S
Note: S is the nominal span in metres.				

Table 5 gives the vertical and horizontal proof loads obtained by applying the classification in Table 4 to the preferred portal culvert dimensions given in SANS 986. A table similar to Table 5 can be obtained by application of the values in Table 4 to obtain the inner leg bending moments and shears. It should be noted that there will be two different values of the horizontal load for each culvert span and class, i.e. when 0.5 < H/S < 1.0 and H/S = 1.0. When H/S < 0.5 no horizontal leg load is required.

TABLE 5: PREFERRED PORTAL CULVERT DIMENSIONS AND PROOF LOADS

Culvert span mm	Vertical proof loads in kN/m of length					Horizontal proof load all classes kN/m
	Culvert class					
	75S	100S	150S	175S	200S	
450	-	-	-	-	90.0	
600	-	-	-	-	120.0	
750	-	-	-	131.3	-	
900	-	-	-	157.5	-	
1200	-	-	180.0	-	-	
1500	-	150.0	-	-	-	30
1800	135.0	-	-	-	-	
2100	157.5	-	-	-	-	
2400	180.0	-	-	-	-	
3000	225.0	-	-	-	-	
3600	270.0	-	-	-	-	

2.4. MANHOLES

2.4.1. Standards

The standard for precast concrete manhole sections, slabs, lids and frames is SANS 1294. The standard manhole dimensions are hard metric, namely:

- 750 mm diameter - used as shaft sections
- 1 000 mm diameter - normally used as chamber sections
- 1 250 mm diameter - used as chamber sections
- 1 500 mm diameter - used as chamber sections
- 1 750 mm diameter - used as chamber sections

These sections are available in lengths of 250 mm, 500 mm, 750 mm and 1 000 mm.

In the past manholes were produced in soft metric dimensions. Hence when components have to be replaced it is essential that actual details and dimensions be checked before ordering replacements as old sizes are no longer available and it may be necessary to replace the whole manhole.

SANS 1294 has been revised. A detailed section on manholes will be added to this publication in a future revision.

3. HYDRAULICS

3.1. CONDUIT CLASSIFICATION

Conduits conveying fluids are classified by various parameters, namely, whether:

- They flow as open channels or closed conduits
- The flow is uniform, in which case the flow depth, velocity and discharge along the whole length of the conduits are constant. If not uniform, the flow is varied
- The flow is steady in which case the flow past a given point has a constant depth, velocity and discharge. If not steady, the flow is unsteady.

A pipeline conveying potable water or other fluids generally flows full and operates under pressure and the flow is both uniform and steady. The total energy in such a system will

have three components, namely conduit height or diameter, velocity head and pressure head as shown in Figure 3.

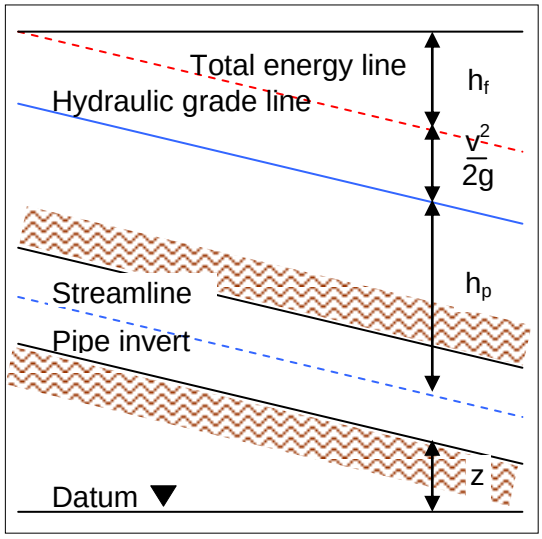


Figure 3: Conduit Flowing Full

The total energy at any point along a conduit operating under pressure can be defined by Bernoulli's equation:

$$H = z + d/2 + h_p + v^2/2g \qquad (1)$$

Where H- total energy above datum in m
z - height of invert above datum in m
d - conduit height or diameter in m
v - velocity in m/s
g - gravitational constant in m/s/s
h_p -pressure head in pipeline in m
h_f -energy loss due to friction in m

As there is pressure in such a conduit, the fluid can be carried uphill provided the value of "h_p" stays positive. Such a system is classified as a pressure pipeline.

On the other hand, a conduit conveying stormwater or sewage generally flows partly full and the flow is frequently both varied and unsteady. There is an air/fluid interface and therefore, no pressure component to the total energy as shown in Figure 4.

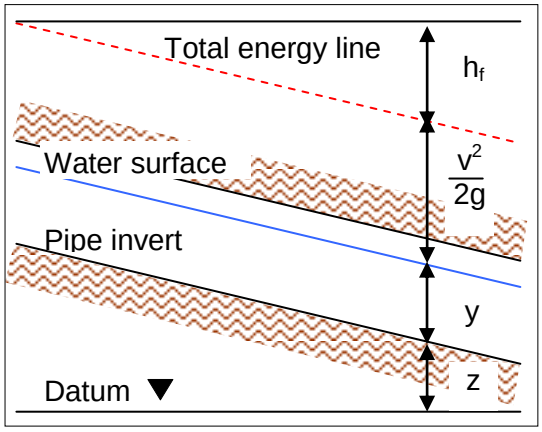


Figure 4: Conduit Flowing Partly Full

The total energy at any point along a conduit flowing partly full can be defined by the Energy equation:

$$H = y + \frac{v^2}{2g} + z \quad and \quad E = y + \frac{v^2}{2g} \qquad (2)$$

Where H, g, v & 8 are as described above
Where E - total energy above invert in m
y - depth of flow in m
v - velocity in m/s
g - gravitational constant in m/s/s

As there is no pressure in such a conduit, the fluid can only flow downhill and the system is classified as a gravity pipeline.

Figures 3 and 4 show systems where the pipe invert, hydraulic grade line or water surface and the total energy line are all parallel. This is called uniform flow and the only energy losses are due to friction. However if there are any transitions such as changes in vertical or horizontal alignment, or the cross sectional shape of the conduit then these will also cause energy losses due to the liquid expanding or contracting.

The means of determining the hydraulic properties of conduits flowing under pressure and those flowing partly full, as open channels are understandably different. A further factor that needs to be considered is the hydraulic length of the conduit.

3.2. HYDRAULIC LENGTH

The hydraulic length of a conduit is determined by the relationship between the energy losses due to friction and those due to transitions. When the energy losses due to friction exceed those due to transitions then the conduit is classified as hydraulically long. When

those due to transitions exceed those due to friction then the conduit is classified as hydraulically short. In general a pipeline is hydraulically long whereas a culvert crossing is hydraulically short.

The energy losses due to friction are determined using one of the friction formulae, such as Manning, to calculate the velocity through the conduit. Manning's equation is:

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

Where v - velocity n m/s

n - Manning's roughness coefficient

R - hydraulic radius

S - gradient of conduit

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 the use of a coefficient as shown in the formula below, is adequate:

$$H_L = k(v^2/2g) \quad (4)$$

Where H_L- head loss in metres (m)

k - a coefficient, usually between 0.0 and 1.0 dependent upon transition details

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

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

Commonly used energy loss coefficients are given in Table 6 below.

TABLE 6: ENERGY LOSS COEFFICIENTS FOR PIPELINE FLOW

Entrance or outlet detail	Entrance	Outlet
Protruding	0.80	1.00
Sharp	0.50	1.00
Bevelled	0.25	0.50
Rounded	0.05	0.20

The friction slope of a pipeline that has no transitions is the energy difference between inlet and outlet, divided by the pipeline length. If there are any transitions in the pipeline, the energy losses due to the transitions will reduce the amount of energy available to overcome friction.

3.3. PRESSURE PIPELINES

The hydraulic performance (velocity and discharge) of a pressure pipeline is determined by using one of the friction formulas such as Manning, in combination with the continuity equation and energy losses at transitions.

The continuity equation is:

$$Q = Av \quad (5)$$

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 low-pressure pipelines flow under gravity and have no additional energy inputs, i.e. no use is made of additional energy to lift the water. If pressure is added to the pipeline by a pump, the energy is increased.

An alternative approach to determining the hydraulic properties of a pipeline is to use a chart for a pipe flowing full as given in Figure 5 and to add any energy inputs or subtract any energy losses at transitions. If the pipeline is flowing under pressure the friction slope should be used, as this will probably be different from the pipeline gradient that could vary along the length of the pipeline.

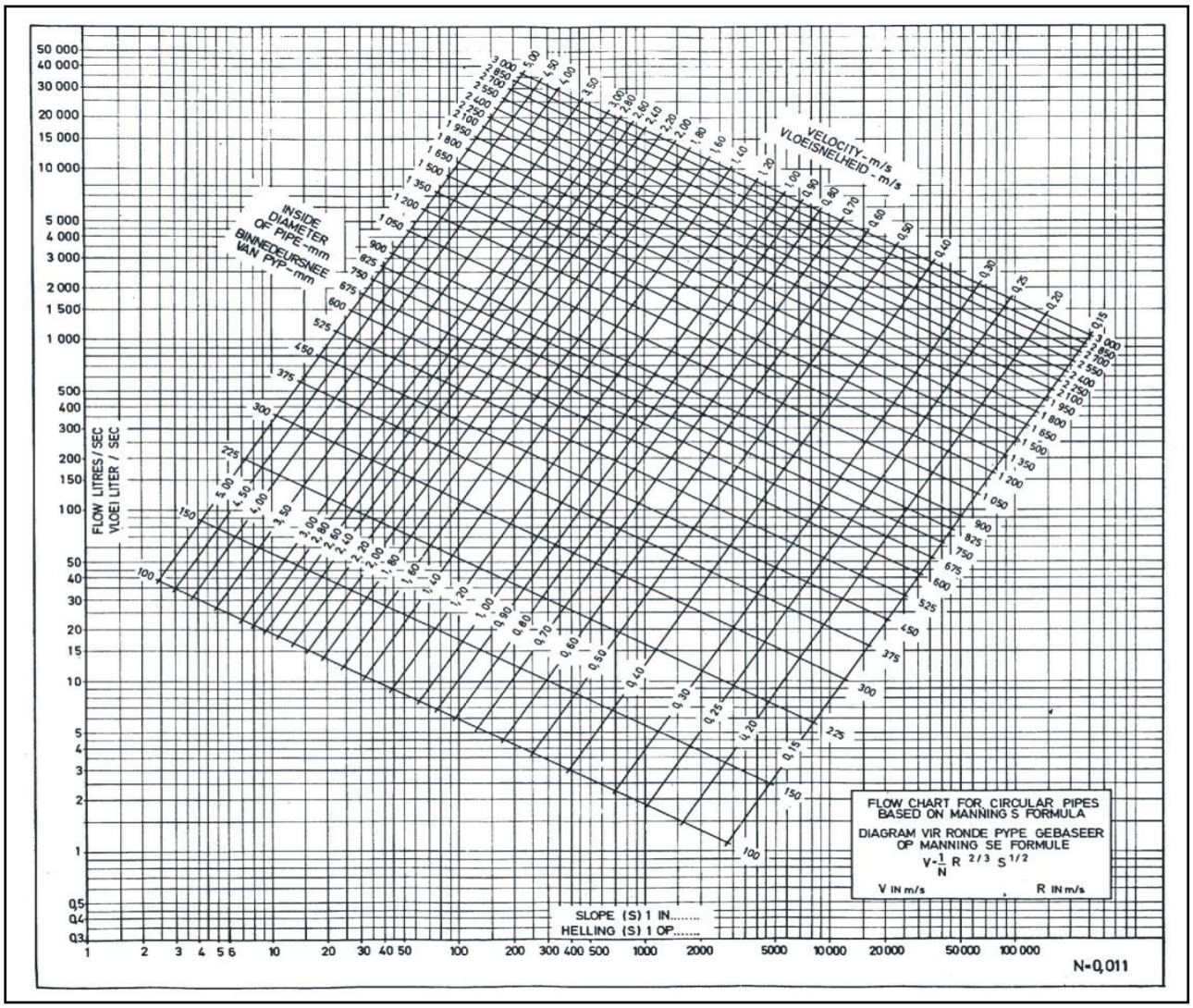


Figure 5: Flow Chart for Circular Pipes Based on Manning Formula

3.4. SEWERS AND STORMWATER OUTFALLS

Most sewer and storm water outfalls consist of sections of hydraulically long conduit flowing party full between transitions (manholes). If the pipeline is hydraulically long and is flowing partly full then the slope of the energy line, the water surface and the pipeline gradient will be the same.

Under these circumstances the sections of pipeline between manholes can be evaluated by using the chart for pipes flowing full, Figure 5 and then adjusting the values using proportional flow as given in Figure 6 that gives the relationship between the relative depth d/D and the other parameters as hydraulic radius, velocity and discharge. Examples of the combined use of these figures are given below Figure 6.

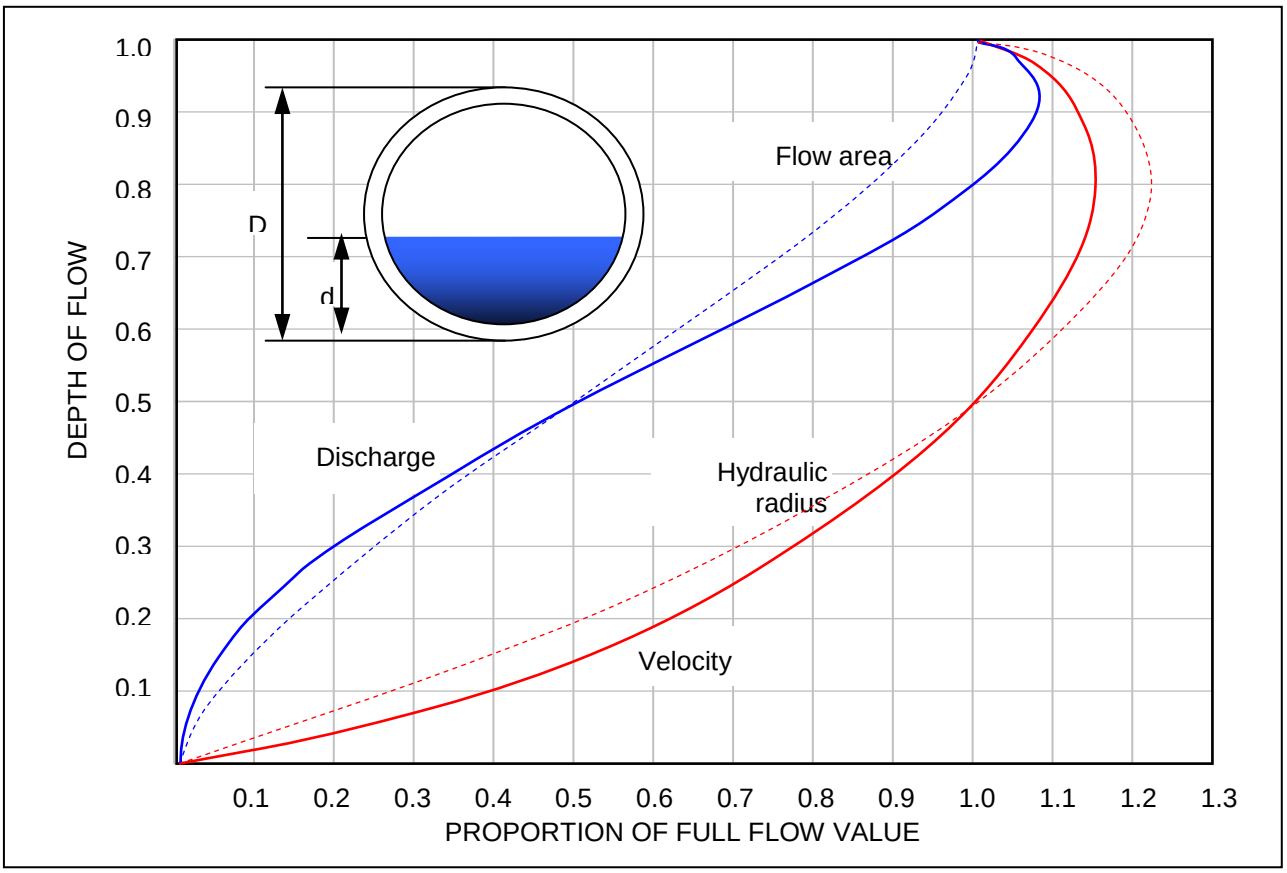


Figure 6: Relative Flow Properties of Circular Pipe Flowing Partly Full

Example 1:Given a 600 mm internal diameter (D) concrete pipeline at a slope of 1 in 1 000 and a discharge of 120 litres per second (Vs), determine velocity and flow depth. Use n = 0.011.

From the flow chart intersecting the co-ordinates of diameter (600) and slope (1 in 1 000) we obtain: Q =240 l/s and V =0,82 m/s

Then Q/Q_{full} = 120/240=0.5 and Figure 6 gives d/D=0.5x600=300 mm and v/v_{full} =1.0x 0.82 = 0.82 m/s

Example 2: Given a flow of 200 l/s and a slope of 1 m in 2 000 m, determine the diameter of a concrete pipe to flow half full. Use n = 0,011

From Figure 6 for d/D = 0.5 ; Q_{full} = Q/0.5 = 200/0.5 = 400 l/s and from Figure 5 for Q = 400 l/s and a slope of 1 m in 2 000 m, D = 900 mm.

3.5. HYDRAULICS OF STORMWATER CULVERTS

The capacity of hydraulically short conduits, such as stormwater culverts is predominantly dependent upon the inlet and outlet conditions. These conduits seldom flow full and the energy losses at inlets and outlets due to sudden transitions far exceed any losses due to friction. Under these circumstances, the charts for pipes flowing full should not be used.

For stormwater culverts the most important hydraulic considerations are:

- Headwater level at the entrance that will determine upstream flooding.
- Roadway overtopping necessitating road closure.
- Outlet velocity that could cause downstream erosion.

The various factors that will influence the flow through a hydraulically short conduit, such as a culvert under a road are illustrated in Figure 7 below.

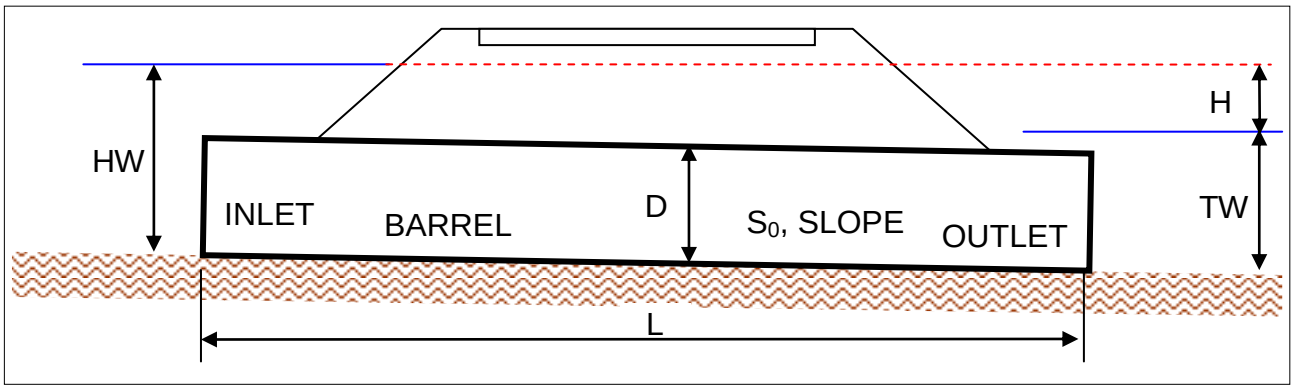


Figure 7: Factors Influencing Flow Through Culverts

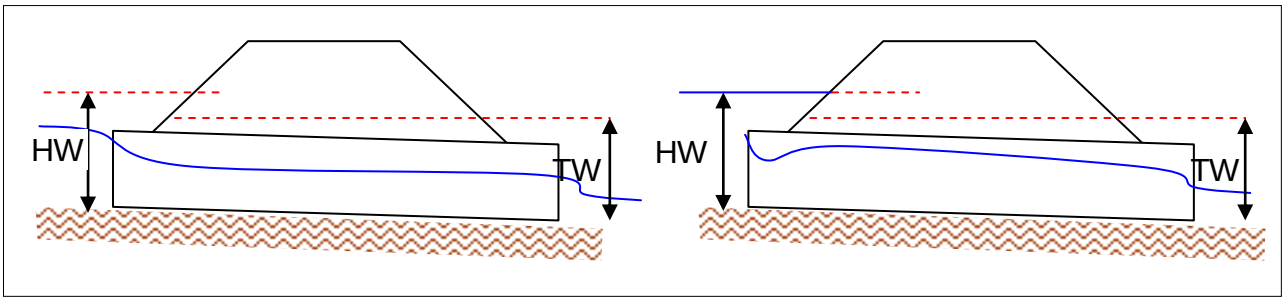
By applying the principle of energy conservation:

$$HW + S_O L = H + TW \qquad (6)$$

Where
- HW - headwater or energy level at inlet in m
- TW - tailwater or energy level at outlet in m
- H - total energy loss between inlet and outlet in m
- D - internal diameter or height of conduit in m
- L - length of conduit in m
- S_0 - culvert gradient in m/m

There are several different types of culvert flow, depending on whether the control is located at the inlet, along the barrel or at the outlet.

Inlet control occurs when the inlet size, shape and configuration controls the volume of water that can enter the culvert. In other words when the capacity of the inlet is less than the capacity of the barrel and there is a free discharge downstream of the culvert.



(a) unsubmerged inlet (b) submerged inlet

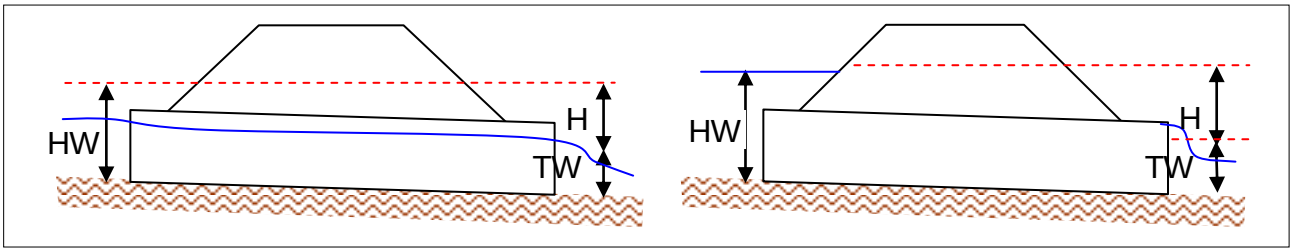
Figure 8: Inlet Control Condition and Variations

This happens when the slope of the culvert is steeper than the critical slope. When the conduit flows with an unsubmerged inlet, the flow passes through critical depth at the entrance to the culvert. When the culvert flows with a submerged inlet, which will occur when HW/D $\geq$ 1.5, the inlet will act as an orifice and the flow will contracted as if flowing through a sluice gate.

The major energy loss will be at the culvert inlet. The total energy through the culvert and the outlet velocity can be calculated from the critical or contracted depth at the entrance.

Barrel control occurs when the barrel size, roughness and shape controls the volume of water that which can flow through the culvert. In other words when the capacity of the barrel is less than the capacity of the inlet and the discharge downstream of it is free.

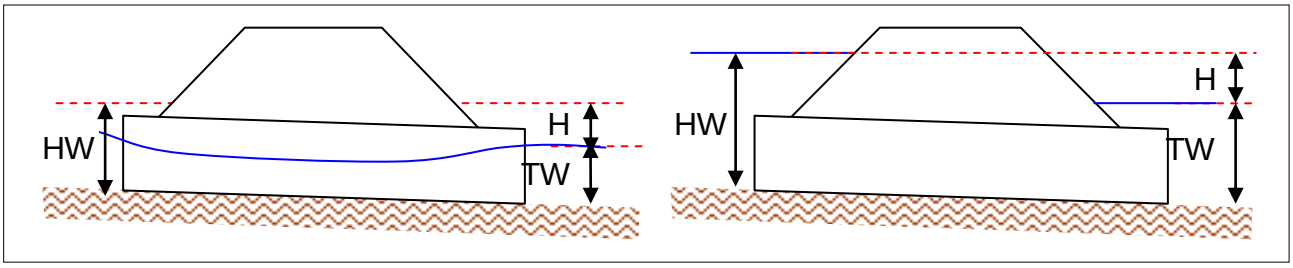
This happens when the slope of the culvert is flatter than critical slope and the constriction at the entrance is drowned out by the flow through the barrel. The major energy loss will be at the outlet. The water surface will pass through critical depth at the outlet and the outlet energy level and velocity can be calculated from this, as described below.



(a) Unsubmerged inlet (b) Submerged inlet

Figure 9: Barrel Control Condition and Variations

Outlet control occurs when the water level downstream of the culvert controls the volume of water that can flow through the culvert by drowning out either inlet or barrel control conditions. In other words when the capacity of the barrel or the inlet cannot be realised because there is no free discharge downstream of the culvert.



(a) Unsubmerged inlet (b) Submerged inlet

Figure 10: Outlet Control Condition and Variations

The water surface will not pass through critical depth at any section of the culvert hence there are no sections where there is a fixed depth discharge relationship. The major energy loss will be at the outlet.

The capacity and headwater depths for the different types of culvert flow can be determined by calculation or from nomographs.

3.5.1. Capacity and Headwater Depth for Hydraulically Short Conduits

When gradients are steep and the flow of water at the outlet of the pipe is partially full, the control will be at the inlet. In other words, more water can flow through the culvert than into it. The capacity and headwater levels for a circular concrete pipe culvert operating under inlet control can be determined using the nomograph given in Figure 11.

When gradients are very flat or the outlet of the culvert is submerged, the control will be either through the barrel or at the outlet. In other words, more water can flow through the entrance to the culvert than through the barrel. The capacity and headwater levels for a circular concrete pipe culvert operating with either barrel or outlet control can be determined using the nomograph given in Figure 12. The value of TW needs to be estimated and HW then calculated from:

$$HW = H + TW - S_O L \qquad (7)$$

Where the terms have already been defined

However, the outlet velocity for the flow through culverts needs to be calculated.

The capacity and headwater levels for a rectangular concrete culvert operating under inlet control can be determined using the nomograph given in Figure 13 and that for a rectangular concrete culvert operating with outlet control is given in Figure 14.

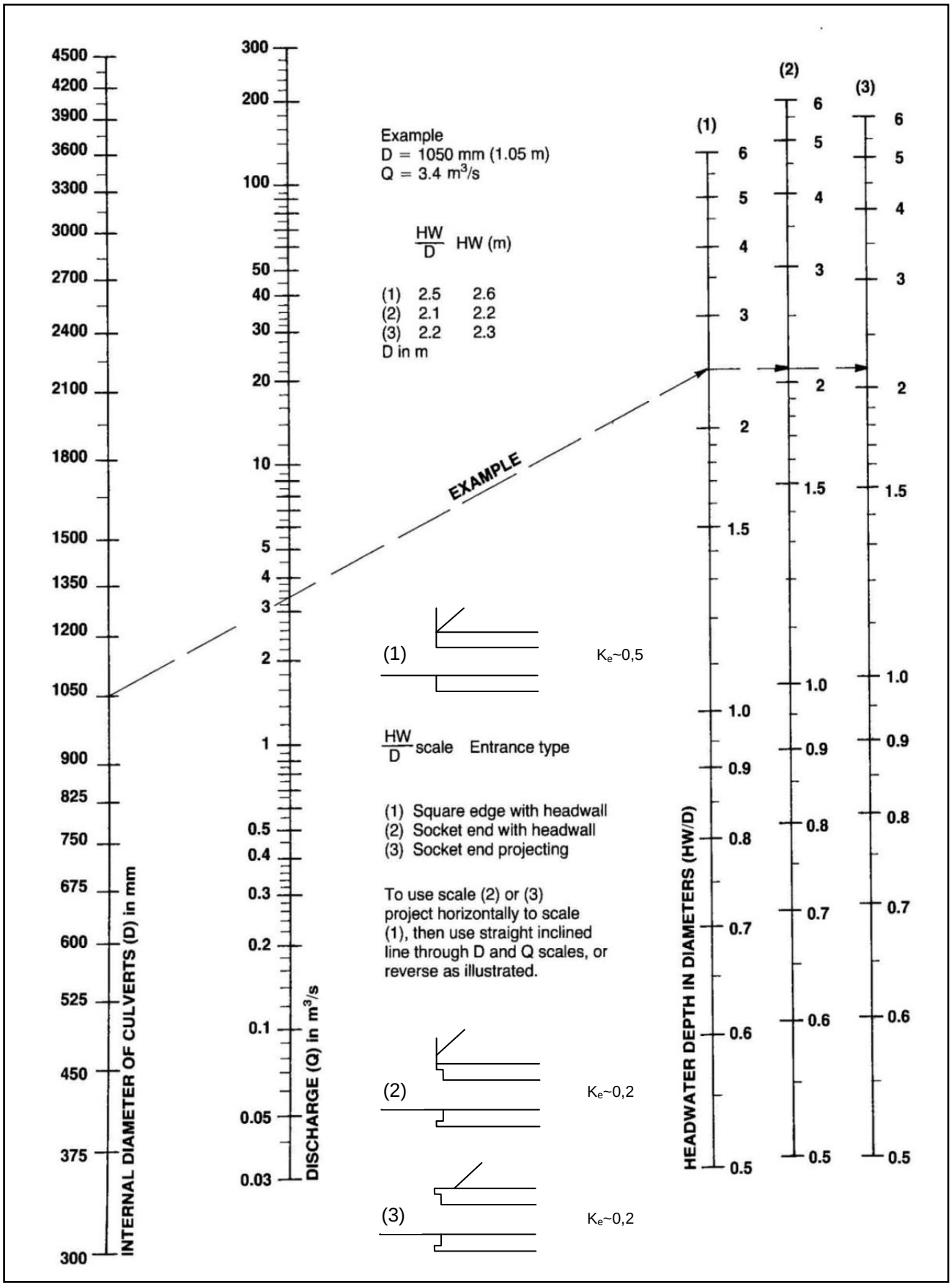


Figure 11: Headwater Depth: Concrete Pipe Culverts: Inlet Control

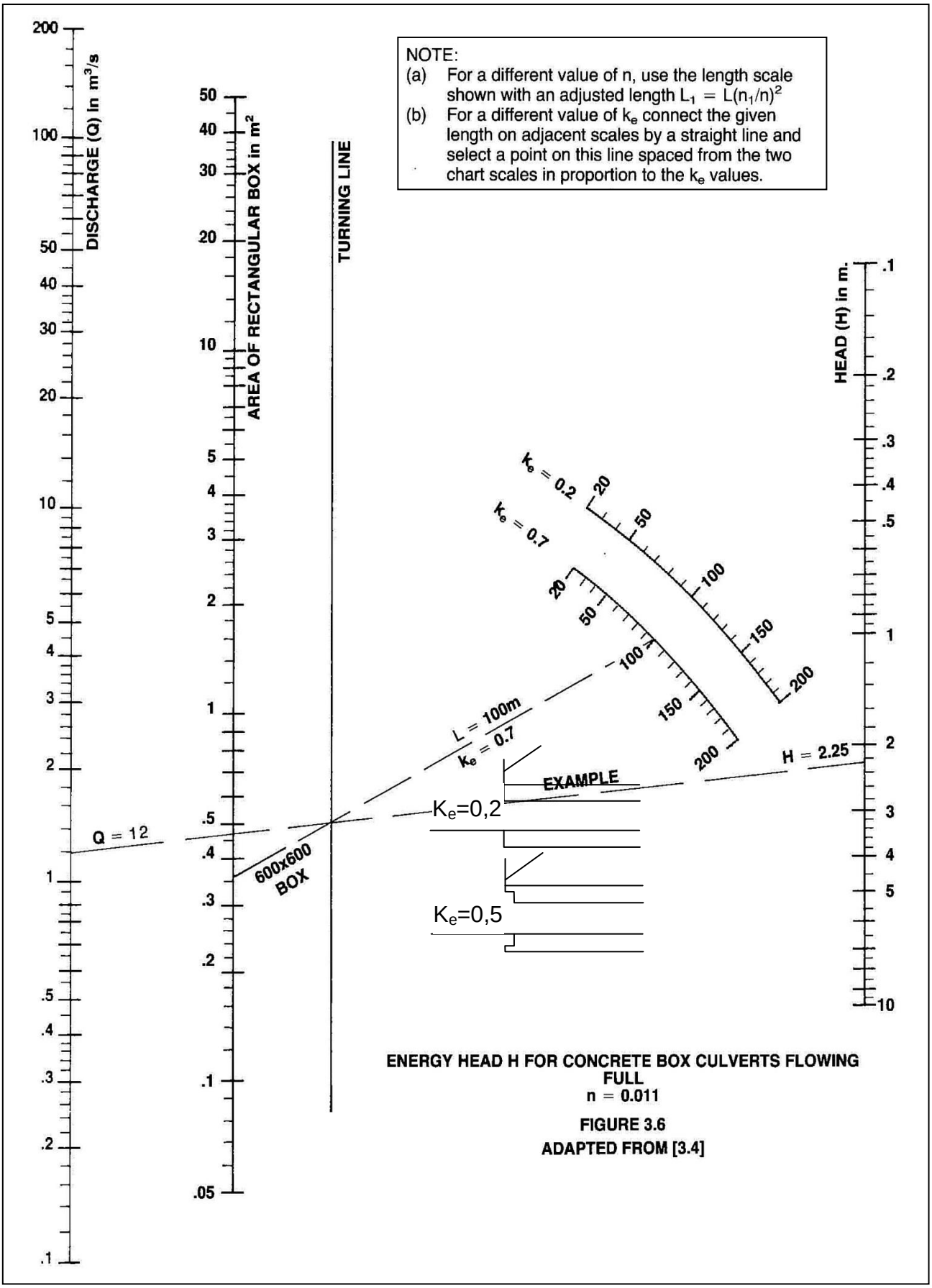


Figure 12: Headwater Depth: Concrete Pipe Culverts: Outlet Control

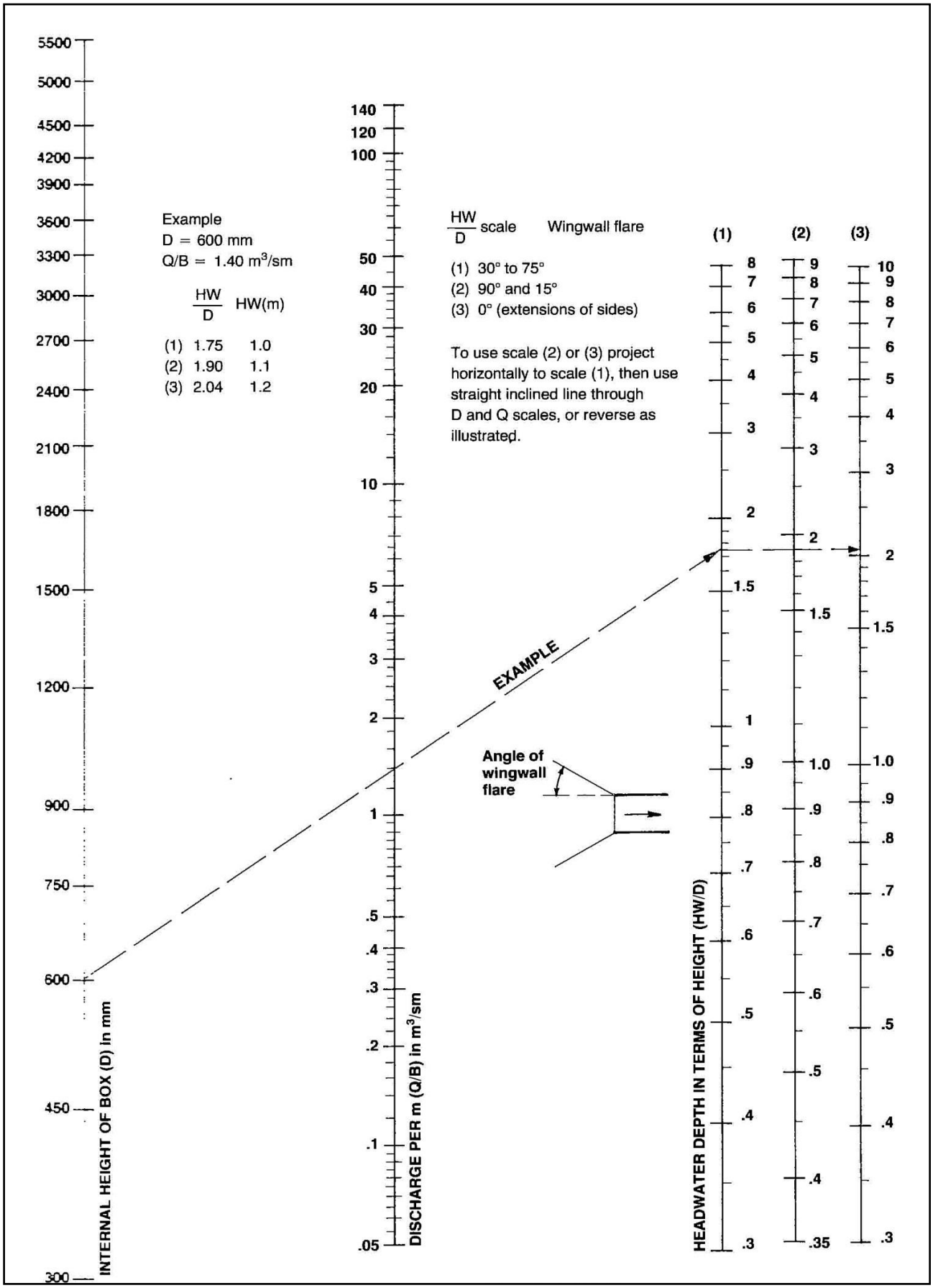


Figure 13: Headwater Depth: Rectangular Culverts: Inlet Control

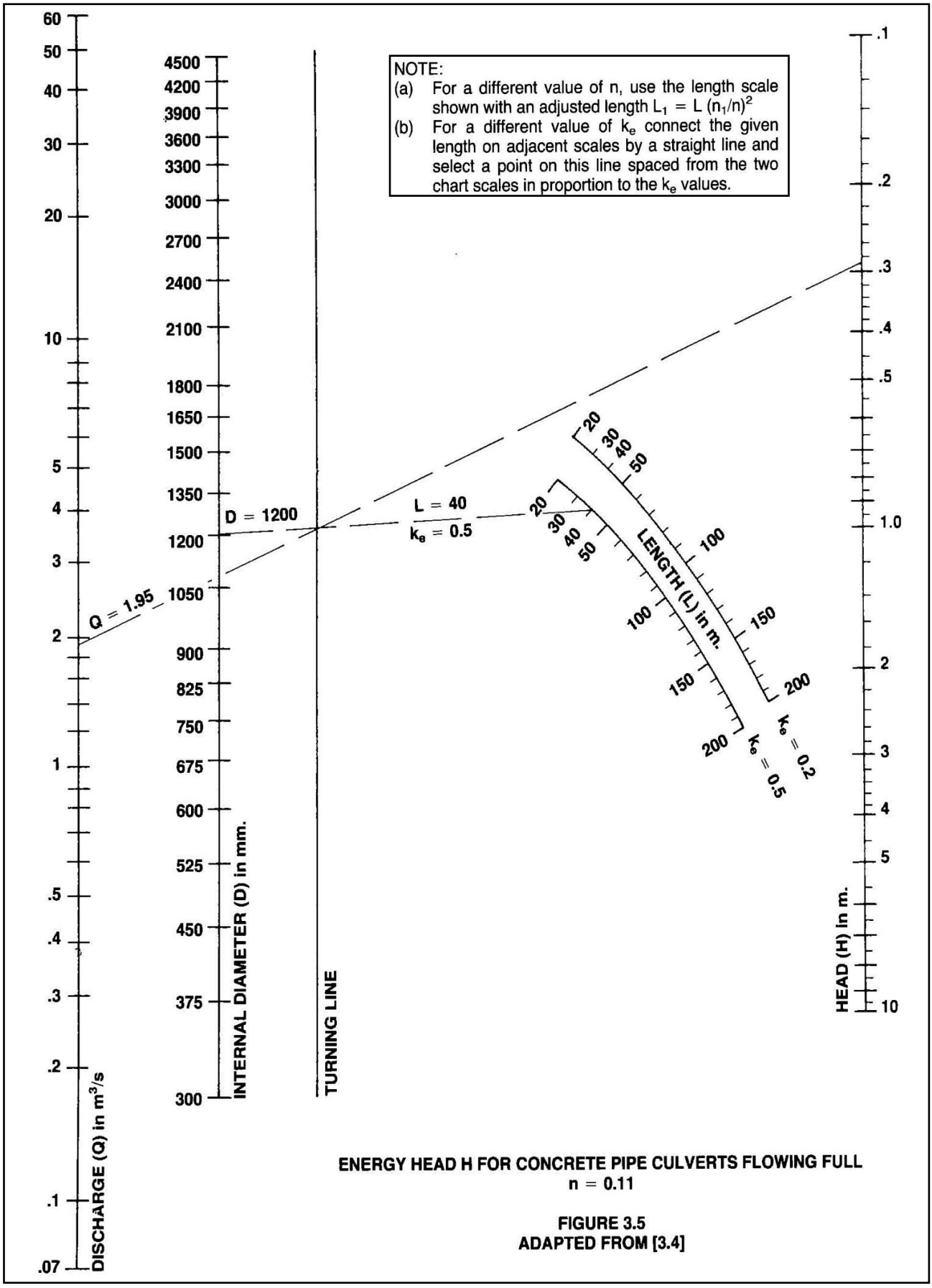


Figure 14: Headwater Depth: Rectangular Culverts: Outlet Control

3.5.2. Outlet Velocity for Hydraulically Short Conduits

Outlet velocity is seldom calculated for culverts, yet it is this that causes downstream erosion and wash-a-ways that can result in recurring maintenance costs. The exact calculation of outlet velocities is difficult. However, conservative estimates can be made using the procedures that follow.

For culverts flowing with inlet or barrel control, the outlet velocity can be calculated by identifying the control point at the entrance or outlet where the depth discharge relationship is fixed. For a culvert of any cross-sectional slope, the critical depth will occur when

$$Q^2T/gA^3 = 1 \qquad (8)$$

Where: Q - discharge in m^3/s
T - flow width in m
G - gravitational constant in meters/second per second (m/s/s)
A - flow area in m^2

For a rectangular section this reduces to

$$d_c = v_c^2/g \qquad (9)$$

Where: d_c - the critical depth in m
v_c - the critical velocity in m/s

There is no simple equation for the relationship between critical depth and velocity in a circular pipe. However, the use of the above equation will over estimate the velocity by about 10%. Hence, it will be adequate for most stormwater drainage applications.

For the inlet control condition with an unsubmerged inlet, the outlet velocity can be calculated from the critical energy level at the inlet to the culvert. If the inlet is submerged, the outlet velocity can be calculated from the energy level at the inlet, which is obtained by subtracting the inlet energy loss from the headwater depth. This is calculated using the relevant coefficient from Table 6.

For the barrel control condition, the flow will pass through critical depth at the outlet and the outlet velocity can be calculated from this.

For the outlet control condition, outlet velocity should not be a problem as it is the downstream conditions that drown the flow through the culvert. If the outlet is not submerged, the outlet velocity can be calculated by assuming that the flow depth is the average of the critical depth and the culvert height in diameter. If the outlet is submerged, the outlet velocity will be the discharge divided by culvert area.

3.6. POROUS PIPES

Porous pipes are used as a means of subsoil drainage and have the following applications:

- Subsurface drainage under roads and railways where the presence of seepage water from a high water table would be detrimental to the foundations of the road or railway
- Under reservoirs and other water retaining structures where the effects of leaks and uplift can be minimised and controlled by subsoil drainage
- Under large areas such as parks, airports and agricultural holdings, where the subsoil must be well drained.

Designing a subsoil drainage system is based on the same hydraulic principles as normally used for determining pipe sizes. The primary problem is determining the flow, which is dependent on soil characteristics, the area to be drained and rainfall. The flow in the subsoil drainage system will depend on the judgement of the designer. Table 7 below gives some guidelines.

TABLE 7: APPROXIMATE FLOW LITRES/SEC PER HECTARE: VARIOUS CONDITIONS

Soil Type	Rainfall per annum – mm			
	<750	750 – 1000	1000 – 1200	>1200
Clays	0.45	0.55	0.75	1.20
Loams	0.60	0.80	1.00	1.70
Sandy soils	0.85	1.10	1.50	2.40

The optimum spacing and depth of a subsoil drain is largely dependent on the type of soil. Where large areas are to be drained Table 8, that gives the capacity of porous pipes and Table 9, that gives a guide to spacing in metres for various soils and drain installation depths can be used to estimate the size and spacing of pipes for a subsoil drainage system.

TABLE 8: FLOW CAPACITY OF POROUS PIPES IN LITRES PER SECOND

Internal diameter (mm)	Slope of pipe in m/m				
	0.001	0.005	0.01	0.05	0.10
100	1.2	2.7	3.9	8.6	12.2
150	3.6	8.1	11.4	25.8	36.4
200	8.3	18.3	26.1	58.9	82.8
300	25.8	57.8	81.9	183.3	258.3

Although a slope of 0.001 is theoretically possible, slopes of less than 0.005 are not practical. The spacing of drains, not hydraulic considerations, normally controls the design of a system.

TABLE 9: POROUS PIPE SPACING IN METRES FOR DIFFERENT SOIL TYPES

Pipe depth in m	Clays	Loams	Sandy clay
0.6 – 0.9	7 – 10	10 – 12	12 – 25
0.9 – 1.2	9 – 12	12 - 15	25 – 30

Although the tables only indicate sizes up to 300 mm in diameter, larger sizes may be available from certain pipe manufacturers. As there is no South African standard for these pipes the porosity standards from BS 1194, as given in Table 10 are used. The manufacturers should be asked for details of the crushing strengths for porous pipes.

TABLE 10: POROSITY VALUES IN LITRE PER SEC PER METRE OF PIPE LENGTH

Pipe diameter in mm	100	150	200	300
Porosity litre per sec per metre length	1.0	2.0	2.5	5.0

4. LOADS ON BURIED PIPELINES

4.1. INTRODUCTION

Every buried pipeline is subjected to loads that cause stresses in the pipe wall. These loads can be broadly defined as primary loads and secondary loads.

Primary loads can be calculated and include:

- mass of earth fill above pipe
- traffic loading
- internal pressure loading.

Other primary loads are pipe and water masses that can be ignored, except in critical situations. For stormwater drains and sewers that in most cases operate as gravity pipelines that flow partly full internal pressures due not have to be considered.

To deal with the possibility of such a pipeline being surcharged due to a blockage the pipes should be able to handle a nominal internal pressure that is well within the capability of the standard strength concrete pipes.

Secondary loads are not easy to calculation as they are variable, unpredictable and localised. They can however cause considerable damage to a pipeline due to differential movements between pipes. It is therefore essential that their potential impact be recognised and that where necessary that precautions are taken. Examples of factors that could cause secondary loads are:

- Volume changes in clay soils due to variations in moisture content
- Pressures due to growth of tree roots
- Foundation and bedding behaving unexpectedly
- Settlement of embankment foundation
- Elongation of pipeline under deep fills
- Effects of thermal and moisture changes on pipe materials and joints
- Effects of moisture changes and movements on bedding
- Restraints caused by bends, manholes etc.

It is preferable to avoid or eliminate the causes of these loads rather than attempt to resist them. Where this is not possible, particular attention must be paid to pipe joints and the interfaces between the pipeline and other structures, such as manholes to ensure that there is sufficient flexibility. The reader is referred to the section of this handbook dealing with joints.

Where pipelines operate in exposed conditions such as on pipe bridges or above ground, the pipes will be subject to thermal stresses and longitudinal movement. The thermal stresses are caused by temperature differences between the inside and outside of the pipe that alternate between night and day resulting in the pipe walls cracking due to cyclical strains. This is generally not a problem when the pipe walls are less than 100mm thick. The longitudinal movement is caused by the expansion and contraction of the pipeline due to temperature changes. The design of the pipe and pipeline for such conditions should be discussed with a competent manufacturer or specialist consultant so that the necessary precautions can be taken to cope with these effects and ensure that the pipeline will operate satisfactorily. These are beyond the scope of this handbook.

4.2. EARTH LOADS

The calculation of earth loads on a buried conduit from first principles is complex. For a thorough understanding, reference should be made to the specialist literature and SANS 10102 Parts 1 and 2. The prime factors in establishing earth loads on buried conduits are:

- installation method
- fill height over conduit
- backfill density
- trench width or external conduit width
- settlement of foundation material

To use the tables in this handbook, it is necessary to understand the various methods of installing buried conduits. The two basic installation types and the corresponding loading conditions are the trench and the embankment conditions. These are defined by whether the frictional forces developed between the column of earth on top of the conduit and those adjacent to it reduce or increase the load that the conduit has to carry.

A useful concept is that of the geostatic or prism load. This is the mass of earth directly above the conduit assuming that there is no friction between this column of material and the columns of earth either side of the conduit. The geostatic load will have a value between that of the trench and embankment condition. These loading conditions are illustrated in Figure 13 below and the key to materials used is given in Table 11.

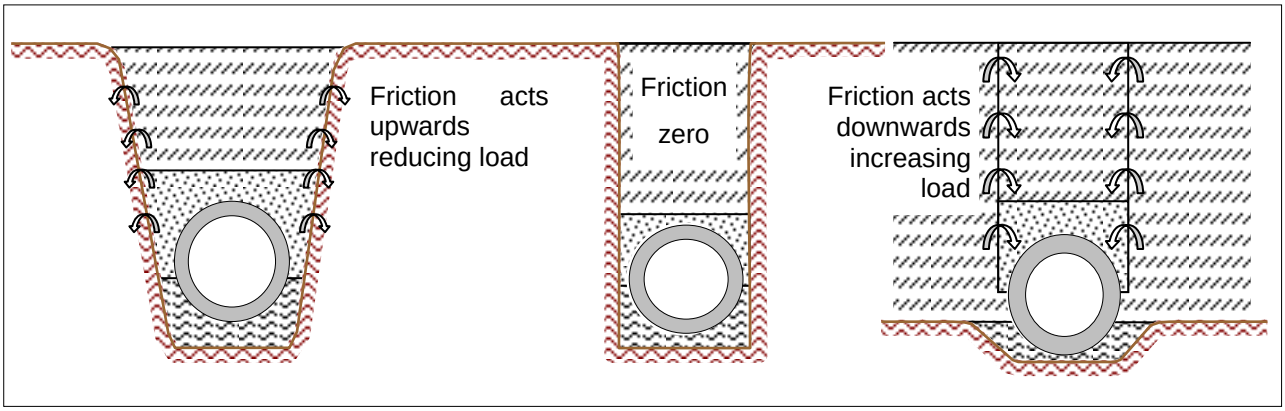


Figure 15: Comparison of Trench, Geostatic and Embankment Loading

TABLE 11: KEY TO MATERIALS USED

Insitu Material	Lightly compacted backfill	Selected granular material
Main backfill	Densely compacted backfill	Fine granular fill material
Loose backfill	Reworked foundation	Compacted granular material
Concrete	Soilcrete	

4.2.1. Trench condition

The trench condition occurs when the conduit is placed in a trench that has been excavated into the undisturbed soil. With a trench installation the frictional forces that develop between the column of earth in the trench and the trench walls act upwards and reduce the load that the conduit has to carry. As a result the load on the conduit will be less than the mass of the material in the trench above it. The load on the conduit is calculated from the formula:

$$W_t = C_t \gamma B_t^2 \qquad (10)$$

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

The coefficient C_t can be calculated from:

$$C_t = \frac{1-e^{-2K'_{\mu}.H/B_t}}{2K_{\mu}'} \quad (11)$$

Where C_t – trench earth grading coefficient

K – coefficient of actual lateral earth pressure

μ' – angle of sliding friction between backfill and trench sides

H – fill height and B_t is trench width

The formula indicates the importance of the trench width B_t that 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 called the transition width. When this occurs the embankment load can be used.

The determination of the transition width is covered in the specialist literature. It is safe to assume that any trench width that gives loads in excess of those given by the embankment condition exceeds the transition width. The settlement of the foundation material has a negligible effect on the trench loading. 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:

$W_E = 2.63\gamma Bt^2$ in sandy conditions (12)

$W_E = 3.84\gamma Bt^2$ in clayey conditions (13)

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

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

B_t - trench width on top of conduit in m

Earth loads due to trench loading on circular pipe where the trench widths as specified in SANS 1200 DB and nominal pipe diameters (ND), are given in Table 12. Earth loads due to trench loading on conduits where the trench widths are specified but the conduit dimensions are not are given in Table 13.

TABLE 12: TRENCH LOADS ON CIRCULAR PIPE IN KN/M; NON-COHESIVE SOIL (GROUP NO 1 SANS 10102 PART 1); TRENCH WIDTHS SANS 1200 DB.

Pipe ND mm	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
225	0.859	9	15	21	26	30	34	37	40	44	48	50
300	0.945	10	17	23	29	34	39	42	46	51	56	59
375	1.031	11	18	26	32	38	43	48	52	59	64	68
450	1.118	13	20	28	36	42	48	54	58	66	72	77
525	1.204	14	22	31	39	47	53	59	64	74	81	87
600	1.290	15	23	33	42	51	58	65	71	81	90	97
675	1.376	16	25	36	46	55	63	70	77	89	99	107
750	1.663	19	31	44	57	69	80	90	99	115	129	141
825	1.749	20	32	47	61	73	85	95	105	123	139	152
900	1.835	21	34	50	64	77	90	101	112	131	148	163
1050	2.208	26	42	61	79	96	112	127	141	167	190	210
1200	2.380	28	45	66	86	104	122	138	154	183	209	233
1350	2.620	31	50	73	95	116	136	155	173	207	237	264
1500	2.800	33	53	78	102	125	147	167	187	224	258	288
1650	2.980	35	57	84	109	134	157	180	201	242	278	312
1800	3.360	39	65	95	125	153	180	206	231	279	323	363

Notes

1) For nominal pipe diameters ≤ 1200mm the external diameter has been taken as 1.15 times the nominal diameter; for larger sizes 1.2 times the nominal diameter.
1. Table 11 for non-cohesive soil; gravel or sand; density = 20 kN/m^3 and K_{μ}' = 0.19.
2. The table is based on the trench widths recommended in SANS 1200DB.
3. If the soil unit weight is known, the loads from the table may be adjusted as follows: Load on pipe = load from table x unit weight of soil / 20
4. Procedure is valid only if the soil properties other than unit weight don't change.

TABLE 13: LOADS ON ANY CONDUIT IN KN/M FOR GIVEN TRENCH WIDTHS

Trench Width in 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 13 is for same installation conditions and soil properties used in Table 12.

4.2.2. Embankment condition

In this condition the conduit is installed at ground level and is covered with fill material. All the earth surrounding the conduit is homogeneous and the compaction is uniform. With an embankment installation the frictional forces that develop between the column of earth directly above the conduit and the columns of earth adjacent to the conduit, act downwards and increase the load that the conduit has to carry. Hence the influence of the founding material on embankment loading is significant. The load on the conduit will be greater than the mass of the material directly above it due to the frictional forces that develop. For the complete projection condition the load on a conduit is calculated from the formula:

$$W_e = \gamma C_e B_c{}^2 \qquad (14)$$

Where W_e - load on pipe in kN/m

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

B_c- overall diameter of pipe

C_e-coefficient that is function of fill material, conduit outside width, fill height, projection ratio, and founding conditions. In addition the founding material under the conduit could yield and partly reduce the load that it has to carry.

The projection ratio is a measure of the proportion of the conduit over which lateral earth pressure is effective. It is calculated from p = x / B_c, where x -height that conduit projects above or below the natural ground level.

The coefficient c_e can be calculated from:

$$c_e = \frac{e^{(2K_\mu . H/B_c)} - 1}{2K_\mu} \qquad (15)$$

Where C_e – embankment earth loading coefficient

K – coefficient of active lateral earth pressure

μ - angle of sliding friction of backfill material

H – fill height

B_c – outside diameter of pipe

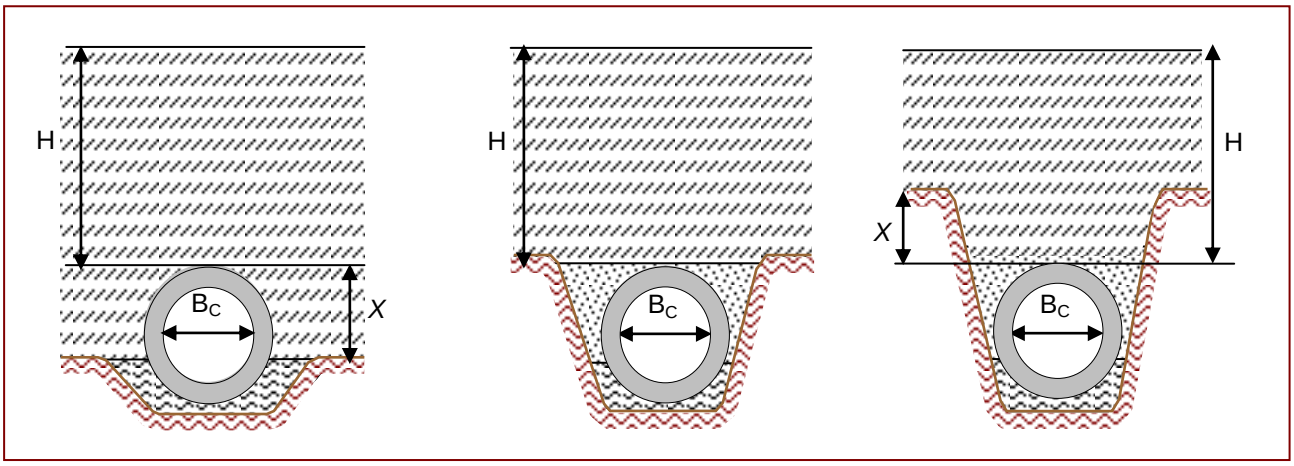
In addition the founding material under the conduit could yield and partly reduce the load that has to be carried. The settlement ratio, designated as r_s, is a measure of the amount that the founding material under the conduit settles. Values of this parameter are given in table 14 below.

TABLE 14: VALUES OF SETTLEMENT RATIO

Material type	Rock or	Unyielding soil	Normal soil	Yielding soil
Settlement ratio, r_s	1.0	1.0	0.7	0.3

The various types of embankment condition, illustrated in Figure 16 are:

- Positive projection where top of the conduit projects above the natural ground level.
- Zero projection where the top of conduit is level with natural ground. 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 where top of the conduit is below the natural ground level. As the trench depth increases, this condition approaches a complete trench condition.



(a) Positive projection (b) Zero projection (c) Negative projection

Figure 16: Types of Embankment Installation.

For all practical purposes the earth loading from a positive projecting installation will have a maximum value when the pr_s ratio has a value of 1.0, and can be calculated from equations (16) and (17) where the relationship between load and fill height is linear.

$$W_e = 1.69\gamma B_c H \text{ in sandy soils} \quad (16)$$

$$W_e = 1.54\gamma B_c H \text{ in clayey soils} \quad (17)$$

Where these symbols are defined above and in figure 16.

As equations (14) and (15) give upper limits, the smaller of the load calculated from them or equations (10) or (11) should be used.

Earth loads due to embankment loading on circular pipes are given in Table 15 below.

TABLE 15: POSITIVE PROJECTION EMBANKMENT LOADING IN KN/M ON BURIED CONDUITS; NON-COHESIVE MATERIAL; DENSITY 20 KN/M^3, Kμ = 0.19; PR$_S$ = 0.7

Pipe N/D mm	Pipe O/D mm	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
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	2016	27	47	74	104	136	171	210	252	348	438	511

Notes:

1) Table 15 compiled for non-cohesive material with density of 20 kN/m^3 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 value falls in shaded area, it may be used irrespective of the pr_s value.
   (b) If load value to the right of shaded area, multiply the value by following factors:

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

Example 1. *Determine of backfill load under the following conditions: Embankment installation, positive projection. Pipe D = 525 mm; Projection ratio: x/D = 0.7; Foundation material: rock (r_s = 1); Density of fill: 1 750 kg/m^3; Height of fill above top of pipe: 3.5 m.*

pr_s = 0,7 *1 = 0.7; Table 15 applicable with correction for density and pr_s. For ND = 525 mm and height = 3.5 m, Load on pipe = 71.0 kN/m. Applying density correction, the actual load on pipe, W = 71(1750/2000) = 62.1 kN/m. Applying ors correction the actual load on pipe, W = 62.1 x 0.94 = 58.4kN/m

Example 2. *Determine backfill load under the following conditions: Embankment installation, positive projection; Pipe D = 750 mm; Projection ratio = 0.70; Foundation material: ordinary soil: (r_s = 0.7); Density of fill: 1 600 kg/m^3; Height of fill above top of pipe = 2.5 m; pr_s = 0.7 x 0.7 = 0.49 (say 0.5)*

From Table 15 for ND = 750 mm and height = 2.5; Load on pipe = 73 kN/m; Applying density correction, W = 73(1600/2000) = 58.4 kN/m. Since pr_s = 0.5 and value of load falls to the right of the heavy line, actual load on pipe is: W = 58.4 x 0.9 = 52.6 kN/m

4.2.3. Induced Trench Installation

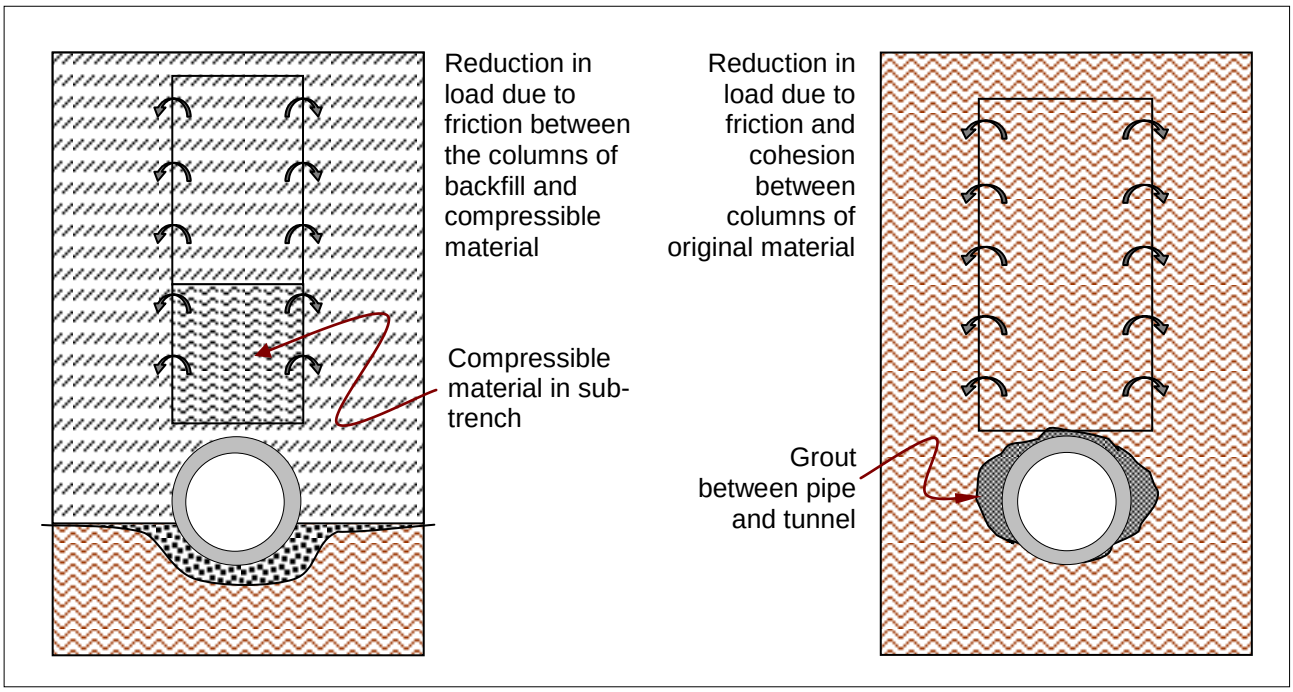
The induced trench installation is a special technique used to increase the height of the fill that can be carried by standard strength conduits under very high embankments (see Figure 15(a)). The procedure followed is to:

- Install the conduit as normally done in an embankment installation
- Backfill over it to the required height
- Dig a trench of the same width as the outside dimension of the conduit down to ± 300mm from the top of the conduit
- Fill the sub-trench with a compressible material as straw or sawdust
- Complete backfilling up to formation level as for a standard embankment installation.

The yielding material in the sub-trench settles and thus produces frictional forces that reduce the load on the conduit. The deeper the sub-trench the higher the frictional forces developed and hence the greater the reduction in load to be carried by the conduit.

Under very high fills, where standard pipe/bedding class combinations or portal culvert classes are inadequate to cope with the earth loads standard product classes are used and the sub-trench depth is adjusted to reduce the load to the required value. An important fact to appreciate with this type of installation is that the settlement in the sub-trench must not be so great that the top of the formation settles. In other words there must be sufficient fill over the conduit to allow a plain of equal settlement to form below the top of the formation. Details of this are shown in Figure 17(a).

The procedure for calculating the depth of sub-trench is given in SANS 10102 Part I. The designer should not use this procedure without first doing a detailed study.



(a) Induced trench (b) Jacked Installation

Figure 17: Special Installations

4.2.4. Jacked Installation

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

This technique involves:

- Excavating a pit at the begging and end of the proposed line.
- Constructing a launching pad in the entry pit
- Pushing a jacking shield against the face of the pit
- Tunnelling through the soil while being protected by the jacking shield by making an excavation slightly larger than the shield just ahead of it
- Pushing conduits into the tunnel as it progresses
- Grouting the space left between the outside of the conduit and the tunnel.

With a jacked installation the vertical load on the conduits will be significantly less than that experienced in a trench installation. This is because the load is dependent on the outside dimension of the conduit and not the trench width and as the soil above the conduits is undisturbed the load is reduced by both cohesion and friction. Once the fill height over the conduit exceeds about 10 times its outside width full arching will take place and no matter how much higher the fill there will be no further increase in the load that the conduit has to carry.

It is seldom that a pipe with a strength class exceeding 70D is required.

However the longitudinal stresses in the pipes will be much higher than those on a pipe installed in an open excavation. It may therefore be necessary to use pipes with thicker walls to handle these forces. For this reason jacking pipes are usually specified as 100D to ensure that the jacking surface area is sufficient.

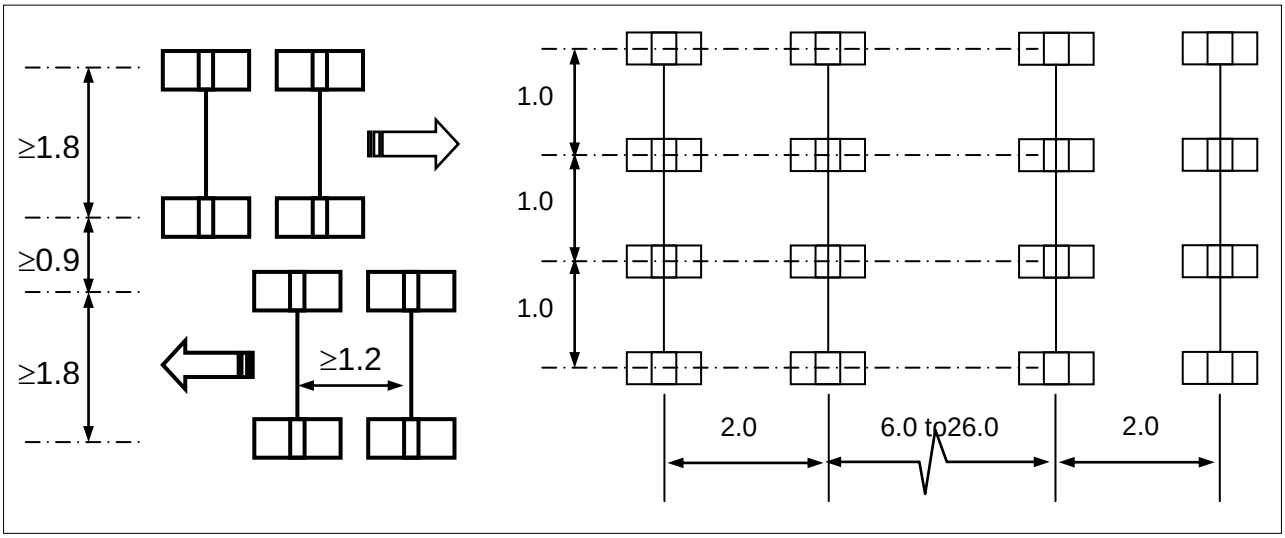
4.3. TRAFFIC LOADING

Where conduits are to be installed under trafficked ways details of the vehicles using them should be determined in terms of:

- Axle spacing and loads
- Wheel spacing, loads and contact areas

The type of riding surface and height of fill over the conduits should also be determined.

Most concrete pipes and portals that are subject to live loads are those used under roads. In this handbook two types of design vehicle have been considered, namely a typical highway vehicle that has two sets of tandem axles and the NB36 vehicle, associated with abnormal loads on national highways (as described in TMH7). As the typical highway vehicle may be overloaded or involved in an accident it is not suitable as a design vehicle under public roads. The design loads as given in TMH7 should be used for the design of all structures under major roads. Under most conditions the loading from the NB36 vehicle is the most critical for buried storm water conduits. The typical legal vehicle would be used for the design of conduits in areas outside public jurisdiction. The most severe loading will occur when two such vehicles pass, or are parked next to each other. Figure 18 illustrates the wheel configuration of these vehicles.



(a) 40kN wheel loads – legal limit (b) NB36 loading – 90kN wheel loads

Figure 18: Traffic Loading on Roads

For the NB loading, 1 unit = 2.5 kN per wheel = 10 kN per axle and = 40 kN per vehicle.
For the NB36 vehicle = 90 kN per wheel = 360 kN per axle.

When the effect of these loads is considered on buried conduits an allowance for impact for impact should be made. For 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 could be applied. The effective contact area for these wheels is taken as 0.2 m x 0.5 m in direction of and transverse to direction of travel respectively.

The loads on pipes due to 40 kN wheel loads with the configuration shown in Figure 18(a) are given in Table 15. The table can be used for any wheel load (P) provided that the wheel arrangement is the same and the load multiplied by P/40.

The loads given in TMH7 for the design of structures under major roads are:

- Normal loading (NA)
- Abnormal loading (NB)
- Super loading (NC)

The NB36 loading is usually the critical one for buried conduits. TMH7 allows an equivalent point load to be used for NB loading that is dependent upon the outside width and length of the conduit. For the NB36 loads this is expressed as:

$$Q_b = 1.25(90 + 12L_s{}^{1.8}) \qquad (18)$$

Where Q_b - equivalent point load
L_s - effective span of conduit in m

The loads on a buried conduit at various depths due to the point load are given in Table 17. Note that the load obtained by the distribution of the NB36 group of wheels as done in Table 16 would be slightly different.

TABLE 16: LOADS IN KN/M ON BURIED CONDUIT, GROUP OF 40 KN WHEELS

Pipe ND mm	Pipe OD mm	Fill height over pipes in m										
		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
456	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
675	776	18.3	10.7	6.4	4.2	3.0	2.2	1.7	1.4	0.9	0.6	0.5
750	863	20.4	11.9	7.1	4.7	3.3	2.5	1.9	1.5	1.0	0.7	0.5
825	949	22.4	13.1	7.8	5.2	3.7	2.7	2.1	1.7	1.1	0.8	0.6
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	2016	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 should certainly be considered for low fills (<diameter of pipe).
3. The tables do not apply to pipes on concrete bedding.
4. Where the cover over the pipe is less than half the outside pipe diameter the bedding factor for the live load must be reduced. Special precautions as concrete encasement may be necessary.

TABLE 17: LOADS IN KN/M ON BURIED PIPES FROM NB36 GROUP OF WHEELS

PIPE I/D mm	PIPE OD mm	FILL HEIGHT OVER PIPES IN M											NB36 PT LOAD
		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
456	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
675	0.776	46	24	14	9	6	4	3	2	2	1	1	120
750	0.863	49	25	15	9	6	5	4	3	2	1	1	121
825	0.949	52	27	17	10	7	5	4	3	2	1	1	123
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

Notes

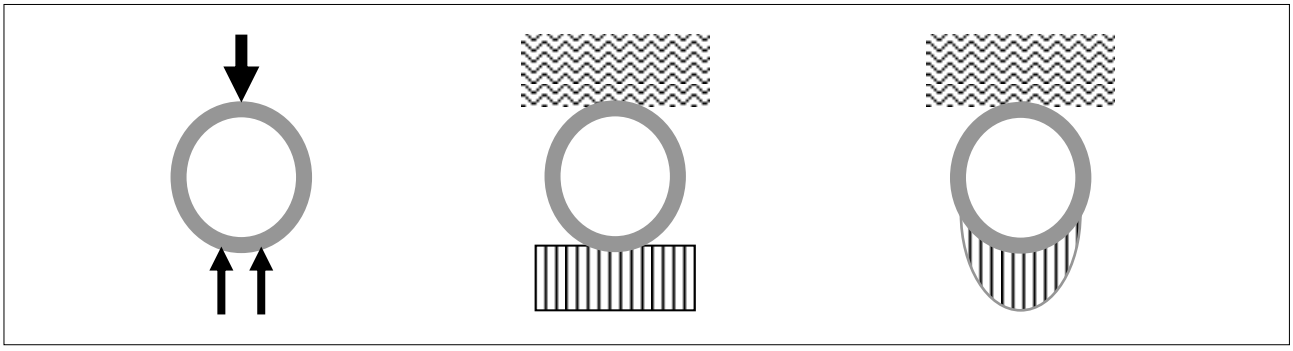
1. The NB36 vehicle travels slowly and generally no impact needs to be considered.
2. Under certain conditions the NB24 vehicle could be used for minor roads, in which cause these loads would be multiplied by 2/3.

5. CONCRETE PIPE STRENGTHS

5.1. EXTERNAL LOADS

The size of circular pipes is defined by one dimension only. This simplifies the relationship between the load to be carried and the strength required to do so. For rigid pipes as concrete the strength is usually determined by using what is called the direct method.

Using the information from the previous sections the required concrete pipe strength can be determined by dividing the installed load by a bedding factor. Factory test loads and reactions are concentrated. The field loads and reactions have a parabolic or radial distribution around a pipe. However it is assumed that the loads are uniformly distributed over the pipe and that the bedding reactions have either a parabolic or uniform distribution dependent upon the bedding material used. A comparison of these loads and reactions is shown in Figure 16



a) Three edge bearing test b) Uniform reaction c) Parabolic reaction

Figure 19: Factory Strength as Model of Installed Load on Pipe

Bedding factors have been derived for standard bedding classes and are described in detail in Section 6 that follows. 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 taken into account and hence the embankment bedding factors are somewhat higher than those used for a trench installation. For most installations the bedding factors given in Table 18 below are adequate.

TABLE 18: BEDDING FACTORS FOR CONCRETE PIPE

Bedding details			Installation details	
Class	**Material**	**Angle**	**Trench**	**Embankment**
A	Reinforced concrete	180	3.4	4.8
A	Concrete	180	2.6	3.9
B	Granular	180	2.0	2.4
C	Granular	60	1.5	2.0
D	Granular	0	1.1	1.2

Note:
1) Class D bedding should only be used when suitable bedding material is not available.
2) Class A bedding should not be used unless there are special requirements to be met.
3) For zero and negative projection installations use trench bedding factors.

For positive projection conditions, where greater accuracy is required the bedding factors can be calculated using the procedure described in Section 6.

5.2. INTERNAL PRESSURE

Where a pipeline is required to work under internal pressure, two conditions must be considered:

- The static head of water in the pipe, excluding the losses due to friction.
- Dynamic factors that can cause pressure surges above and below the static or working head.

The factors to be considered are:

- Whether or not the flow can be unexpectedly stopped and if so whether the stoppage is gradual or instantaneous
- Whether the surges below the working head can give rise to negative pressures.

For pressure pipes a factor of safety of 1.5 is normally used where only the working pressure is known. Where the pressures along the pipeline have been accurately calculated, 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.

When concrete pipes are used for a pressure pipeline it is usually a gravity system or a siphon where surges cannot develop. Hence specifying a factory hydrostatic test pressure that is 1,5 times the maximum static or operating head is adequate.

Gravity pipelines such as sewers and stormwater drains that normally flow partly full, may at times be surcharged and operate under nominal pressure. The maximum pressure in a section of pipeline will then be the difference between the invert level of the downstream manhole and the ground level at the upstream manhole. Pipes used in applications such as this are unlikely to have to take a temporary pressure in excess of 9m. They are factory tested to 14m of head.

5.3. SAFETY FACTORS

The choice and application of safety factors is left to the discretion of the designer. It is suggested that either each load be considered independently and a factor of safety ranging from 1.0 to 1.5, applied directly to the value of the load, or the required pipe strength. Recommended values are given in Table 19.

The determination of a factor of safety to be used in designing a pipe depends on:

- Field working conditions
- Degree of supervision
- Height of fill above a pipe
- Whether or not there are corrosive elements in fluid transported, or the groundwater.

TABLE 19: 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

5.4. SELECTION OF THE CONCRETE PIPE CLASS

As the size of circular pipes is defined by one dimension only, the relationship between the load to be carried and the pipe strength required, is simplified. The strengths can be determined by using an indirect approach. This means that the installed loads are connected into a factory test load by using a bedding or safety factor.

5.4.1. External load

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

$$W_T = W_I FS / B_F \qquad (19)$$

Where W_T - required proof load for 0.25 mm crack
W_I - external load (kN/m)
FS - safety factor
B_F - bedding factor

The pipe class is selected so that:

$$W_T < S \qquad (20)$$

Where S - proof load of a standard D-load class pipe (kN/m), as per Table 2

5.4.2. Internal pressure

The selection of the pressure class is made as follows:

$$t = p \times FSt \qquad (21)$$

Where t - required test pressure (kPa)
p - design pressure in pipeline
FS- factor of safety, usually taken as 1.5

The pipe class is selected so that

$$t < T \tag{22}$$

Where T - test pressure of standard pressure class pipe (kPa), as per Table 3

5.4.3. Combined internal pressure and external load

Where pipes are to be subjected to combined external load and internal pressure the following formula is used for pipe selection:

$$T = t/(1 - (W_T/S)^2) \tag{23}$$

When selecting a pipe for these conditions, a balance between T and S should be found. A pipe should not be selected that is required to withstand a very high pressure and a very low vertical load or vice versa, as such a pipe would be uneconomical.

Example 3

Determine the strength of a 900mm internal diameter storm water pipe under the following conditions: Trench installation, using trench width in accordance with SANS 1200 DB. Backfill material: dry sand (w = 1 600kg/m^3). Height of fill on top of pipe: 3.5m. Traffic loading: NB 36. Bedding: Class B. Pipeline in corrosive soil conditions.

From Table 11, load due to fill = **(**1600/2000) x 101 = 80.80 kN/m and from Table 16 NB 36 loading = 4,0 kN/m

Since pipe is in corrosive conditions a safety factor of 1.3 should be applied to total load. The class B bedding factor is 2.0, therefore required minimum proof load, S will be:

$$W_T = \frac{(earth\ load + live\ load) \times SF}{B_F}$$

$$= \frac{(80.80+4)\times 1.3}{2.0} = \frac{84.80\times 1.3}{2.0} = \frac{110.24}{2.0} = 55.12\ kN/m$$

A class 75 D (67.5 kN/m proof load) will therefore be adequate. If an economic evaluation of the installation is required the pipe bedding class combinations that are adequate as well as their costs are needed so that the total cost can be calculated.

Bedding class	Bedding factor	Required Test	Required D-load	Standard D-load
C	1.5	73.49	81.7	100
B	2.0	55.12	61.24	75
A non reinforced	2.6	42.4	47.1	50
A reinforced	3.4	32.4	36.0	50

Example 4

Determination of strength of a 1 200 mm internal diameter pipe culvert to be installed under the following conditions: Embankment installation Positive projection: projection ratio p = 0.7 Foundation material: rock (r_s = 1) Height of fill above top of pipe: 2.5 m Backfill density: 1 650 kg/m^3. Light traffic conditions are expected, assume 4 000 kg maximum wheel load, Class B bedding and Non-corrosive conditions

For a value of pr_s = 0.7 x 1.0 = 0.7, Table 14 gives a backfill load = (1650/2000) x 100 = 82.5 kN/m. Loads to left of shaded area; no Pr_s adjustment needed.

and from Table 15 traffic load = 5.3 kN/m. The factor of safety is 1 (See 4.6)

The required proof load will be:

$$W_T = \frac{(earth\ load + live\ load) \times SF}{B_F} = \frac{(82.5+5.3)\times 1}{2.4} = 36.6\ kN/m$$

$$Required\ D_{load} = \frac{36.6}{1.2} = 30.5\ D$$

A class 50D pipe (60 kN/m load should be specified.)

Example 5

A 300 mm internal diameter pressure pipeline is to be installed in a trench under the following conditions: The maximum pressure expected in the line including surge and water hammer is: 150 kPa Trench width : 900 mm, Height of fill: 1.5 m, Material: wet sand (density 2 000 kg/m^3), Bedding: Class C and non-corrosive conditions

From Table 12 for trench width 900 mm and height 1.5 m; Load on pipe = 20 kN/m

Class C bedding factor = 1.5 and Factor of safety = 1.0 (See section 4.6)

Required pipe strength

(i) Because surge and water hammer have been considered a FS of 1.0 can be used. Therefore can take T ≥ k = 150 kPa

$$W_T = \left(\frac{20}{1.5}\right) \times 1 = 13.3\ kN/m$$

$T = \frac{t}{(1-(W_T/S)^2)}$ (see Par 5.4.3)

Where t = 150 kPa, W_T = 13.3 kN/m and S = 15.0 kN/m

Therefore T = 150 / (1-(13.3 / 15)^2) = 702 kPa

(ii) If this was beyond the capacity of concrete pipe that could be produced

- Should check if the earth load is correct and;
- If not will have to increase the pressure rating.
- The embankment load from Table 14 is 17 kN/m. (Trench width greater than transition width)

Required pipe strength is $W_T = \left(\frac{17}{1.5}\right) \times 1 = 11.33$

Assuming a Class 50D pipe is used (15 kN/m proof load) determine the minimum resistance to internal hydraulic pressure, by using the formula again:

$$T = 150/(1 - \left(\frac{11.33}{15.0}\right)^2) = 349$$

The pipe specification should be pressure class T4 and strength class 50D.

Alternative classes could be determined by starting with a 100D pipe (30kN/m)

$$T = \frac{150}{1-\left(\frac{11.3}{30}\right)^2} = 174.8\ kPa$$

In this design the pipe specification would be T2 (200 kPa) and Class 100D that would possibly be more economic than the first alternative, but not as economic as the second.

6. BEDDING

6.1. GENERAL

The bedding supporting a pipe transfers the vertical load on the pipe to the foundation below the bedding. It also provides a uniform support along the pipeline and prevents any load concentrations on the pipe due to irregularities in the foundation. The ability of a rigid pipe to carry field loads that are larger than the test load depends on the degree of support given to the pipe by the bedding. The ratio between the load that a pipe can support on a particular type of bedding, and the test load is called the bedding factor.

When selecting granular materials for Class B, C and D beddings the designer must consider the interface between the bedding material and the surrounding natural material. Material suitable for this type of bedding is a free draining graded gravel or aggregate consisting of rounded particles that can flow easily. However crushed aggregates containing angular particles are more stable than rounded particles and will minimise the settlement of the pipe into the bedding material. Precautions must be taken to prevent the ingress of fine material into the bedding layer, as this will result in a loss of support to the pipe and the possibility of excessive settlement. Should the material on the trench floor have inadequate or variable bearing capacity it should be reworked or may have to be replaced to ensure that it provides uniform support in a longitudinal direction along the trench.This will ensure that the pipeline has and keeps its gradient and that relative settlement is eliminated.

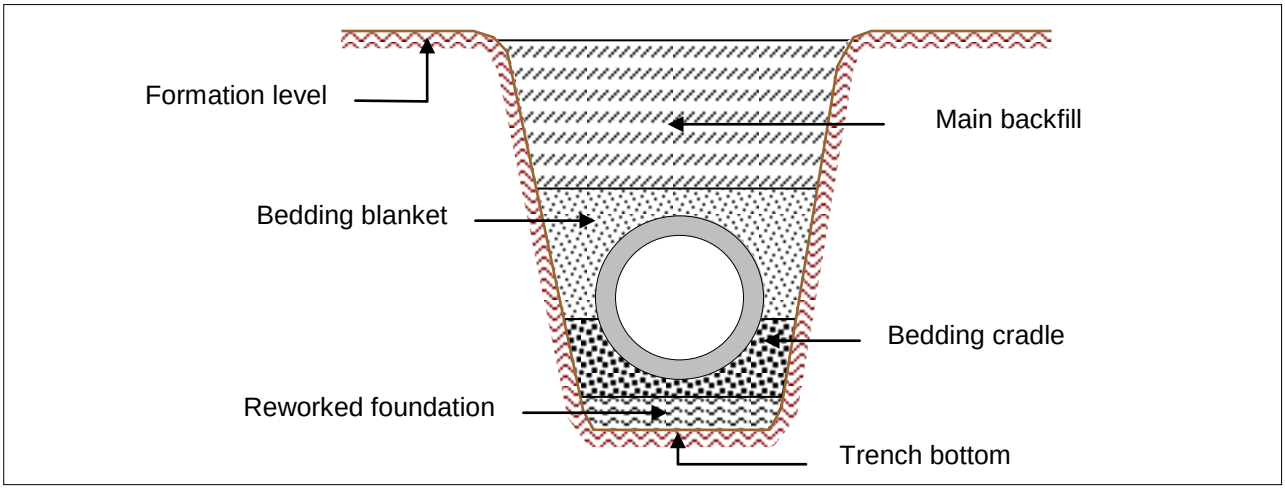


Figure 20: Terminology for Pipe Bedding

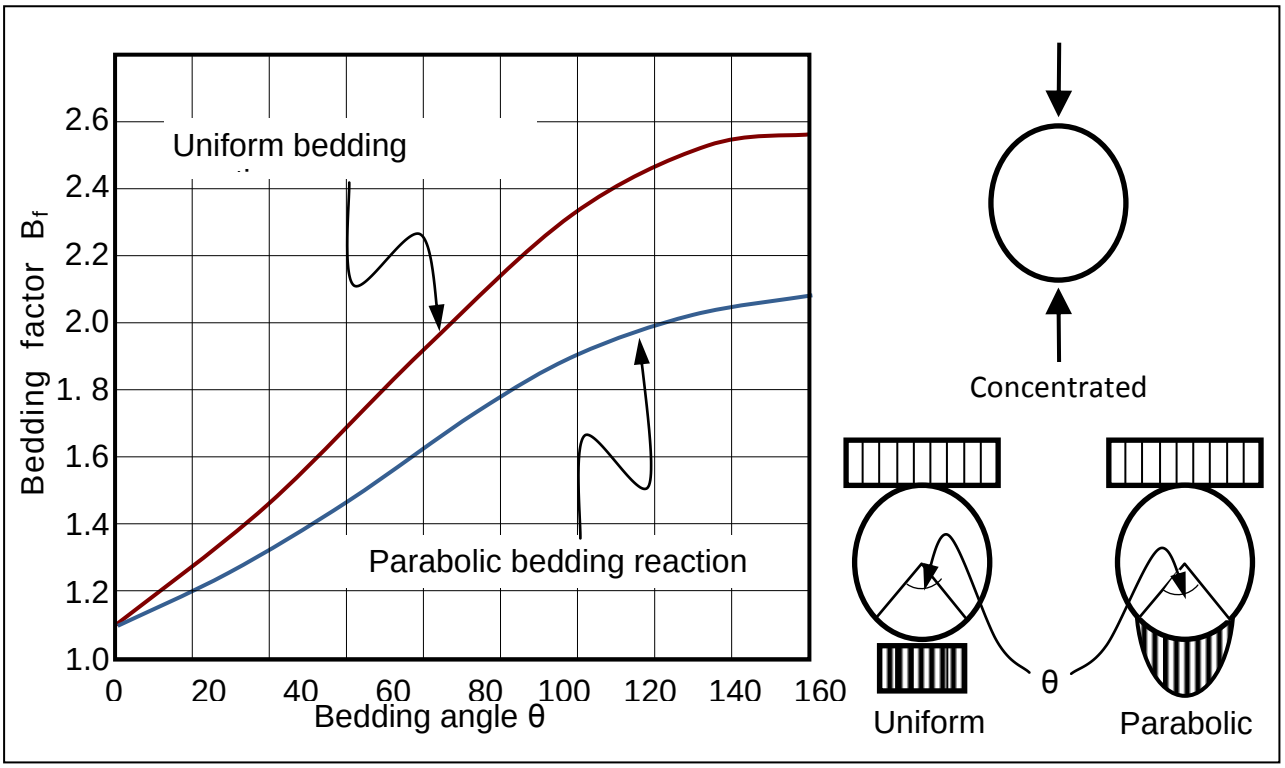


Figure 21: Relationship between Bedding Factor and Bedding Angle

6.2. TRENCH AND NEGATIVE PROJECTION INSTALLATIONS

6.2.1. General

The pipe weight and the loads on it are transferred to the foundation through the bedding. The amount the bedding yields under this load determines the pressure distribution of the reaction between bedding and pipe. For trench installations no allowance is made for lateral earth pressure. Pressures are assumed to act on the pipe in the vertical direction only.

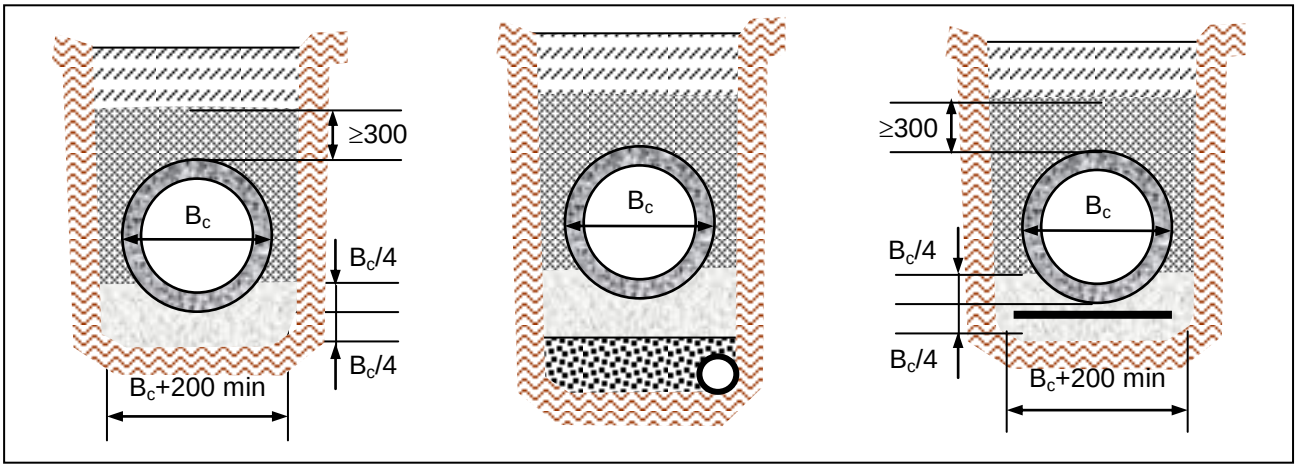
Compacted granular beddings have some flexibility and will yield more than the pipes which surround when they are loaded. The pressure distribution of the reaction from this type of bedding is parabolic. Most South African standards call for the bedding material to be compacted to 90 % mod ASSHTO. This is adequate for granular materials that are free draining with no PI, but may not be adequate for materials with a clay content. For this reason it is recommended that the material used in the bedding cradled is as described above. A rigid bedding with the same flexural stiffness as the pipe will deform the same amount as the pipe under load and the pressure distribution of reaction between pipe and bedding will be rectangular and uniform. Figure 21 gives the relationship between the bedding factor and the angle of bedding support for uniform and parabolic reactions. The maximum bending moment occurs at the invert of the pipe under these loading conditions.

In negative projection installations, where the limits are the trench condition and the zero friction condition, the development of lateral soil pressures is ignored, as it is difficult to obtain adequate compaction of the backfill in confined spaces.

Where the design corresponds to one of the bedding classes given below, the bedding factor for that class should be used. Alternatively, Figure 21 may use to obtain an appropriate bedding factor. The key to the materials used is given in Table 11 earlier in this text.

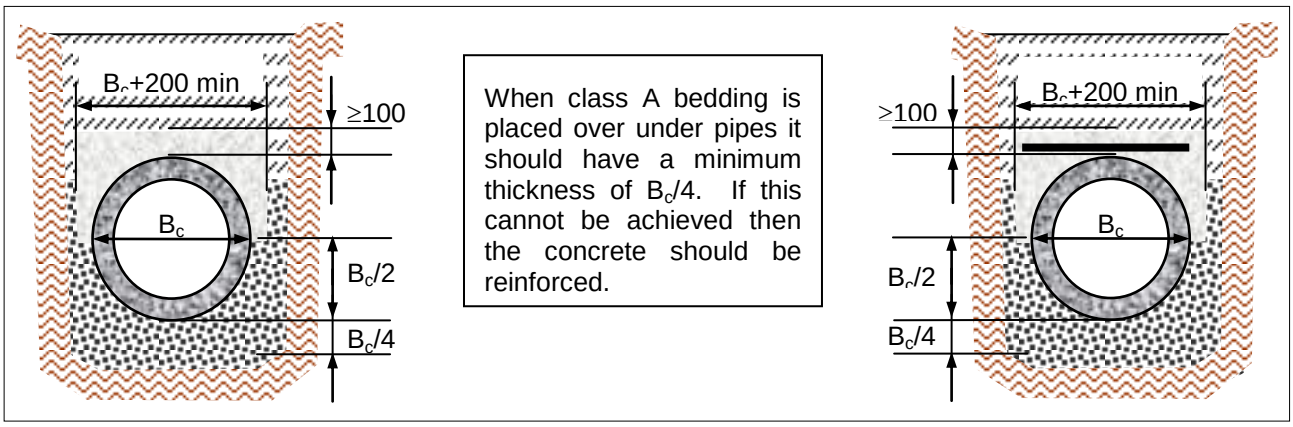
6.2.2. Class A beddings

The concrete beddings commonly used are given in Figure 22. The bedding width shall not be less than Bc + 200 mm but may extend the full width of the trench. Steel reinforcement if used must not be less than 0.4 % of the concrete cross-section and must be placed transversely beneath the pipe and as close to it as possible allowing for the minimum cover required for reinforced concrete. The concrete shall have a 28-day cube strength of not less than 20 MPa.



(a) Class A (non-reinforced) (b) For Wet Conditions (c) Class A (reinforced)

Figure 22: Class A Trench Beddings under Pipes



(a) Concrete Arch (b) Reinforced Concrete Arch

Figure 23: Class A Trench Beddings over Pipes

The class A bedding factors are:

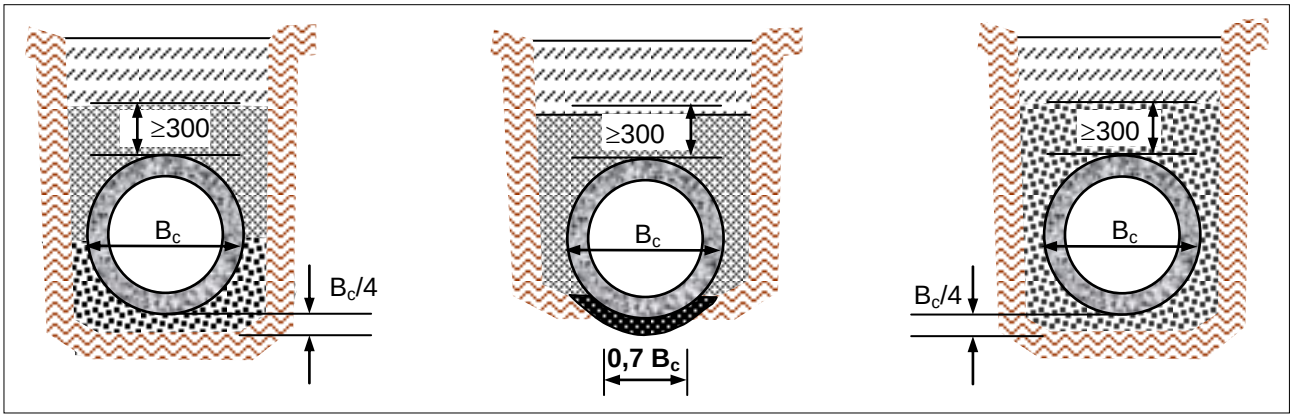
Unreinforced	0.4% Reinforcement	1.0% Reinforcement
2.6	3.4	4.8

These factors are slightly higher than the values given in Figure 21 as it is assumed that the Class A concrete bedding is stiffer than the pipe it supports. As a result the pressure under the pipe will have an inverse parabolic distribution, giving a lower bending moment at the pipe invert than the uniform distribution.

6.2.3. Class B Beddings

The Class B Granular bedding commonly used is shown in Figure 24(a). The bedding angle is 180° and the pressure distribution under the pipe is assumed to be parabolic. The selection, placement and compaction of the granular material in the cradle must be done so that this assumption is not compromised. The bedding blanket consists of a densely

compacted selected granular material, which may have some plasticity in it. (The PI should not exceed 6 unless otherwise approved by the engineer)



(a) Granular Bedding (b) Shaped Sub-grade (c) Fully Encased

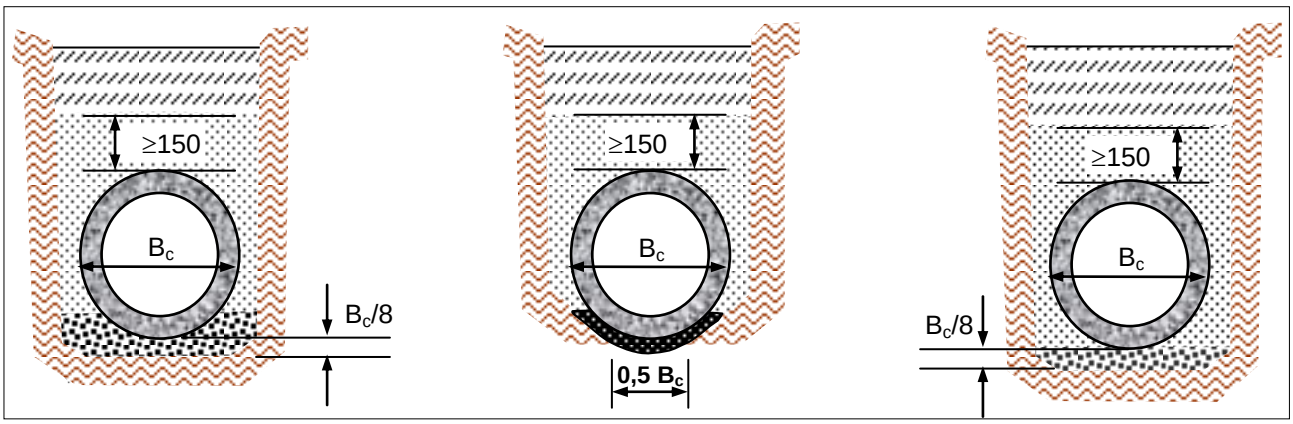
Figure 24: Class B Trench Beddings

The construction detail of the shaped sub-grade bedding with a granular cradle is shown in Figure 24(b). The width of the bedding is 0.7 Bc (90° bedding angle) and the pressure distribution under the pipe is assumed to be uniform. The depth of the fine granular blanket over the pipes must not be less than 300 mm and the side fill must be well compacted. The Class B bedding factors are:

Granular Bedding	Shaped Sub-grade	Fully Encased
2.0	2.0	2.2

6.2.4. Class C beddings

To allow for a smaller bedding angle than used with Class B beddings and the possible poorer quality of bedding cradle material and its compaction a reduced bedding factor is used. However the selection, placement and compaction of the granular materials must not be compromised. Details are given in Figure 25 below.



(a) Granular cradle (b) Shaped sub-grade (c) Selected granular

Figure 25: Class C Beddings

When a granular cradle is used the bedding angle is 90° and the pressure distribution is assumed to be parabolic. The blanket consists of a selected material that is lightly compacted and covers the pipe by at least 150 mm before the main backfill is placed.

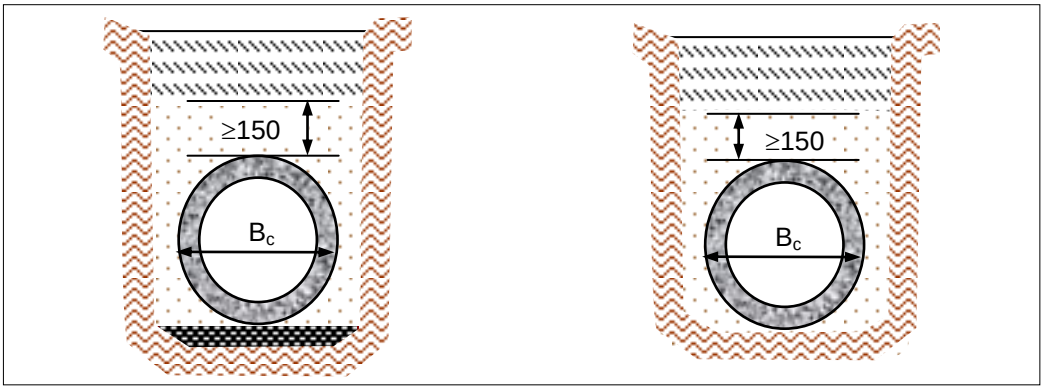
The construction detail of the shaped sub-grade bedding is shown in Figure 25(b). The bottom of the trench is compacted, levelled and shaped so as to support the pipe barrel over a width of 0.5 Bc (60° bedding angle).

The construction detail of the flat granular bedding is shown in Figure 25(c). It is assumed that when the pipe is placed that it settles into the cradle material to achieve a support angle of angle of at least 45 ° with a uniform pressure distribution under the pipe. The bedding factors for class C granular beddings are:

Granular support angle 60°	Shaped sub-grade	Uncompacted granular	Granular support angle 90°
1.5	1.5	1.5	1.7

6.2.5. Class D beddings

No special precautions are required for this class of bedding except that the sub-grade must fully support the pipe in the longitudinal direction and that holes must be excavated in the floor of the trench to accommodate sockets or joints that have a diameter greater than that of the pipe barrel. Load concentrations on the pipe must be avoided. This class of bedding is not suitable in situations where the founding conditions consist of very hard or very soft insitu material such as rock, hard gravel or soft clay.



(a) Compacted granular material (b) Flat sub-grade

Figure 26: Class D Trench Beddings

The construction detail of the flat sub-grade bedding is shown in Figure 26(a). Where the flat sub-grade surface is not suitable as bedding it should be improved by compacting and levelling a layer of suitably graded granular material. This layer will provide uniform support along the length of the pipe, without the risk of load concentrations occurring (see Figure 26(b)). The type D beddings should only be used for smaller diameter pipes where the pipe cost is much less than the total installation cost. The bedding factor for class D beddings in a trench or negative projection installation is 1.1

6.3. POSITIVE PROJECTION INSTALLATIONS

6.3.1. General

In positive projection installations, where the limits are the zero friction or geostatic condition and the complete projection condition, active lateral soil pressures develop in the fill and these help to carry the vertical load on the pipe. The bedding factors used for these installations are therefore higher than those used for trench and negative projection installations. The bedding classes are the same as those used in negative projection installations. The enhanced values of the bedding factors as given below are determined by using Spangler's method.

The comments about bedding materials given in the proceeding sections covering trench beddings are equally applicable to embankment beddings.

6.3.2. Spangler's Method

The bedding factor applicable to positive projection installations is calculated using formula below. Figures 27 and 28 show the standard embankment bedding details

$$B_f = \frac{A}{N - x \cdot q} \qquad (24)$$

where A - 1.431 for circular pipes
N - is obtained from Table 14
x - is obtained from Table 15
q - is calculated from formula below

$$q = \frac{mK}{Cc} \times \frac{H}{Bc} + \frac{m}{2} \qquad (25)$$

where q - ratio of total lateral pressure to total vertical load
K - Rankine's coefficient of active earth pressure, usually taken as 0.33
Cc - fill load coefficient for positive projection
m - proportion of Bc over which lateral pressure is effective. See Figure 27.
H - fill height over pipe

TABLE 20: VALUES OF N FOR POSITIVE PROJECTION BEDDINGS

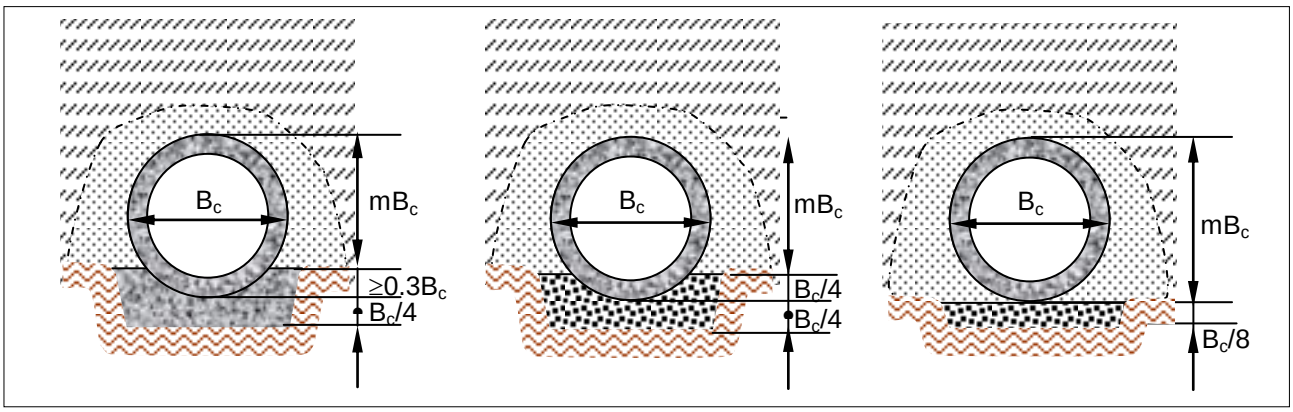
Type of bedding	Value of N
Class A – restrained	0.421
Class A – unrestrained	0.505
Class B	0.707
Class C	0.840
Class D	1.310

Note: Reinforced or plain concrete beddings cast against stable rock, are restrained

TABLE 21: VALUES OF x FOR POSITIVE PROJECTION BEDDINGS

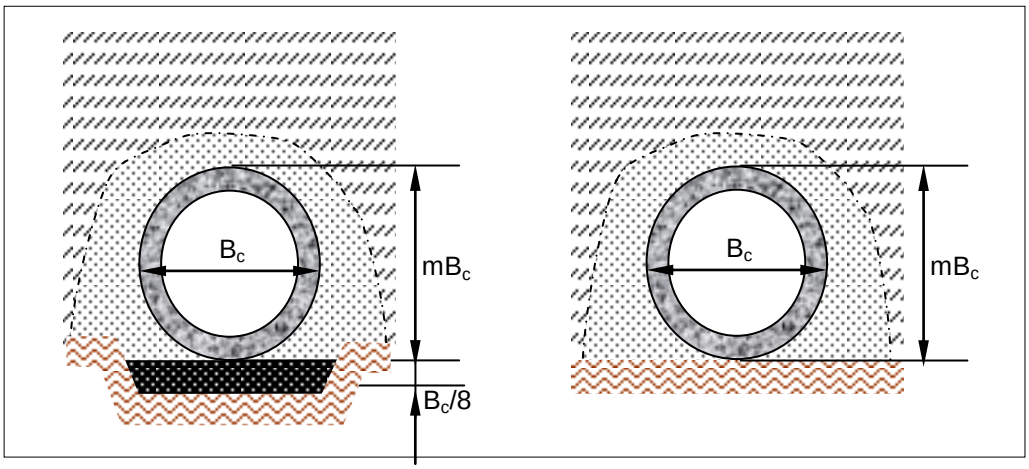
Value of m	Concrete	Other
0.0	0.150	0.000
0.3	0.743	0.217
0.5	0.856	0.423
0.7	0.811	0.594
0.9	0.678	0.655
1.0	0.638	0.638

Note: The parameter x is a function of the proportion of the pipe over which active lateral pressure is effective.



(a) Class A concrete (b) Class B granular c) Class C granular

Figure 27: Embankment Class A, B and C Beddings



(a) Class D granular (b) Class D natural material

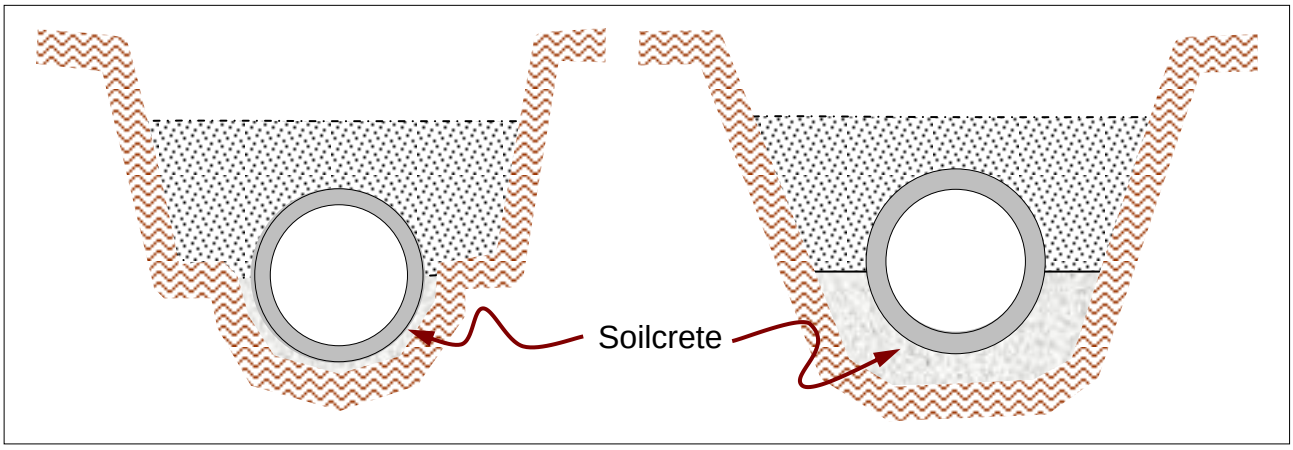
Figure 28: Embankment Class D Beddings

6.4. SOILCRETE BEDDING

Soilcrete or soil-cement as it is sometimes called is an alternative bedding material that is used under certain circumstances such as when there are:

- concerns about bedding material washing away and causing piping next to pipeline
- time restraints on the installation
- trenches that are narrow and side compaction is difficult.

Soilcrete consists of a granular material that has between 3% and 6% of cement added to it and is made as a flowable mix with a slump of >200mm. There should be no organic material in the soil used and ideally the clay content should be minimal. The soilcrete is stronger that soil, having a strength between 0.5 and 1.0 MPa. This material can be used in two ways, namely as a gap filler or as a bedding as illustrated in Figures 29 (a) and (b).



(a) Soilcrete as gap filler (b) Soilcrete as bedding

Figure 29: Use of Soilcrete around Pipes

The purpose of the Soilcrete is to transfer the load on the pipe to the surrounding soil. As it is stronger than soil it does not matter if there are small cracks in it. The important issue is that the material is stable and supports the pipes. To ensure that there is support all around the pipe this material needs to be flowable and vibrated once placed. To prevent floatation the soilcrete is placed in two stages, the first should not be higher than ± a sixth of the pipe OD. The second stage can be placed as soon as the initial set has taken place. (When a man can walk on it.) for installation details reference should be made to the Installation Manual that is a companion publication to this one.

When soilcrete is used as a gap filler the distance between the pipe and excavated material should be ± 75 mm. When it is used as bedding the dimensions should be the same as those used for concrete bedding. The bedding factors for soilcrete beddings will depend on the bedding angle and can be taken from the curve for concrete on Figure 21.

6.5. JACKING CONDITIONS

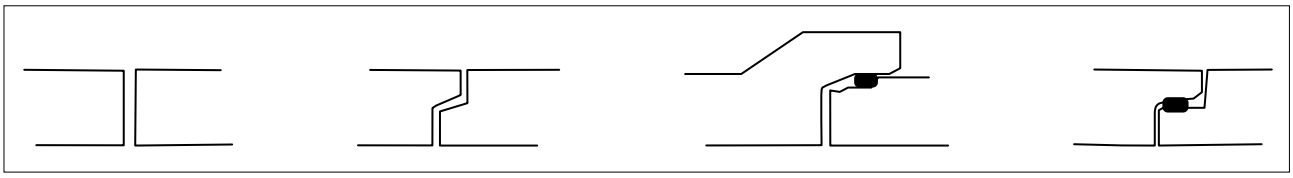
When the pipes are jacked the excavation is slightly larger than the external diameter of the pipe. However, the process of installing a pipe ensures that positive contact is obtained around the bottom portion of the pipe and that ideal bedding conditions are obtained. If the pipe carries all or part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

When determining the bedding factor, the behaviour of the insitu material after the jacking is completed and the post installation treatment given to the void between the pipe and the excavation should be considered. If this is grouted, a value of 3 can be used.

7. PIPE JOINTING

7.1. JOINT TYPES

The function of the joint is to provide flexibility and sealing for the pipeline. Joints are designed to cope with the movement that occur due to the secondary forces within the soil mass. There are four types of pipe joints, namely, butt (or plain ended), interlocking (or Ogee), spigot and socket and in-the-wall joints. These are used for different applications that are determined by the amount of movement to be tolerated and the importance of



keeping the pipeline sealed.

(a) Butt (b) Interlocking (c) Spigot and socket (d) In-the-wall

Figure 30: Joint Types for Concrete Pipe

7.2. BUTT AND INTERLOCKING JOINT PIPES

Butt ended and interlocking pipe joints are not intended to prevent infiltration and exfiltration of water hence they are only used for stormwater drainage and culvert pipe. Butt ended pipes are seldom used as they do not have any means of self-centering when being jointed.

If there is a potential problem with the loss of bedding material into the drainage system the joints should be sealed either with mortar or sealing tape. When stormwater drains are placed on steep slopes and the flow velocity exceeds 4 or 5 m/s it is advisable to use one of the joints that can be sealed with a rubber ring to prevent the high velocity water going through the joints and scouring cavities in the soil around the pipes.

Pipes for sewers or pressure pipelines should have spigot and socket or in-the-wall joints that include seal in the form of either a rolling ring or confined ring.

7.3. SPIGOT AND SOCKET JOINTS

Pipes with this joint type are the most commonly used for sewers. They are designed to seal as well as tolerate movements in three directions, namely:

- Draw or longitudinal movement.
- Deflection or radial movement.
- Relative settlement or displacement of a pipe relative to the adjacent ones.

In addition to this these joints take into consideration tolerances on concrete surfaces, laying procedures and seal dimensions. The rubber ring enables this type of joint to be deflected as shown in Figure 31 so that pipes can be laid around curves and still remain watertight.

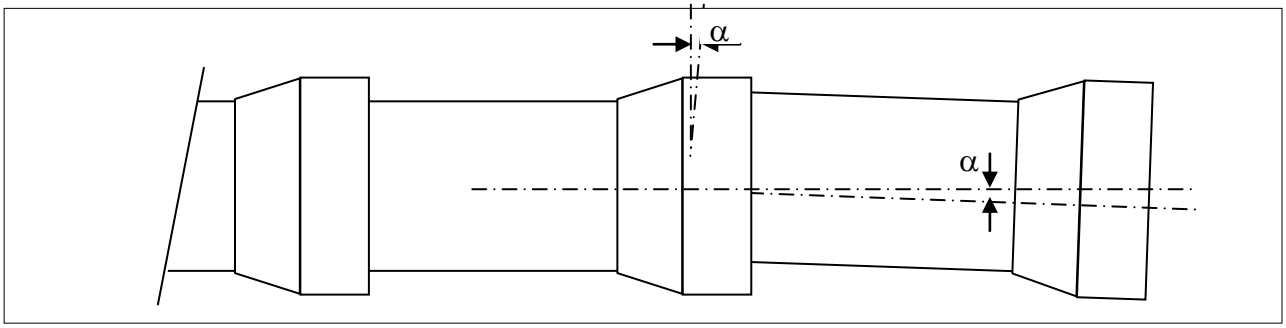


Figure 31: Angular Deflection of Spigot and Socket Pipes

The amount of movement that can be tolerated at a joint will depend on the pipe size and the manufacturer's details. The radius of the curve is dependent on the angular deflection that is permitted for each pipe size. Typical deflections and curve radii are given in Table 29. Specific projects should be discussed with the manufacturer concerned.

TABLE 22: ANGULAR DEFLECTIONS AND CURVE RADII

Nominal Pipe Diameter - mm	Permissible Degrees	Minimum Radius - m
300 - 375	2.00	70
450 - 600	1.50	93
675 - 900	1.00	140
1 050 - 1 200	0.75	186
1 350 – 1 800	0.50	280

The radius of curve that can be negotiated is directly proportional to the pipes' effective length. The values in this table were calculated using an effective pipe length of 2.44m. If a different length is used the radius from the table should be corrected by the ratio of the lengths. Where sharp curves in excess of these values are required special pipes with deflected spigots or sockets, or radius pipe can be produced. This should be discussed with the manufacturers.

When a curve is being negotiated, the pipes must first be fully jointed in a straight line and only then deflected.

The spigot and socket pipe has traditionally been made with a rolling rubber ring. The South African standard for rubber rings is SANS 974-1: *Rubber joint rings (non-cellular) Part 1: Joint rings for use in water, sewer and drainage systems*.

7.4. IN-THE-WALL JOINTS

With large diameter pipes the wall is so thick that a rubber ring joint can be accommodated within the wall thickness. With this type of joint nibs on the jointing surfaces or a groove in the spigot confine the seal. As the seal remains in a fixed position the socket slides over this so both the seal and the socket of the pipe to be jointed should be should be thoroughly lubricated before the joint is made. This type of joint is sometimes called a confined or sliding rubber ring joint. Particular attention should be paid to lubricating the lead-in section of the socket that makes the first contact with the seal.

The advantage of this type of joint is that the outside diameter of the pipe remains constant making the pipe ideally suited for jacking. For jacking pipes from 900 mm in diameter and larger use this joint type. However for sewers this type of joint is seldom used for pipes of less than 1500 mm in diameter. Most pipes larger than 1800 mm in diameter are made with this type of joint.

The joint is designed to cope with the same criteria as the spigot and socket joint, but as it is shorter than the spigot and socket joint the amount of movement that it can tolerate will in general be a little less. These joints can, in general, cope with a deflection of 0.5 degrees and be used to negotiate curves if required to do so.

For details of how pipes should be jointed the reader is again referred to the *Concrete Pipe and Portal Culvert Installation Manual* or the pipe supplier.

8. FLOATATION

8.1. GENERAL

Any buried pipeline, even when full of water will weigh less than the soil that it displaced. Hence there will be a tendency for pipelines to lift rather than settle. When the groundwater level is higher than the bottom of the pipeline the buoyancy forces can lift the pipeline due to. If these conditions can occur either during the installation or operation of the pipeline the designer should check that the pipeline will not float off its bedding.

SANS 10102 Part II (4, p51) lists several conditions that could give rise to this, namely:

- Flooding of trench to consolidate backfill
- Pipelines in flood plains or under man-made lakes that will be below groundwater level
- Sub aqueous pipelines
- Pipelines in other areas that may be subject to a high water table

If any of these exist the designer should calculate the forces to establish whether or not floatation will be a problem. These forces are:

- Weight of pipe
- Weight of water displaced by pipe
- Weight of load carried in pipe
- Weight of any backfill over the pipe

Two floatation conditions can occur, namely:

- Pipeline is submerged partly or fully before backfilling
- Pipeline becomes submerged after backfilling

8.2. FLOATATION BEFORE BACKFILLING

The weight of the displaced water in kN/m of pipeline, w_w is calculated from:

$$W_w = \gamma_w L_1 A_1 \tag{26}$$

Where γ_w - density of water in kN/m^3

L_1 - length of pipeline in m

A_1 – cross-sectional area of pipeline below water surface in m^2

The pipeline will float if

$$w_w > w_p \tag{27}$$

Where w_p - the pipeline mass in kN/m

8.3. FLOATATION AFTER BACKFILLING

The vertical soil load acting on the pipeline in kN/m of length, w_b can be calculated from:

$$w_b = \gamma' B_c H \tag{28}$$

Where γ' – submerged density of saturated backfill in kN/m^3 see formula (29) below

B_c - outside diameter of pipeline in m

H – fill height over pipeline in m

$$\gamma' = \gamma_w - \frac{G_1 - 1}{1 + E} \tag{29}$$

Where G_1 – specific gravity of soil particles

e – void ratio of soil

The pipeline will float if

$$w_w > w_p + w_b \tag{30}$$

9. SEWER CORROSION

9.1. CORROSION MECHANISM

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 physical appearance of corrosion is

first detected as a white efflorescence above the water line, and it takes several months before this starts. Thereafter deterioration may be rapid in which case the concrete surface becomes soft and putty-like and there is aggregate fallout.

There are three sets of factors contributing to this phenomenon, those resulting in the generation of the gas hydrogen sulphide (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 32 below.

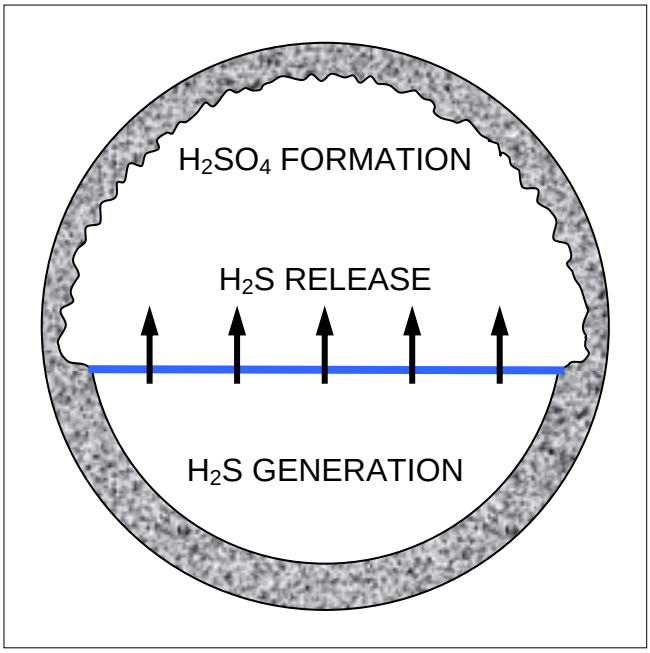


Figure 32: Corrosion Mechanism

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
- Rate of acid formation
- Amount of moisture on sewer walls
- Rate of acid runoff

If there is insufficient oxygen in the effluent 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 influence 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 so that there is equilibrium. The second set of factors influence 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. This is influenced by the third set of factors.

The acid formed then attacks the cement in the concrete above the water line, as it is alkaline. 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 is rapid. If concrete is made using a calcareous aggregate, which is alkaline, 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.

9.2. CORROSION PREDICTION AND CONTROL

Research by Pomeroy and Kienow [8] led to the development of 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"[9]. This quantified the LFM [10] by giving equations for predicting the 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 additional cover to reinforcement, referred to as "sacrificial layer" in South Africa, for a concrete pipe to ensure that it will remain serviceable for its design life.

The theoretical prediction of H_2S generation in the sewerage is based on an analysis of the effluent and is beyond the scope of this document. If the reader requires the procedure reference should be made to reference 10. Once the DS in the effluent has been determined the rate of H_2S release from effluent, called the H_2S flux can be calculated from:

$$\emptyset_{sf} = 0{,}69(sv)^{3/8}J[DS] \qquad (31)$$

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 as 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 from a modification of the above equation:

$$\emptyset_{sw} = 0{,}69(sv)^{\frac{3}{8}}J[DS](B/P') \qquad (32)$$

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

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

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

The concrete corrosion rate can be estimated by calculating the rate at which the H_2S flux to the pipe wall will be oxidised to H_2SO_4. "34g of H_2S are required to produce sufficient

H_2SO_4 to neutralise 100g of alkalinity expressed as calcium carbonate ($CaCO_3$) equivalent. (3p23)

If all the ϕ_{sw} is oxidised the annual corrosion rate for the concrete can be predicted from:

$$Cavg = (11{,}5k/A)\emptyset_{sw} \quad (33)$$

Where Cavg - average corrosion rate, mm/year

K - efficiency coefficient for acid reaction based on the 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 the cement-bonded material expressed as its calcium 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 combined with the equation for the flux of H_2S to the wall of a pipe [11] the LFM equation is expressed as:

$$Az = 11.5\ k\ \emptyset_{sw} L \quad (34)$$

Where z - additional concrete cover, over reinforcement, mm (sacrificial layer)

L - required design life of sewer in years

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

- preventing acid formation
- modifying concrete
- protecting concrete.

Acid formation can be prevented or minimized by adjusting the hydraulic design of the sewer. However, due to physical constraints this is not always possible and some corrosion can be anticipated. For most sewers modifying the concrete by changing the concrete components and/or providing additional cover to reinforcement is the most cost effective option. Protecting concrete by using an inert lining or coating is effective, but only economically justified when severe corrosion is predicted.

9.3. DEVELOPMENTS IN SOUTH AFRICA

Since the 1960's most concrete sewers in South Africa have been made using calcareous aggregates, usually dolomitic and the principle of a sacrificial layer. This solution followed the recommendations of a 1959 CSIR publication[3]. The experience in Johannesburg where the first calcareous pipes were laid in 1960 and other cities and towns in South Africa indicates that this results in a considerable increase in a sewer's life. As far as can be established, there have been no reports of serious problems on sewers made using this approach.

However, concern had been expressed by local authorities and consultants that the "dolomitic aggregate" solution might not be adequate for certain sewers in the long-term. This concern was substantiated by application of the Life Factor Model (LFM)[4], developed in the USA by Pomeroy and Parkhurst that quantifies the biogenic corrosion of concrete, to several sewers where severe corrosion was anticipated.

When the CSIR undertook a literature search during the 1980s, no reference could be found to field trials established to calibrate the actual performance of various sewer materials. [5] 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 to monitoring material performance and conditions in this sewer.

- Phase One was undertaken by the CSIR to monitor the conditions in the sewer and the performance of traditional sewer pipe materials in the sewer and subject to pure acid attack in a laboratory.
- Phase Two was undertaken by the University of Cape Town (UCT) to continue monitoring the conditions in the sewer and to investigate ways of simulating these conditions in a laboratory.
- Phase Three is being undertaken jointly by UCT and an independent consultant as a continuation of the second phase and involves measuring the actual corrosion that occurred during the first two phases; 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 has been rehabilitated and the actual corrosion on the samples installed during phase one has been determine and is summarised in Table 25 below. From this table it can be seen that measured average corrosion rates after 14 years for all the materials was somewhat greater than the estimates made following earlier inspections. As these measurements were taken on the samples removed from the sewer and the actual wall thicknesses could be measured, giving greater accuracy.

To date the LFM has been applied to PC concretes only. The corrosion rates measured in this experimental section of sewer mean that the LFM can now be applied where other concretes are used. The effective alkalinity of alternative concretes can now be allocated values in excess of unity. In particular the effective alkalinity of an inert material can be taken as infinity. The LFM can now be used to calculate the required sacrificial layer thickness by incorporating a material factor, M_F that is the ratio of corrosion rate for the alternative material being considered and a standard concrete made from PC and siliceous aggregate.

TABLE 23: MEASURED & ESTIMATED CORROSION AND MATERIAL FACTORS

Material (cement/ Aggregate)	5 year estimate		12 year estimate		14 year measured		Material
	Total	Ave	Total	Ave	Total	Ave	factor***
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	0.400
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			0.320
CAC/DOL *	3,0	0,6	7,2	0,6	8,4	0,6	0.085
CAC/ALAG™ **							0.025

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

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

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

By applying the LFM as described in equation 4 above to a particular sewer and assuming an 'A' value of 0.16 as would be appropriate for a standard siliceous aggregate concrete the required sacrificial layer can be established. The sacrificial layer thickness for another material can be calculated by multiplying this value by the appropriate material factor, M_F from Table 23.

$$z_{ANOTHER} = 72\ M_F k \emptyset_{SW} L \qquad (35)$$

Where M_F - Material Factor for chosen material and is obtained from Table 23.

This extension to the LFM has been called the Material Factor Model (MFM). The application of this and how it can be used to determine the most cost effective pipe material for a given sewer is described in section 9.5.

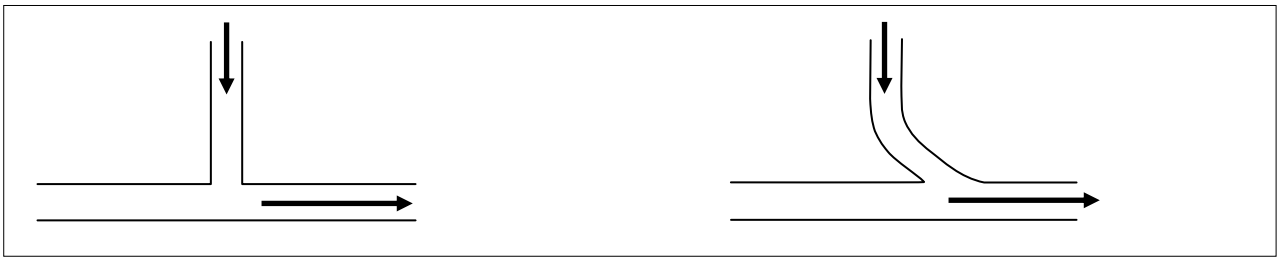
Based on the 5 year findings from the Virginia Sewer the concept of making a host pipe of one type of concrete to provide the strength and an additional layer of another concrete to cope with the corrosion was investigated. A effective technique for doing this was developed and since 1997has been used on many of the major outfall sewers in South Africa. The most commonly used combination of materials has been a host pipe made of PC/SIL concrete and a sacrificial layer of CAC/DOL. When such pipes are made an allowance of 3 to 5 mm is made for the interface between the two concretes.

9.4. DESIGN AND DETAIL CONSIDERATIONS

There are many publications on sewer corrosion. However few have been written in a South African context and to date as far as can be established none have quantified the corrosion rates of non-PC concretes as described above. These do however address the issues of the hydraulic design and detailing of sewers. The following points should be considered when designing and detailing a sewer:

- The longer the sewage stays in the sewer the greater are the chances that it will turn septic and the rate of sulphide generation increase. Where practical retention times should be kept to a minimum.
- 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. Flow velocities at minimum discharge should be at least 0.8m/s
- Moisture condensation on sewer or manhole surfaces provides the habitat for the bacteria that produce H_2SO_4. Taking steps to reduce moisture condensation are not always possible.
- Junctions between sewers with different velocities can obstruct in the sewers with the slower flows causing long retention times in them. When sewers are joined the upstream gradients should be adjusted so that the entry velocities are as close as possible to each other.
- Most junctions are affected at manholes. Energy losses and turbulence are associated with the release of H_2S and the possibility of local corrosion. The inverts of such manholes should be carefully benched with smooth transitions to minimise energy losses and turbulence.
- If a fast flowing lateral discharges into a manhole benching as shown in Figure 33(a) and at the same invert level as that of the collector, the flow in the collector can 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 33(b).



(a) Sharp junction (b) Transitioned junction

Figure 33:Connection Between Collector and Lateral.

The rate of H_2S generation in rising mains and siphons is much greater than 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 sewers downstream of these especially when sewage retention times exceed much more than an hour. When the sewage discharges into the gravity section of sewer the accumulated H_2S is liberated and can cause severe local corrosion. Procedures for minimising retention times and the resultant corrosion are:

- Use the smallest practical pipe diameter for the full flowing section of sewer
- Make the section as short as possible
- Operate pumps frequently, particularly in 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 higher sulphide content and this could result in the corrosion of the structures at the purification works. Various measures that can be taken to reduce this are:

- 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 the addition of hydrated lime to increase the sewage pH, or alternatively ventilating the outfall using a forced draught should be considered.

Careful hydraulic design and attention to detail has a positive contribution in reducing 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 and MFM when designing and detailing sewers; not as a substitute an analysis.

9.5. PIPE MATERIAL CHOICE FOR SEWERS

There are several concrete pipe alternatives that could be used for sewers. These are:

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

The relative corrosion rates of these sacrificial layer materials are given in Table 1above. By applying the LFM and the MFM as described above a technically sound solution that is also the most cost effective alternative for a sewer operating under a particular set of circumstances can be selected.

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 of velocity namely:

- A minimum value (0,7m/s) at low flow that will ensure self-cleansing.
- A maximum value of 0,8 times the critical velocity to prevent excessive turbulence.

The internal diameter (ID) and the 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 assuming a PC/SIL concrete. The relative corrosion rates of other types of concrete being considered for the project should then be calculated based on the details given in Table 25 above. The sacrificial layer thickness with an appropriate allowance for an interface if the sacrificial layer and host pipe are made from different concretes should be added to the required internal diameter to give the host pipe internal diameter.

From the installation conditions do a preliminary assessment of the pipe class that will be required based on the worst-case scenario as given in table xy above. If the pipe class indicated were 75D or 100D then the outside diameter (OD) of the pipe would be 1.2 times the indicated host pipe ID. If the pipe class indicated was 50D or less then the outside diameter would be 1.14 times the host pipe ID.

The manufacturers brochures should be consulted to determine the nearest actual external diameter that would give at least the external diameter as indicated by the calculations done following the above procedure. 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 OD's and this could mean that the pipes using a different corrosion control measures would be made in moulds of different OD's. This is illustrated in the example that follows.

Once the mould OD's for the different solutions have been established the exercise should be repeated for each of these alternatives but in the reverse order namely:

- For the required OD determine the pipe strength and class required to handle the installed conditions.
- Add the required sacrificial layer or lining thickness to the host pipe ID to determine the actual pipe ID.
- Check the hydraulics of the sewer using the actual ID.

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

Example 6: Determine most cost effective pipe with an actual ID of 900mm for a range of Az values, namely 5, 10, 20 and 40. Assume required pipe class is 100D.

TABLE 24: CORROSION ANALYSIS USING PC/SIL, PC/DOL AND CAC/DOL

MATERIAL	PC/SIL			PC/DOL			CAC/DOL		
Az VALUE	5	10	20	5	10	20	5	10	20
PIPE ID-	900	900	900	900	900	900	900	900	900
SACR L	30	60	125	12	24	50	5	9	15
HOST ID	960	1020	1150	924	948	1000	910	918	918
PIPE OD	1152	1224	1380	1109	1138	1200	1092	1102	1116
HOST - kg	822	928	1179	757	803	892	738	753	771
SACR L - kg	226	467	1038	89	178	385	37	66	111
TOTAL - kg	1046	1395	2217	845	981	1277	775	819	882
% HOST PRICE	145	193	307	117	136	177	123	132	153

TABLE 25: CORROSION ANALYSIS USING PC/DOL, CAC/DOL AND HDPE

MATERIAL	PC/DOL				CAC/DOL				HDPE
Az VALUE	5	10	20	40	5	10	20	40	all
PIPE ID-	900	900	900	900	900	900	900	900	900
SACR L	12	24	50	100	5	9	15	25	0
HOST ID	1100	910	1000	1100	930	950	950	950	905
PIPE OD	1320	1116	1200	1320	1140	1102	1116	1140	905
HOST - kg	757	803	892	1079	738	753	771	805	787
SACR L - kg	89	178	385	811	37	66	111	187	0
TOTAL - kg	845	981	1277	1889	775	819	882	992	0
% HOST PRICE	117	136	177	262	123	132	153	189	178

This table illustrates the impact of the corrosion potential on the cost effectiveness of the various materials commonly used as corrosion control measures for sewers in South Africa. As the corrosion potential increases the solutions that are more costly to produce actually become more cost effective. The following shows this:

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

It should be noted that the costs used in this exercise are hypothetical and that do make this comparison on an actual project it would be necessary to obtain actual prices from the pipe suppliers.

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

From the above example it can be seen that all sewer pipes and manholes should be manufactured using calcareous aggregates even if no corrosion is expected. The concrete made for these should contain not more than 25% insolubles when tested in hydrochloric acid. (Details of the test method are given in SANS 676.) In some parts of South Africa the aggregates available have insolubility levels of 12% to 18%. If available the lowest practical level should be specified.

The standard sacrificial layer thicknesses used in South Africa are 13 mm for pipes up to 1050 mm in diameter and 19 mm for diameters larger than this. 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 sacrificial layer and host pipe are made from different concretes an allowance should be made for the interface between the two concretes. Under these circumstances it would be realistic to consider the design values for the standard sacrificial layers as being minimum values of 10 and 15 mm instead of nominal values of 13 and 19 mm.

9.6. SACRIFICIAL THICKNESS AND ALLOWABLE CRACK WIDTHS

Where a increased sacrificial layer thickness is specified the allowable crack width should be increased in proportion to the increase in concrete cover to steel. The allowable crack width can be calculated from the formula (36) given in SANS 677.

$$r = q(t - x)/(t - x - C_2) \qquad (36)$$

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

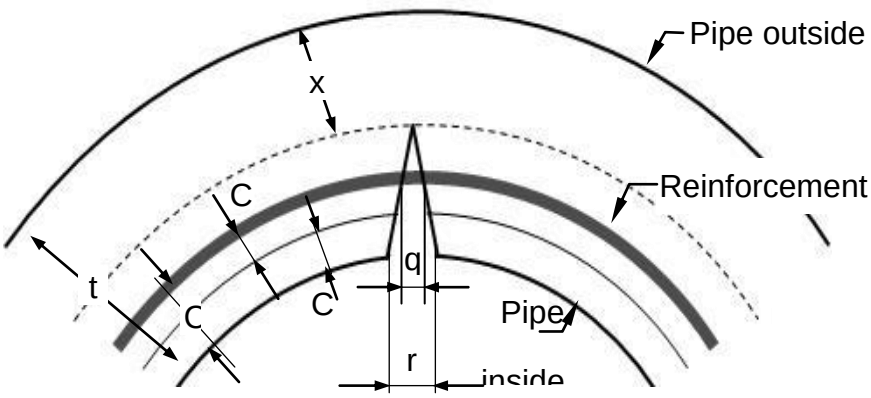
x – distance from the outside surface of pipe to the neutral axis in mm

$$C_2 = C - C_1 \qquad (37)$$

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

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

The neutral axis of the pipe can be taken as being half the host pipe wall thickness. The relationship between these symbols is shown in Figure 34 below.



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

Figure 34: Relationship between Crack Width and Sacrificial Layer

There are two practical factors that should be considered when sacrificial layers that are thicker than standard ones are specified, namely:

- If the sacrificial layer is thicker than one third of the wall thickness the reinforcement will be close to the centre of the pipe wall and will not be effective in controlling cracks.
- If the sacrificial layer thickness is more than twice the standard concrete cover to reinforcement the crack widths that could be accepted if equation (6) were blindly applied could be excessive and allow aggressive elements to enter the cracks and move the corrosion front closer to the reinforcement.

The ACPA Concrete Pipe Handbook (4,p57) states that problems have not been experienced with pipes that have cracks in them of up to 0.5 mm when the concrete cover to reinforcement is 25 mm. As this cannot be substantiated by any scientific study it is recommended that the serviceability limit for crack widths be limited 0.4 mm even if equation (6) above indicates a larger value.

By applying the correct procedure for predicting corrosion and then choosing the pipe material that cost effectively meets the requirements the above problems will in general be avoided.

10. PORTAL CULVERTS

10.1. GENERAL

The terms used with portal culvert installations are detailed in Figure 35 below.

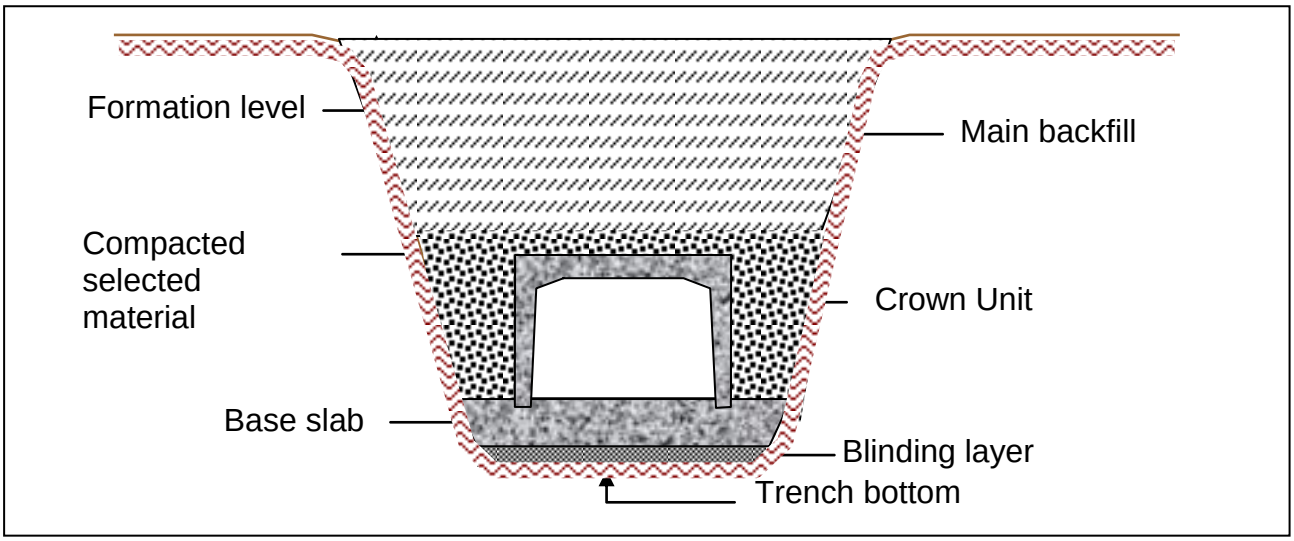


Figure 35: Terminology for Portal Culverts

As portal culverts are rectangular two dimensions determine their size. Hence, the relationship between the load to be carried and the required strength cannot be simplified as it can with pipes. Hence, the strength required is determined by using a direct approach. The procedure adopted is:

- Determine the structural properties of the portal
- Calculate loads and load combinations
- Calculate the bending moments and shear forces generated in the portal by the various load combinations
- Determine the bending moment and shear force envelopes that cover all the loading cases
- Determine combinations of test loads to model the installed bending moment and shear force envelopes.

This procedure can be followed by using ultimate values for both the installed and test loading conditions or by factoring the installed parameters and determining the proof load parameters that match them.

10.2. DETERMINING PORTAL CULVERT STRENGTHS

As mentioned at the beginning of this handbook, there is no National Standard for determining the loads on or Strengths of Portal Culverts. In TMH7, the Code of Practice for the Design of Highway Bridges and Culverts in South Africa, Clause 2.3.3.1 provision has been made for three solution levels, namely:

"General: With due recognition of the complexity of the problem of determining loading on culverts, but also of the need for simple procedures which can be used routinely, provision is made for the three-fold approach, viz:

- *The application of simple design rules that can be applied to rigid and flexible culverts but that require the use of increased partial safety factors which allow for the approximate nature of the formulae used.*
- *The application of more sophisticated design theories to rigid and flexible culverts that take into account the type of culvert, the properties of the undisturbed ground and the fill materials as well as the effects of the actual width of excavation, and the positive or negative projection. These theories also allow for the use of reduced partial safety factors. (In positive and negative projecting culverts, the tops of the structures are above and below undisturbed ground level respectively.)*

- *The application of sophisticated design theories or the design techniques based on the phenomenological approach to flexible and special types of culvert that required more accurate assessments of soil-structure interaction.*

This Code covers the first approach only, which is an extension of the AASHO[1] and CPA[2] formulae. The designer shall use his discretion in deciding on the best applicable method for any particular case and is referred to publications on the subject."

In the simplified approach the earth loading has been reduced to four combinations of foundation and installation conditions, namely:

Condition 1: Culverts in trench on unyielding foundation with no projection.

Condition 2: Culverts untrenched on yielding foundation.

Condition 3: Culverts untrenched on unyielding foundation for H>1.7B

Condition 4: Culverts untrenched on unyielding foundation for H≤1.7B

Where H - fill height in metres

B - if trenched overall trench width, or if untrenched overall culvert width, in metres.

Conditions 1 and 2 correspond to the geostatic loading condition and 3 and 4 to the positive projection installation condition with an r_{sd} p ratio of 1.

Approximate methods for determining the effects of traffic loading on rigid conduits are given in Clause 2.6.6 of TMH7.

This combination of the earth and traffic loading was applied to the standard portal culvert dimensions to determine the product strengths required. These strengths were compared with those of the standard S-load culverts and the appropriate classes selected. (Refer to Tables 4 and 5 of this document.)

The relationship between standard portal culvert classes and maximum fill heights for TMH7 loading conditions applied to the standard sizes is given in Table 29 below.

The assumptions, and clauses from TMH7 Parts 1 and 2 used to compile this table are:

- The table is applicable to rectangular portal culverts only
- When sizes other than given in this table the manufacturer should be contacted.
- A minimum fill height of 300 mm over the culvert units. Where this cannot be achieved a 100 mm reinforced concrete slab must be used.
- Standard traffic loading (SN A, B and C) as described in Clause 2.6.1.2
- Fill material unit weight 20 kN/m^3 [Clause 2.3.1]
- Concrete unit weight 24 kN/m^3 [Clause 2.2.1]
- Horizontal earth pressure 7,8 kN/m^2 per metre depth [Clause 2.4.2]
- Ultimate Limit State load factors Table 7.

If portal culverts are required where the fill over them is less than 300 mm or more than the amount stated in this table the loads must be calculated using the procedures in TMH7 and the strength by following the procedure given at the end of section 10.1 above.

TABLE 26: MAXIMUM FILLS: S-LOAD PORTAL CULVERTS UNDER TMH7 LOADING

Span x height	Portal	Conditions		Span x height	Portal	Conditions	
mm x mm	Class	1 & 2	3 & 4	mm x mm	class	1 & 2	3 & 4
600 x 300	200 S	10.2	5.6	2100 x 600	75 S	3.3	2.4
600 x 450		11.0	6.0	2100 x 900		3.5	2.6
600 x 600		12.0	6.4	2100 x 1200		3.7	2.7
750 x 300	175 S	8.7	4.9	2100 x 1500		3.9	2.8
750 x 450		9.2	5.2	2100 x 1800		4.1	2.9
750 x 600		10.0	5.5	2100 x 2100		4.3	3.0
750 x 750		10.5	5.8	2400 x 600	75 S	3.2	2.4
900 x 300	175 S	8.6	4.8	2400 x 900		3.4	2.5
900 x 450		9.0	5.1	2400 x 1200		3.5	3.5
900 x 600		9.5	5.3	2400 x 1500		3.7	3.7
900 x 750		10.0	5.6	2400 x 1800		3.8	3.8
900 x 900		10.2	5.8	2400 x 2400		4.0	4.1
1200 x 300	150 S	7.1	4.1	3000 x 900	75 S	3.2	2.5
1200 x 450		7.4	4.3	3000 x 1200		3.3	2.6
1200 x 600		7.7	4.4	3000 x 1500		3.4	2.7
1200 x 900		8.2	4.7	3000 x 1800		3.5	2.7
1200 x 1200		8.8	5.0	3000 x 2400		3.7	2.9
1500 x 450	100 S	4.7	3.0	3000 x 3000		3.1	2.7
1500 x 600		4.9	3.1	3600 x 900	75 S	3.1	2.5
1500 x 900		5.3	3.3	3600 x 1200		3.2	2.6
1500 x 1200		5.6	3.5	3600 x 1500		3.3	2.6
1500 x 1500		6.0	3.7	3600 x 1800		3.3	2.7
1800 x 600	75 S	3.3	2.4	3600 x 2400		3.5	2.8
1800 x 900		3.6	2.5	3600 x 3000		3.5	2.7
1800 x 1200		3.8	2.7				
1800 x 1500		4.0	2.8				
1800 x 1800		4.3	2.9				

10.3. PORTAL BASE SLABS

Most pre-cast portal culverts are placed on cast in place base slabs. These should be designed to take the actual loads that will be applied to them.

It is important to realise that the moments and shears generated by the installed loads on base slabs are different from those generated on the portal unit. The total load on the portal unit will be transferred down the legs to the base slab. The moments and shears will then be transferred through the base slab to the founding material. If this material is unyielding the load will be transferred directly through the slab generating shear but no bending moments. If however, the material under the slab is yielding, both shear forces and bending moments will be generated.

The installed loads on the portal crown are assumed to be distributed over the whole width of the portal, except for the very low fill heights where there are concentrated loads

from the traffic. The test loads on the portal crown are concentrated live loads. If there were no factoring of installed loads, the test loads would be about half the installed ones. Hence, the test loads model the installed loads on the portal crown, but do not model the installed loads on the base slab. If pre-cast bases are to be used, a check should be done to ensure that they are sufficiently strong to take the imposed loads.

11. CONCLUDING COMMENTS

The satisfactory performance of any buried conduit is dependent upon the quality of decisions made in the design office, the quality of the products supplied to site and the quality of workmanship on site.

It is essential that the designer's requirements are clearly stated in the project specifications so that the supplier knows what requirements the products must meet and the contractor knows how the products should be installed.

By correctly applying the principles stated in this handbook and its companion volume the "Concrete Pipe and Portal Culvert Installation Manual" precast concrete pipes and portal conduits should provide a service life in excess of 100 years.

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