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PIPELINE STRUCTURAL DESIGN FOR TRENCHLESS APPLICATIONS

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ABSTRACT: The structural design methodology for buried pipelines for conventional (trenched) applications is well documented in existing published standards. Such information is however very limited for trenchless installations. It is common practice to use these conventional trench installation design methods for trenchless installations both for new pipelines (pipe jacking, microtunnelling and horizontal directional drilling) and pipeline rehabilitation. This approach often results in conservative outcomes. This paper identifies what information is available in existing Australian and New Zealand standards, overseas standards and other publications and provides design guidelines for the structural design of both rigid and flexible pipelines installed using trenchless installation methods.

1. INTRODUCTION

The structural design of any buried pipeline involves a consideration of loads applied to the pipe and the related load effects on the pipe as a result of these loads. Consideration must be given to both the structural properties of the pipe material and the support provided to the pipe by the surrounding soil. In Australia and New Zealand the structural design of buried pipes is generally carried out in accordance with existing and separate published standards for both rigid and flexible pipe materials. Rigid pipe materials in Australia and New Zealand are typically concrete and vitrified clay and design of the buried installation is carried out in accordance with existing published standards AS/NZS 3725 (Standards Australia / Standards New Zealand, 2007) and AS 4060 (Standards Australia, 1992) respectively. Flexible pipes include a much wider range of materials and the structural design of the buried pipe is carried out in accordance with AS/NZS 2566.1 (Standards Australia and Standards New Zealand, 1998). Each of these standards contains varying degrees of guidance for trenchless installations. AS/NZS 3725 and AS 4060 both contain a methodology for pipes installed by tunneling whereas AS/NZS 2566.1 specifically states that this type of installation is excluded from the scope of this standard.

The purpose of this paper is to stimulate discussion within the industry. It details the different types of trenchless pipe applications, how different design standards provide design methodologies for such installations and finally how such structural design can be completed for different types of pipes for these installations.

2. DIFFERENT TYPES OF TRENCHLESS PIPE INSTALLATIONS

In this paper the term “pipe” is used somewhat generically to describe both new pipe installations in the conventional sense, but also structural linings of existing conduits which are typically designed in a similar manner to new pipe installations. The International Society for Trenchless Technology (ISTT) provides a good summary of different construction techniques for installing “pipes” and Figure 1 is based on this. The fundamental difference between traditional pipe installations installed either in a trench or embankment construction and a typical trenchless installation is the amount of disturbance of soil around and above the pipe and what happens to this soil in the years immediately after this installation. In traditional methods there is quite significant soil settlement and/or consolidation in the years immediately after the pipe installation. In trenchless installations there is often no immediate contact between the new pipe and surrounding soil, generally little or no disturbance in soil above the new pipe but there could be settlement above the pipe as a result of the trenchless installation.

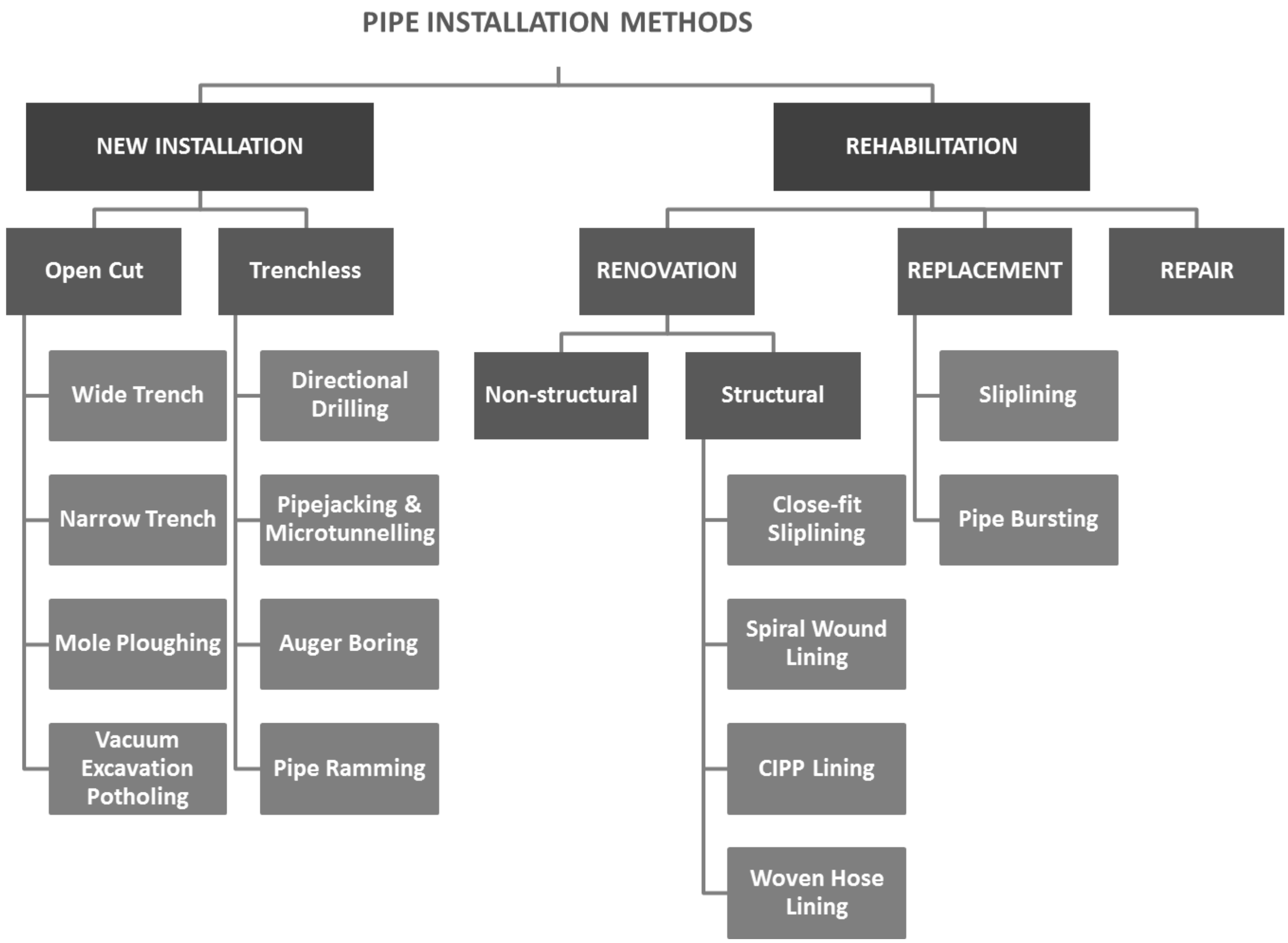


Figure 1 - Pipe Installation Techniques (after ISTT Guidelines - http://www.istt.com/guidelines)

Marston and Anderson (Marston, A. and Anderson, A. O., 1913) as early as 1913 identified that the loads acting on a buried pipe depends on the method of installation. In many respects the same engineering principles detailed in this early work can be applied to trenchless installation methods.

3. DIFFERENT TYPES OF PIPES INSTALLED USING TRENCHLESS TECHNIQUES

The selection of the type of pipe should consider both the installation method and the required service life. Table 1 contains a summary of the different types of pipe materials used for different trenchless installation methods. The type of pipe has also been included as this differentiation is the main feature which determines the design methodology to be adopted. Some techniques have been grouped together that have common features with respect to the actual pipe installation method as relevant to the pipe structural design.

Table 1 - Different Pipe Materials and Type for Different Trenchless Installation Methods

Method # [1]	Trenchless Installation Method	Common Pipe Materials	Type of Pipe
1	Horizontal Directional Drilling (HDD)	Polyethylene	Flexible
		Steel	Flexible
		Fusible PVC	Flexible
		Ductile iron	Flexible
2	Pipe Jacking & Microtunnelling Auger Boring	Concrete	Rigid
		Vitrified clay	Rigid
		Glass Reinforced Plastic (GRP)	Flexible or Rigid[4]
		PVC	Flexible
		Steel	Flexible
3	Pipe Ramming	Steel	Flexible
4	Renovation (Structural)	Polyethylene	Flexible
		PVC (folded and spiral wound)	Flexible
		GRP[2] (CIPP)	Flexible
5	Sliplining	Polyethylene	Flexible
		GRP	Flexible or Rigid[4]
		PVC	Flexible
6	Pipe Bursting	Polyethylene	Flexible
		GRP[3]	Flexible or Rigid[4]
		PVC[3]	Flexible
		Concrete[3]	Rigid
		Vitrified clay[3]	Rigid

Notes to Table 1:

1. The Method number listed has no particular significance and has been included for reference purposes for this paper only.
2. The term GRP here has been used as a generic description of a range of cured in place pipe (CIPP) which includes some form of fabric and resin.
3. In Australia and New Zealand most pipe bursting involves replacement with a welded polyethylene pipe. Replacement with discrete pipe sections (GRP, PVC, Concrete of Clay) with flexible joints is more common overseas.
4. Depends on pipe stiffness – see section 6.4.

One of the important features of each of the installation methods with regards the design, is how the new pipe is placed in relation to the existing ground. Figure 2 contains a graphical representation of new “pipe” installed using different trenchless techniques. This somewhat theoretical representation shows the location of the new pipe in relation to the existing ground immediately after installation. Most techniques result in some gap between the pipe external diameter and the surrounding soil during and immediately after installation. If such conditions are maintained, there are likely to be no external loads acting on the pipe once it has been installed other than external hydrostatic pressure. What happens in the minutes, hours, weeks or even years following the installation will influence loads acting on the installed pipe. Depending on the soil conditions above the pipe, in time, movement of soil above the pipe will occur. This movement results in two things happening. Soil pressures due to the weight of soil above the pipe or induced by live loads will be imposed on the pipe. Secondly this movement will induce shear forces in the column of soil above the pipe which may limit the final vertical soil pressure acting on the pipe. This is where the actual methodologies adopted will influence these loads.

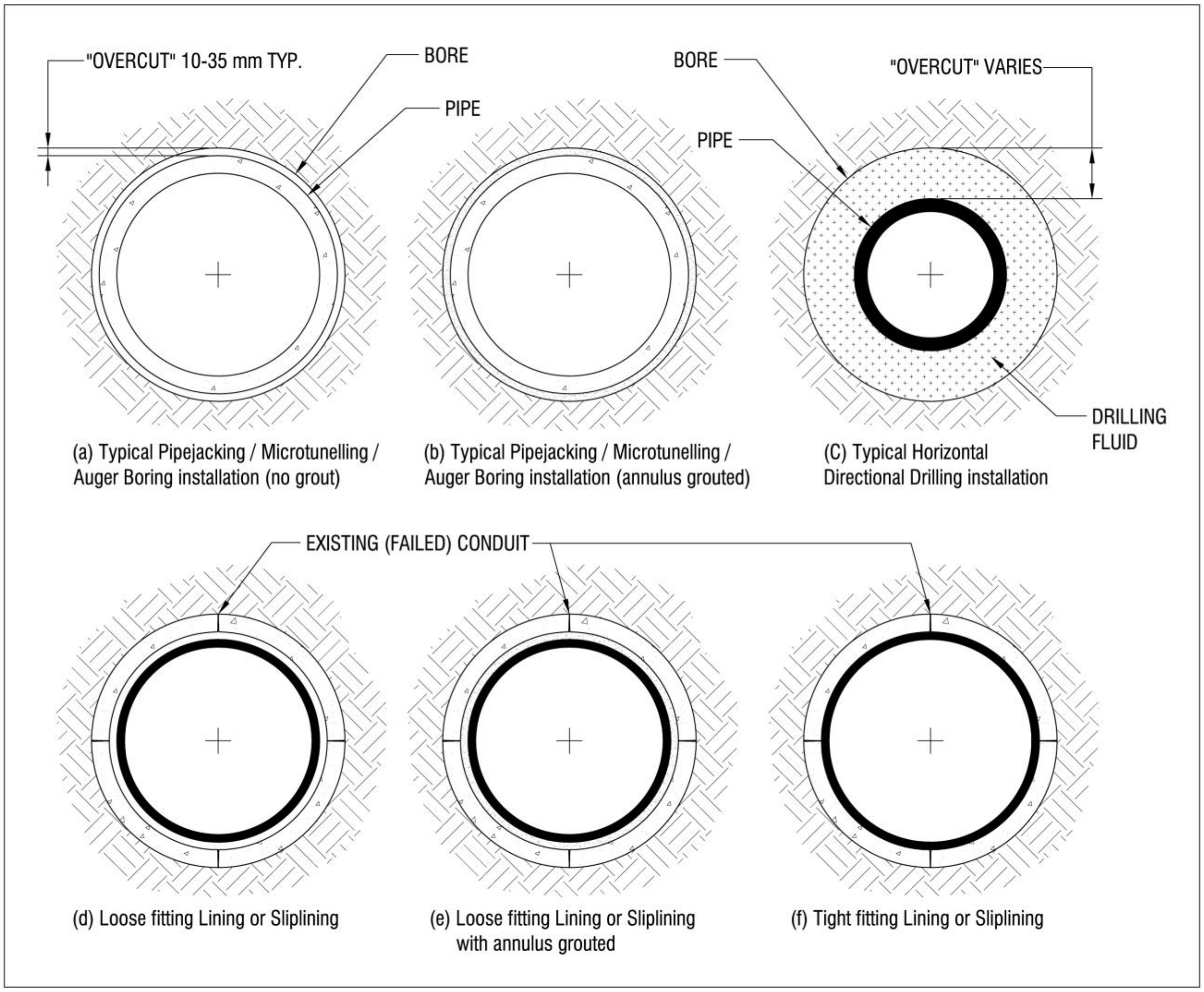


Figure 2 - Typical "pipe" installations using Trenchless techniques

4. LOADS ACTING ON PIPES FOR TRENCHLESS INSTALLATIONS

There are potentially a large number of different loads that can act on a pipe installed using trenchless techniques and all need to be considered. In many early trenchless installations success was simply measured as "getting from A to B". It is suggested that this is now the norm and required asset life for trenchless installations should be no different to conventional installation techniques. Most authorities would require an asset life in the range of 50 – 100 years. Ensuring that the pipe is not subject to loads that exceed its capacity during this asset life is an obvious and important criterion.

There are a number of different ways to group or classify such loads. Stein (Stein, 2005) classifies different loads according to their direction of application, specifically, loads applied perpendicular to the pipe axis and loads applied in the direction of the pipe axis. Thomson (Thomson, 1993) considers different loads according to the different design phases namely, design for the permanent works and design for the temporary works. These two classifications practically are very similar as most loads acting on a pipe perpendicular to the pipe axis tend to be permanent design loads and most loads acting in the direction of the pipe axis tend to be temporary or technique related installation loads. For the balance of this paper loads will be classified as either:

i. **Permanent design loads** – these are loads that may be applied to a pipe after installation and for the balance of its service life after it has been installed. Such loads are usually only known by the pipeline designer and the initial pipe selection should be made by them after a consideration of these loads; or

ii. **Installation design loads** – these are loads that are applied to the pipe during installation. Such loads are usually only known by the pipe installation contractor and the pipe to be installed should be either checked or selected based on a consideration of these loads.

This is a useful classification particularly when it comes to assigning design responsibilities. However designers need to be aware that for some installation techniques there may be some residual load effects that may need to be considered with the permanent design loads. For example, steel pipe installed using Horizontal Directional Drilling may have some bending and other residual stresses that need to be considered (Watson, 1995) as part of the permanent works design.

Temporary design load calculation methodologies are generally well documented in a number of publications and have not been considered further. Due largely to differences in loads and design methodologies in existing published standards it is a further useful classification to consider both rigid and flexible pipes separately.

5. RIGID PIPES AND TRENCHLESS INSTALLATIONS

The most common rigid pipe material used in trenchless applications in Australia and New Zealand is precast concrete. Structural design of precast concrete pipes is carried out in Australia and New Zealand in accordance with AS/NZS 3725 (Standards Australia / Standards New Zealand, 2007) and the methodologies contained in this standard relevant to trenchless installations are summarised in the following sections. Similar requirements are contained in AS 4060 (Standards Australia, 1992) for vitrified clay pipes. High stiffness GRP pipe in some instances could alse be classified as rigid and related design issues for such pipes are beyond the scope of this paper.

5.1 Permanent Design Loads – Method 2

In AS/NZS 3725 the following types of loads are considered:

(a) Working loads due to fill or in situ materials.

(b) Working loads due to superimposed dead loads.

(c) Working loads due to superimposed live loads.

(d) Working loads due to weight of internal water.

(e) Internal fluid pressure (for pressure pipes only).

Loads (c) through to (e) are calculated independently of the method of installation. Loads due to fill or insitu material are, however, calculated for the different methods of installation based on the various theories developed by Marston and later by Spangler (Handy, R.L. and Spangler, M.G., 2007). Different formulae are provided for the different types of installations including trench, embankment (both positive and negative projection) and for jacked (thrust) or bored pipe condition this load is calculated in accordance with Equation 1 (but not less than $1.5wB^2$):

$$W_g = C_t w B^2 - 2cC_t B \qquad [1]$$

Where:

W_g is the working load on a pipe due to external dead loads;

w is the unit weight of fill;

B is bore width;

c is defined as the apparent soil cohesion.

C_t is obtained from Figure 6 or from equation C1 from AS/NZS 3725 Supp 1 replicated as Equation 2 below.

$$C_t = \frac{1-e^{-2K\mu'(\frac{H}{B})}}{2K\mu'} \qquad [2]$$

Where:

$K = (1\text{-}sin\o)/(1+sin\o)$ = Rankine lateral earth pressure coefficient;

$\O$ = angle of internal friction in soil above the pipe;

$\varnothing'$ = friction angle between soil and trench walls;

$\mu' = \tan \varnothing'$ = coefficient of friction between fill material and sides of trench; or

$\mu = \tan \varnothing$ = coefficient of friction within the fill material.

Figure 3 shows the results for a DN900 pipe of calculating the working load due to fill above the pipe for a number of different conditions:

1. Using Equation 1 above for a value of soil cohesion c = 0. This is equivalent to pipe buried in a trench all be it with a very narrow trench width.
2. Using Equation 1 above for different values of soil cohesion c = 2 and 5.
3. The prism load. This is simply the vertical soil pressure at the top of bore (*wH*) multiplied by the pipe external diameter (D). The prism load is also often referred to as the value *wH*.

In effect, Equation 1 calculates the load acting on the bored pipe as the weight of the prism of soil above the pipe minus the effects of frictional forces acting on this prism and minus the forces acting on the soil prism due to cohesion in the soil above the pipe.

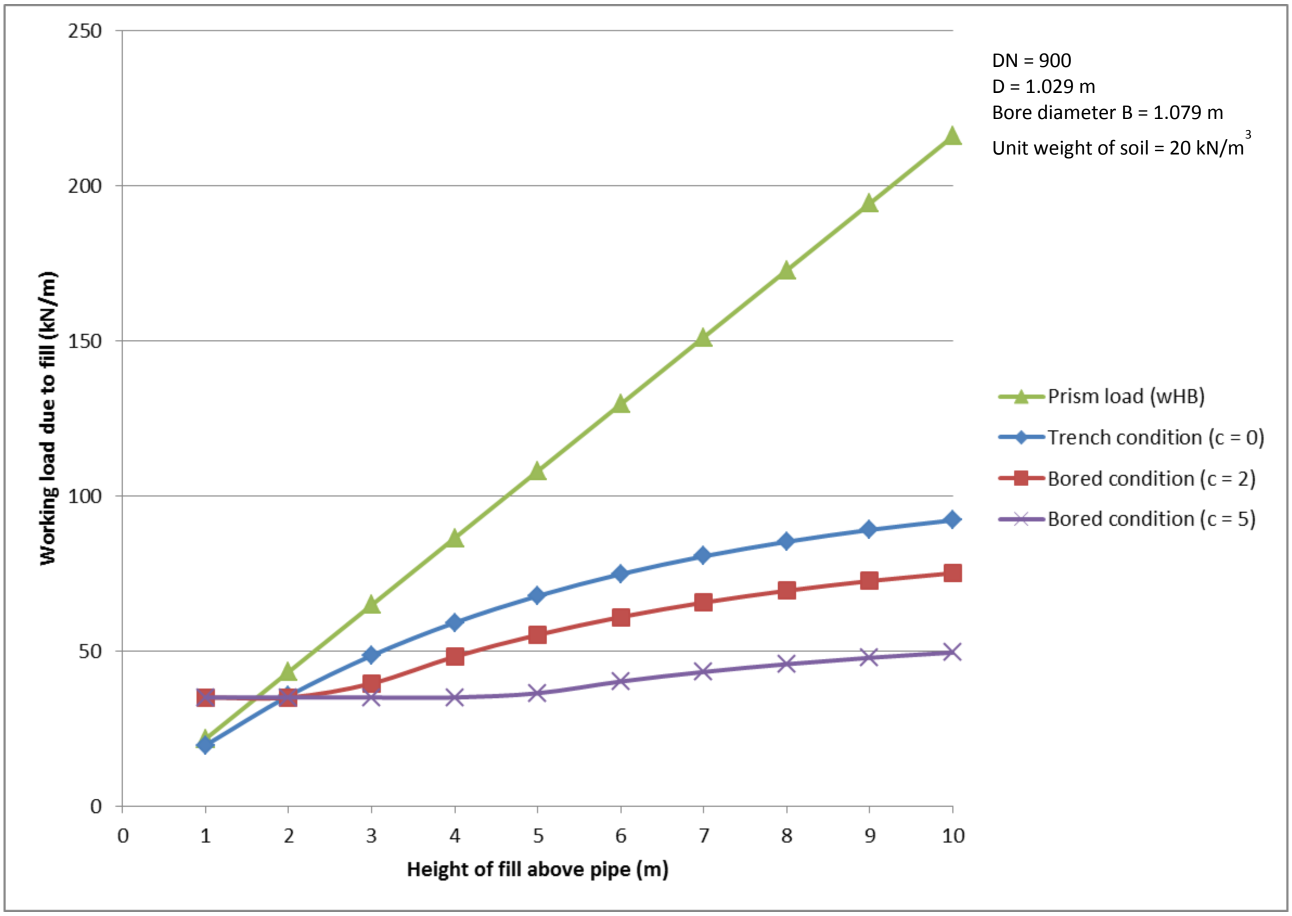


Figure 3 - Working loads due to fill for different installation methods

5.2 Pipe Structural Design – Rigid Pipes Method 2

For precast concrete pipes, once working loads acting on the buried pipe are determined the required class strength of pipe is determined by dividing this total load by a bedding factor (*F*). According to AS/NZS 3725 for jacked or bored pipes this bedding factor shall be in the range of 2 to 3. AS/NZS 3725 Supp. 1 states that the bedding factor selected depends on the degree of over excavation, which in trenchless terminology, usually is referred to as the amount of overcut. It is common practice to use a value of 2 for installations where the annulus between the pipe and the excavated bore is not grouted or 3 when grouted.

6. FLEXIBLE PIPES AND TRENCHLESS INSTALLATIONS

A variety of flexible pipe materials are installed using trenchless techniques. Structural design of buried flexible pipes is carried out in Australia and New Zealand in accordance with AS/NZS 2566.1 (Standards Australia and Standards New Zealand, 1998).

6.1 Permanent Design Loads – AS/NZS 2566.1 Approach

Similar to AS/NZS 3725, AS/NZS 2566.1 considers the following types of loads:

(a) Trench or embankment fill;

(b) External hydrostatic loads;

(c) Internal pressure;

(d) Superimposed dead loads;

(e) Superimposed live loads; and

(f) Mass of the contents of the pipe, if appropriate.

Unlike AS/NZS 3725, however, AS/NZS 2566.1 excludes trenchless installations and states that it "*does not give design guidelines for … bored, jacked or mole-ploughed installations".* It is suggested that loads (b) through (f) listed above for a trenchless installation could be calculated in the same manner for the conventional trench or embankment installations. If this suggestion is accepted, then the calculation of the load acting on the pipe due to fill above the pipe is the main area of difference between trenchless and conventional installations. Determination of the load effects (i.e. deflection, strain, buckling etc.) is another matter and this is discussed in Section 6.4.

The dead load due to trench or embankment fill (w_g) in AS/NZS 2566.1 is calculated in accordance with Equation 2:

$$w_g = \gamma H \tag{2}$$

where:

Υ = the assessed unit weight of trench fill or embankment fill; and

H = cover, the vertical distance from the top of the pipe to the finished surface.

In the commentary to this standard, AS/NZS 2566.1 Supp1 (Standards Australia and Standards New Zealand, 1998), it is stated that this approach has been adopted because of its simplicity and because it gives conservative values. An alternative formula (Equation 3 below), based on Terzaghi's silo theory, is included in standard which includes what is termed the silo reduction factor (κ).

$$w_g = \kappa \gamma H \tag{3}$$

where:

$$\kappa = \frac{1-e^{-2\frac{H}{B'}K_o tan\delta}}{2\frac{H}{B'}K_o tan\delta} \tag{4}$$

Where:
B' = width of slip plane at the top of the pipe.

K_o = ratio of lateral to vertical soil pressure (has a value between active and passive),

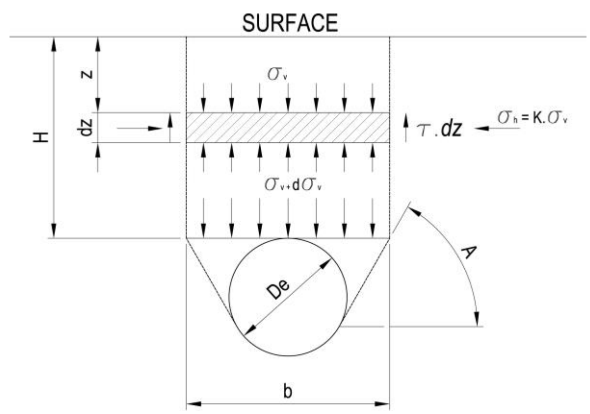
δ = friction angle on the slip plane, $0<\delta<\emptyset$;

$\emptyset$=soil friction angle for fill material.

6.2 Vertical Soil Loads Methods 1 & 2 – Overseas Publications

The issue of determining an appropriate value of load due to the weight of soil above the pipe for pipes installed using trenchless techniques is detailed in a number of international publications.

The French Society for Trenchless Technology (French Society for Trenchless Technology, 2006) provides a good summary of a number of approaches for microtunnelling installations without specifically stating whether the recommendations refer to rigid or flexible pipes. In this 2006 publication they state that there "*currently exists no French regulation for the sizing of pipes installed "without trenches" but that the information contained in this guide* "*will be adapted to the specifications for trenchless work*." The French guide provides details of different methods for calculating the vertical loads due to fill above the pipe including Terzaghi, Leonard and Marston theories. They provide an explanation of the general Terzaghi model which is replicated as Figure 4 below. They include varying recommendations for calculation of the vertical load due to fill above the pipe depending on the value of "H" and the width "b". The basic Terzaghi model is that the ground located above the pipe slips in relation to two vertical planes separated by a width "b". At a depth H above the pipe the vertical soil pressure at the top of the pipe (σ_{EV}) is calculated using equation 5 in Figure 4 – this expression includes both friction and cohesion of the soil.



$$\sigma_{EV} = \frac{H.\left(\gamma - \frac{2.c}{b}\right)}{2.K.tan\delta.\frac{H}{b}}.\left(1 - e^{-2.K.tan\delta.H/b}\right) \qquad [5]$$

$$A = \frac{\pi}{4} + \frac{\varphi}{2}$$

$$b = D_e.\tan\left(\frac{3\pi}{8} - \frac{\varphi}{4}\right)$$

$$\varphi = soil\ internal\ friction\ angle$$

$$\delta = friction\ angle\ along\ vertical\ planes$$

$$K = Lateral\ earth\ pressure\ coefficient$$

$$c = soil\ cohesion$$

$$\tau = c + \sigma_V.K.tan\delta$$

Figure 4 - General Terzaghi Model (after French Society for Trenchless Technology, 2006)

The French Guide provides a number of recommended equations based on the different theories listed above for determining the vertical soil pressure (σ_{EV}) considering both the cohesion (c) and internal friction angle of the soil (φ) for different conditions:

(a) In the case of homogeneous ground above the pipe and c and ϕ are known σ_{EV} is determined in accordance with the logic contained in Table 2.

Table 2 - Summary of French Guidelines for Homogeneous Soils with known Soil Friction & Cohesion

Condition	Formula
If H > b	$\sigma_{EV1} = \gamma.b = \gamma.D_e.\tan\left(\frac{3\pi}{8} - \frac{\varphi}{4}\right)$
If H < b	$\sigma_{EV1} = \gamma.H$
If c = 0	$\sigma_{EV2} = k_M.\gamma.H$ where $k_M = \frac{1 - e^{-2.K.tan\varphi.\frac{H}{De}}}{2.K_a.tan\varphi.\frac{H}{De}}$ and $K_a = tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)$
If c ≠ 0	$\sigma_{EV3} = k_M.H.\left(\gamma - \frac{2c}{D_e}\right)$ with k_M and K_a as above.
If $\sigma_{EV1} > \sigma_{EV2}$	$\sigma_{EV} = \sigma_{EV2}$ even if c ≠ 0

If $\sigma_{EV1} < \sigma_{EV3}$	$\sigma_{EV} = \sigma_{EV3}$
If $\sigma_{EV3} < \sigma_{EV1} < \sigma_{EV2}$	$\sigma_{EV} = \sigma_{EV1}$

(b) In the case of heterogeneous ground above the pipe or if the characteristics of the homogeneous ground are not well known the recommendation is that a basic value of 30° for the friction angle and:

$$\sigma_{EV} = k_M.\gamma.H$$

with k_M as detailed in Table 2 above.

(c) If the host ground is made up of clay or very plasic marl:

$$\sigma_{EV} = \gamma.H$$

Whilst the above seems like a complicated set of criteria the common features are that the vertical soil load is the prism load (γH) with or without a reduction due to soil friction and/or soil cohesion. It should be noted that the French Guidelines do warn that soil cohesion should be used with great care.

The German standard ATV-A 161E (German Association for the Water Environment, 1990) contains guidelines specifically for "driven pipes". It is assumed that "driven" in this case would be relevant to pipes installed using pipe jacking / microtunnelling or auger boring techniques. This standard states that "*the calculation formulae and material characteristic values refer to pipes constructed of reinforced concrete, fibre cement, steel and vitrified clay*" – i.e. a combination of what would be locally classified as rigid or flexible pipe materials. Stein (Stein, 2005) suggests that ATV-A 161E can also be used for jacking pipes of flexible materials such as plastic and GRP with the "*sequence of calculation analogous to that for steel pipes, material properties can be taken from ATV-A 127E*".

In ATV-A 161E the vertical earth load is determined for a particular case of the Terzaghi model with angle φ = 30°. In the diagram in Figure 4, angle A = 60° and width $b = D_e.\sqrt{3}$. In this standard the vertical pressure at the top of the pipe (p_{GV}) is calculated in accordance with Equation 6. Symbols used in Equations 6 and 7 are generally consistent with those detailed in Figure 4 except ϕ' is simply defined as the "angle of internal friction".

$$p_{GV} = \mathbf{k}.\gamma.H \qquad [6]$$

$$\text{Where } \mathbf{k} = \frac{1-e^{-2.K1.\tan\left(\frac{\phi'}{2}\right).\frac{H}{b}}}{2.K1.\tan\left(\frac{\phi'}{2}\right).\frac{H}{b}} \qquad [7]$$

In this case silo reduction factor is calculated using a value of $\phi'/2$ in the equation. In the Explanatory Notes it suggests that "procedural measures" are made to keep deformations small (i.e. reduce settlement at surface) and suggests that about half of the frictional forces are activated at about 10% of the total settlement, hence justifying the use of $\delta = \phi'/2$ in the Terzaghi equation. This does result in higher values of the estimated loads acting on the pipe than if the full value of the soil friction angle was used. ATV-A 161E also tabulates values for K1 = 0.5.

Horizontal Directional Drilling (Method 1) typically involves a larger "overcut" than the other techniques but the principles should apply. ASTM F1962 (ASTM, 2011) suggests that "Terzaghi's equation" could be used for this installation method and lists the same equations as (6) and (7) above. It notes that the friction angle, has been reduced in Terzaghi's equation by 50%. It goes onto suggest that "*credit for arching should only be considered where the depth of cover is sufficient to develop arching (typically exceeding five pipe diameters), dynamic loads such as traffic of rail loads are insignificant, the soil has sufficient internal friction to transmit arching, as confirmed by a geotechnical engineer*".

In the preceding discussion regarding overseas publications the common use of the Terzaghi's equation or silo theory is not specifically related to flexible pipes. The common feature however is that with Methods 1 and 2, there is an amount of overcut when the pipe is installed which is generally considered sufficient to induce the slip planes as described by Terzaghi independent of the type of pipe. In such installations if there is no movement in the column of soil above the pipe then there will be no contact between this soil and the pipe and hence no load will be applied due to this height of fill. (An exception may be over consolidated clays which may expand as the bore is opened).

6.3 Vertical Soil Loads – Comparison of Different Methods

In an earlier section of this paper details were provided of vertical soil loads calculated in accordance with AS/NZS 3725 for a DN900 pipe with varying heights of fill up to 10 m (Figure 3). Being a concrete pipe these loads are calculated as a line load (kN/m) based on a prism width equal to the bore width, *D*. If one simply divides these loads in (kN/m) by the prism width, a vertical soil pressure (kPa) at the top of the pipe is easily calculated. Some of these results are from Figure 3 are presented in this manner in Figure 5 along with the results for other calculations in accordance with AS/NZS 2566.1 Supp 1 and ATV-A 161E. The results indicate:

(a) Other than at low heights of fill, the estimates of vertical soil pressure are all signifcantly lower than the prism load.

(b) The ATV-A 161E calculations are the next highest values. This is solely as a result of using only 50% of the angle of internal friction in the calculation of the silo reduction factor.

(c) Other than the prism load, the other results are all quite similar demonstrating the similarities between the Marston and Terzaghi theories. The formulae are basically the same. The differences (if one ignores cohesion) are largely in what value of the lateral earth pressure coefficient K one uses and in the case of ATV-A 161E, the value of soil friction.

(d) Inclusion of a soil cohesion of 5 kPa has a significant impact on the estimation of the vertical soil pressure.

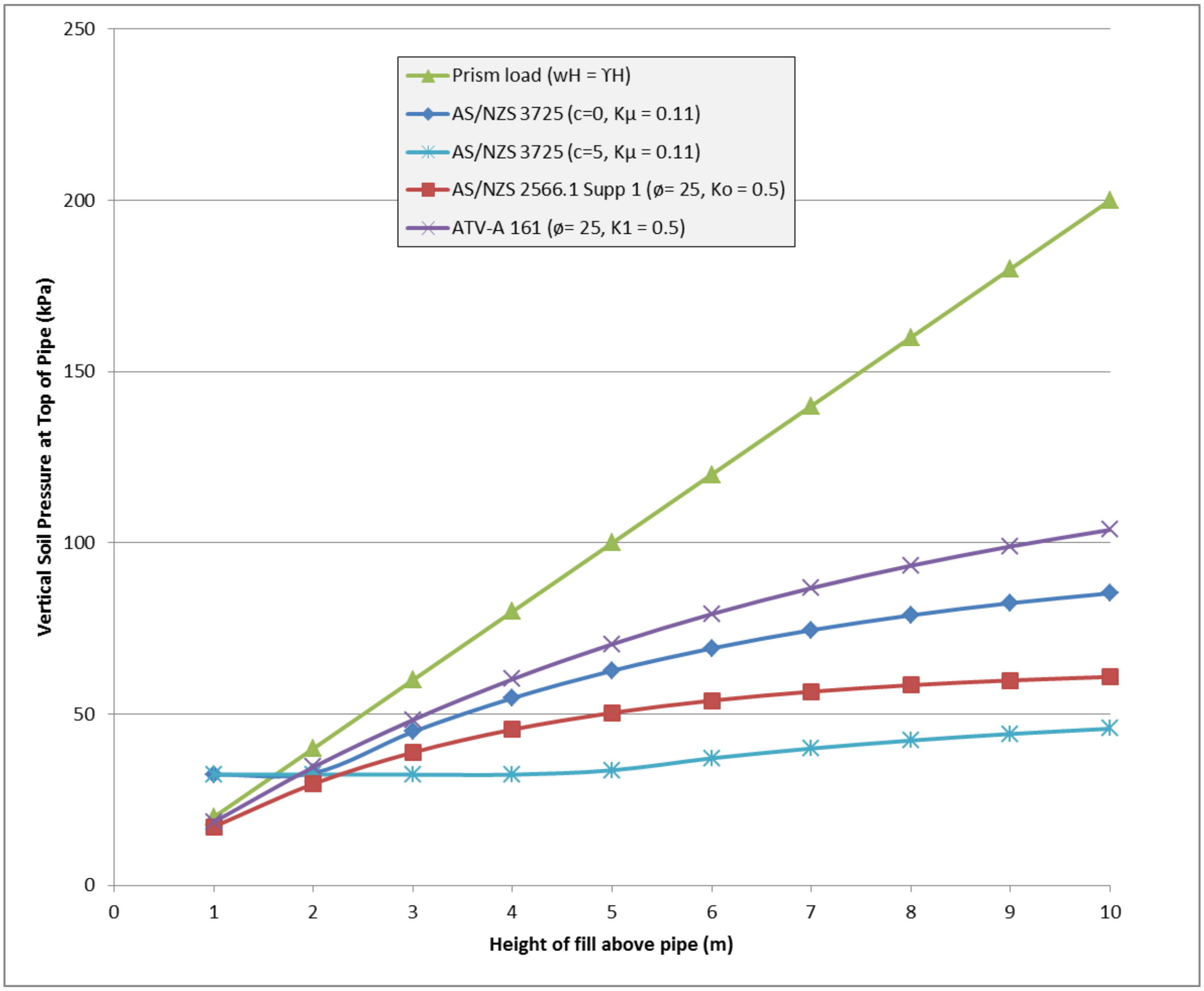


Figure 5 - Comparison of Vertical Soil Pressures Using Different Methods

6.4 Flexible Pipe Design – Trenchless Installation Methods

In the previous sections a discussion of loads acting on the flexible pipe installed using trenchless methods has been discussed. Once loads have been determined the next step is to consider the load effects. AS/NZS 2566.1 considers the following load effects for a buried flexible pipe installed in either a trench or embankment:

(a) Deflection.

(b) Strength – Strain, internal pressure and combined loading.

(c) Buckling.

Prior to discussing each of these it is worth considering the definition of a flexible pipe. AS/NZS 2566.1 Supp. 1 states that the response of flexible pipes with a very high long term ring bending stiffness (S_{DL}) is likely to be in either the flexible or rigid mode depending on whether the value of S_{DL} is less than or greater than 7500E' respectively, where E′ is the combined soil modulus. If one adopted the definition of a flexible pipe and considered typical native soil moduli in the range 1-5 MPa (Table 2.1 AS/NZS 2566.1), then a pipe would be considered as being "flexible" with a long-term stiffness less than or equal to 7500 (for E′=1 MPa) or 37500 (for E′=5MPa) N/m/m.

Some GRP jacking pipes in use in Australia and New Zealand have quoted stiffness (SN) values varying between SN=32000 and SN=1000000. These would typically be considered short term stiffness values. Long term stiffness values would typically be in the range of 50-70% of these values. Such pipes at the lower end of this stiffness range could be considered as flexible pipes and the load effects listed in (a) to (c) above could be considered relevant. Pipes with SN values of say 100000 would not be considered flexible and would therefor need to be considered as rigid pipes. It is beyond the scope of this paper to consider these issues further.

The following sections provide comment on the suitability of the design requirements of AS/NZS 2566.1 for trenchless installation methods for flexible pipes.

6.4.1 Pipe Deflection – Trenchless Installation Methods

Pipe deflection is an important criteria and for trenchless installations it should be no different. The deflection formula contained in AS/NZS 2566.1 is what is generally referred to as the modified Iowa formula. This determines the deflection of a flexible conduit for a given load based on both the pipe stifness and the combined soil modulus. This formula was originally developed by Spangler and later modiifed by Watkins (Handy, R.L. and Spangler, M.G., 2007). Figure 6 is an extract from Handy and Spangler showing the assumed pressure distribution around the pipe relevant to this deflection formula.

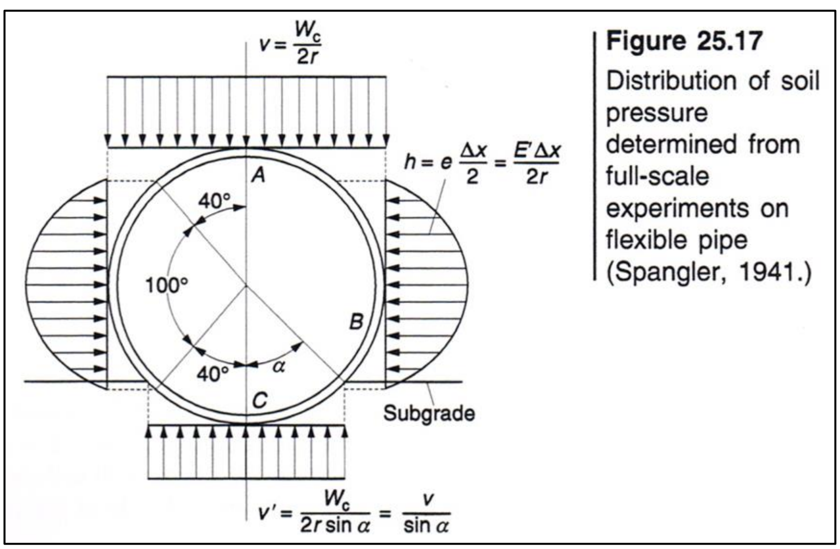
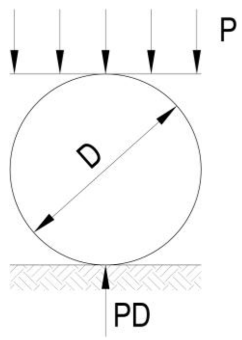


Figure 6 - Pressure distribution around pipe - modified Iowa Formula (Hardy & Spangler, 2007)

It is suggested that such a pressure distribution would not be possible for a number of trenchless installation due to a lack of side support from the surrounding soil – see Figure 2 installations (a), (c) and (d). If the annulus is grouted - see installations (b) & (e) – or in the case of tight fitting lining it could be argued that such side support exists. In the absence of such side support it is suggested an alternative formula for predicting deflection should be used. Watkins and Anderson (Watkins, R.K. and Anderson, L.R., 2000) provide formulae for deflection of thin walled rings with symmetrical loads. One of these is included in Figure 7 as Equation 8. In this case the support at the base of the pipe could be considered as somewhat extreme but it is further suggested that any such deflection predicted by using this formula is likely to be greater than the actual. In the formula provided the denominator is the pipe stiffness (EI/D^3) as defined in Section 2 of AS/NZS 2566.1.

Loading Diagram



Deflection Equation for Loading Diagram shown

$$\frac{\Delta}{D} = \frac{0.0145P}{\frac{EI}{D^3}} \qquad [8]$$

Figure 7 - Loading diagram and deflection equation for pipe with no side support (Watkins & Anderson)

6.4.2 Strength Requirements – Trenchless Installation Methods

Strength requirements in AS/NZS 2566.1 include strain, internal pressure and combined loading. The strain calculation is a function of the shape factor, actual deflection and pipe geometry. With no side support (i.e. $E'=0$) the shape of the deflected pipe would approach that of a pure ellipse and according to AS/NZS 2566.1 Supp1 a shape factor value of 3.0 could be adopted. This would allow a calculation of the actual strain as the deflection and pipe geometric properties are known. Internal pressure (if relevant) is purely a function of the actual vs allowable internal working pressures. Similarly once strain and pressures values are known a combined loading (if relevant) calculation in accordance with AS/NZS 2566.1 could be carried out for trenchless installations in the same manner as for conventionally installed pipe.

6.4.3 Buckling Requirements – Trenchless Installation Methods

The final load effect to be considered is buckling. Buckling calculations in AS/NZS 2566.1 involve a comparison of actual buckling pressure due to applied loads and then a comparison with a calculated allowable pressure. The allowable buckling pressure is calculated based on a comparison of values from two different equations. AS/NZS 2566.1 Supp1 provides the origin of these two equations. The first equation is generally what is known as Timoshenko's buckling equation in which the maximum allowable pressure is calculated based on the pipe properties only. The second equation, referred to as Moore's equation in AS/NZS 2566.1 Supp1, includes the effects of both pipe stiffness and soil support. In the absence of soil support it is suggested that only Timoshenko's equation should be used.

6.5 Design - Methods 4 & 5

The various methods for both renovation and sliplining potentially represent the most complex loading situation due to the difficultly in understanding the loading history, existing conduit condition and the capacity of the lined conduit. Common practice in Australia is to adopt in part the requirements of ASTM F1216 (ASTM, 2009) and in part the requirements of AS/NZS 2566.1 for both determination of design loads and consideration of load effects.

ASTM F1216 provides non-mandatory guidelines for the design of pipe rehabilitation using a technique referred to as inversion and curing of resin-impregnated tubes and generally refers to this as cured in place pipe (CIPP). Similar

ASTM standards also exist for other lining methods. It identifies two different conditions based on the condition of the existing pipe and refers to these conditions as a:

- *Partially deteriorated pipe*. In this condition the existing pipe is assumed to be capable of sustaining the soil and surcharge loads. For a pipe not subject to internal pressure (non-pressure) the lining is then designed to only sustain the external hydrostatic pressure and for a pressure pipe both the external hydrostatic loads and internal hydrostatic loads spanning across any holes in the original pipe wall. For the non-pressure pipe the standard provides a formula for determining a buckling capacity of the lining which is an adaption of Timoshenko's buckling equation with additional factors to account for the increased capacity due to the existing pipe and the surrounding soil (Enhancement Factor) and the ovality of the existing conduit (Ovality Reduction Factor). These similar requirements are often included in specifications in Australia except that the condition is often referred to as an "intact pipe".
- *Fully deteriorated pipe*. In this condition the original pipe is considered to be incapable of sustaining soil and live loads. In this case the lining thickness is determined largely from equation X1.3 which is replicated as Equation 9 below. In addition the design is also checked to ensure that the partially deteriorated condition is met along with a minimum thickness requirement. According to an ASCE report (ASCE, 2007) the origin of this formula is a modified buckling equation developed for the direct burial of fibreglass pipe (AWWA, 2014).

$$q_t = \frac{1}{N}\left[32\,.R_w.B'.E'_S.C\left(\frac{E_L I}{D^3}\right)\right]^{0.5} \qquad [9]$$

Where:

q_t = external pressure on pipe = $0.00981H_w + (wHR_w)/1000 + W_s$

R_w = water buoyancy factor (0.67 min.) = $1 - 0.33(\frac{H_w}{H})$

w = soil density

W_s = live load

H_w = height of water above top of pipe

H = height of soil above top of pipe

B' = coefficient of elastic support = $1/(1 + 4e^{-0.213H})$

I = moment of inertia of CIPP = $t^3/12$

t = thickness of CIPP

C = ovality reduction factor

N = factor of safety

E'_s = modulus of soil reaction

E_L = long term modulus of elasticity for CIPP

D = mean inside diameter of original pipe

For both these different conditions for non-pressure applications, the external pressure acting on the pipe is compared with an allowable buckling pressure. For the partially deteriorated condition the external pressure is simply the groundwater pressure. For the fully deteriorated condition the external pressure includes the external groundwater loads, the soil loads and any live loads. The soil load is equal to the full prism load (*wH*) with an adjustment for the effects of ground water.

It is common practice in Australia to adopt the equation from ASTM F1216 for the partially deteriorated condition and adopt the requirements of AS/NZS 2566.1 for the fully deteriorated condition. Table 3 contains some calculations comparing the requirements of ASTM F1216 (equation X1.3 only) for the fully deteriorated condition with buckling calculations in accordance with AS/NZS 2566.1 for a range of hypothetical design parameters. Careful consideration must be given to a variety of parameters. Four examples are provided to illustrate potential outcomes based on different combinations of values.

Table 3 - Example calculations ASTM F1216 and AS/NZS 2566.1

Item	Parameter Description	Units	Value	Value	Value	Value	Reference
1	Common Design Parameters		Example 1	Example 2	Example 3	Example 4	
1.1	Soil density	kN/m^3	18	18	18	18	
1.2	Height of fill above pipe	m	5	5	5	5	
1.3	Height of water table above pipe	m	3	3	3	3	
1.4	Live load (typical value)	kN/m^2	8	8	30	30	
1.5	Internal diameter existing pipe	mm	800	800	800	800	
1.6	Modulus of soil reaction / combined soil modulus	MPa	4	2	4	4	
1.7	Existing pipe ovality	%	5	5	10	10	
2	CIPP Design Parameters						
2.1	Wall thickness	mm	18	22	23	23	
2.2	Diameter at neutral axis	mm	782	778	777	777	
2.3	Moment of inertia / second moment of area	mm^4/mm	486	887	1014	1014	
2.4	Moment of inertia / second moment of area	m^4/m	4.86E-07	8.87E-07	1.01E-06	1.01E-06	
2.5	Long term modulus of elasticity / ring bending modulus	MPa	2000	2000	2000	2000	
2.6	Poisson's ratio (v) CIPP lining	-	0.3	0.3	0.3	0.3	
3	ASTM F1216-09 Design Calculations (Fully Deteriorated)						
3.1	Water buoyancy factor (R_w)	-	0.802	0.802	0.802	0.802	
3.2	Total external pressure acting on pipe (q_t)	MPa	0.10961	0.10961	0.13161	0.13161	
3.3	Total external pressure acting on pipe (q_t)	kPa	110	110	132	132	
3.4	Factor of safety (N)	-	2	2	2	2	
3.5	Coefficient of elastic support (B')		0.420	0.420	0.420	0.420	
3.6	Ovality reduction factor (C)		0.640	0.640	0.412	0.412	
3.7	Allowable buckling pressure	MPa	0.114	0.109	0.133	0.133	Equation X1.3
3.8	Allowable buckling pressure	kPa	114.5	109.4	132.6	132.6	
3.9	Actual factor of safety		2.09	2.00	2.02	2.02	
4	AS/NZS 2566.1 Design Calculations						
4.1	Assessed unit weight of liquid external to pipe (Υ_L)	kN/m^3	10	10	10	10	
4.2	Specific gravity of soil particles in fill (ρ_s)	-	2.65	2.65	2.65	2.65	
4.3	Submerged unit weight of fill (Υ_{sub})	kN/m^3	11.21	11.21	11.21	11.21	Equation 5.4(2)
4.4	Buckling pressure	kPa	116	116	138	138	Equation 5.4(1)
4.5	Long term liner stiffness (S_{DL})	N/m/m	2033	3769	4323	4323	Equation 2.2.1.1 (2)
4.6	Factor of safety (F_s)	-	2.5	2.5	2.5	2.0	AS/NZS 2566.1 rec. value
4.7	Allowable buckling pressure 1 (q_{all1})	kPa	21.4	39.8	45.6	57.0	Equation 5.4 (4)
4.8	Allowable buckling pressure 2 (q_{all2})	kPa	127.7	98.8	164.2	205.2	Equation 5.4 (5)
4.9	Maximum allowable buckling pressure 1 (max q_{all1} *and* q_{all2})	kPa	127.7	98.8	164.2	131.7	Example 4 reduced by $C^{0.5}$
4.10	Actual factor of safety		2.8	2.1	3.0	1.9	

There are a number of observations that can be made albeit with a very limited range of calculations:

(a) Apart from the formulae used, the fundamental difference is that ASTM F1216 includes a reduction in buckling capacity due to ovality of the existing pipe and AS/NZS 2566.1 recommends a higher factor of safety to be applied to the allowable bucking capacity.

(b) Example 1 shows reasonable correlation between the two methods for the design parameters selected.

(c) Example 2 includes a lower soil modulus (2 MPa) and for this AS/NZS 2566.1 would require an increased lining thickness (stiffness) of 26 mm (+4 mm or +18%).

(d) Example 3 includes a higher value of ovality and AS/NZS 2566.1 would allow a lower lining thickness of 20 mm (- 3mm or -13%).

(e) Example 4 is similar to Example 3 except that the factor of safety for the AS/NZS 2566.1 calculations has been reduced to 2.0 (same as ASTM F1216) but the buckling capacity has been reduced by the same ovality reduction factor as ASTM F1216 of $C^{0.5}$. The results between the two standards in this case are very similar.

Whilst it is interesting to note that there is reasonable correlation between ASTM F1216 and AS/NZS 2566.1 an American Society of Civil Engineers (ASCE) Task Committee, Emerging Concepts for Pipeline Renewal Systems (ASCE, 2007) suggest that fully deteriorated condition requirements of ASTM F1216 are too conservative and that soil loads should be ignored. The committee argues that a rigid pipe that has lost its ring stiffness by longitudinal cracking or corrosion and that it would deflect like a flexible pipe to establish a new equilibrium with the surrounding soil. They go onto argue that the lining would simply stabilise this situation and very little (if any) thrust loads and as such bucking should be ignored and that the lining should be designed for external hydrostatic loads only.

This suggestion of equilibrium implies that ongoing or additional deflection of the existing pipe will not occur. A rigid pipe (concrete or clay) can lose its load carrying capacity at a relatively low crack width (1-2 mm for a reinforced concrete pipe and much lower for an unreinforced concrete or clay pipe). Ongoing deflection will then occur reducing the soil load acting above the pipe due to both friction and soil cohesion. At the same time lateral soil pressure increases such that at some crack width and at some deflection (ovality) the equilibrium described may be reached. If not lined water and soil infiltration through wide cracks will ultimately lead to complete collapse.

The UK based WRc publication, Sewerage Rehabilitation Manual 4th Edition (SRM), has evolved over many years of research and experience of rehabilitation of sewers. In more recent years it has been developed into the Sewerage Risk Management website (http://srm.wrcplc.co.uk/). This website contains details of different renovation methods along with a summary of these methods in relation to ASTM 1216 and the German publication ATV-DVWK 127E, Part 2 (German Association for the Water, Wastewater and Waste (DWA), 2000).

Table 4 - Summary of common international design methods (from SRM website)

Type of design	SRM	ASTM 1216	ATV-DVWK-127E Part 2
Rigid composite sewer	Type I	-	-
Flexible liner in structurally stable sewer	Type II	Partially Deteriorated	Condition I & Condition II
Flexible liner in structurally unstable sewer	-	Fully Deteriorated	Condition III

It is generally accepted that design methods in accordance with either AS/NZS 2566 or ASTM 1216, as described herein for the fully deteriorated condition, are conservative. Much more work needs to be done to better refine these design methods. It is, however, suggested that care does need to be taken in applying some of the design methodologies developed, particularly for deep and often small diameter sewers, for rehabilitation of other pipelines such as road and rail culverts. Such structures are often shallow, are typically larger diameter and due to their often shallow depth and location can be subject to significant live loads.

7. CONCLUSIONS

This paper has attempted to identify how the structural design of pipes installed using trenchless installation methods might be undertaken. This work has led to a number of conclusions:

(a) The main difference between trenchless and conventional trenched installations, from a structural design perspective, is how one might estimate the loads due to vertical soil pressure.

(b) Other permanent design loads (e.g. water loads and live loads) should be considered in the same manner for any method of installation.

(c) There are a number of temporary design loads that are installation related and it is important that these are considered but this paper does not provide guidance as to how these loads should be treated.

(d) It is suggested that the vertical soil loads acting on a pipe installed using trenchless techniques, at least in a new pipe installation, is largely independent of whether the pipe is considered rigid or flexible.

(e) Existing Australian and New Zealand Standards provide guidance for design for trenchless installations for concrete and vitrified clay pipes only.

(f) With limited modification, the existing standard for the structural design of buried flexible pipes, AS/NZS 2566.1, could be used for design for new pipe trenchless installations as detailed in this paper with the estimation of vertical soil loads being the greatest challenge.

(g) There is little difference between the Marston and Terzaghi theories for estimation of vertical soil loads (pressures) for new pipe trenchless installations.

(h) Soil cohesion can have a significant effect on vertical soil loads but care needs to be taken with its use in calculations. Unless geotechnical test results are available it perhaps should be ignored.

(i) Estimation of the vertical soil loads for pipeline rehabilitation methods still represent the greatest challenge and area for research.

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