CHAPTER 2: LITERATURE REVIEW - THE DESIGN OF BURIED FLEXIBLE PIPES

2.1 INTRODUCTION

The purpose of this Chapter is to introduce the terminology and current philosophy of design of thermoplastic pipes buried in trenches. The review is not constrained to shallow burial or traffic loading. Indeed, much of the available literature has more to do with deep burial and the pressures induced by the self-weight of the backfill. In this review, areas in which knowledge are limited are identified.

Ring theory is discussed and approaches to design of pipes for external loads are reviewed. Considerations of both stifffness and strength are reviewed. An indication of the sensitivity of the stiffness of uPVC to time of loading and temperature is given. The design methods are constrained generally to two-dimensions (plane strain) and require knowledge of the stresses in the supporting soil around the pipe (sidefill or surround soil).

2.2 LOADS ON PIPES BURIED IN TRENCHES

2.2.1 No External Forces

How much load a pipe can sustain depends on the relative height of cover, the nature of both the backfill material and the natural soil, the geometry of the trench installation and the relative stiffness of the pipe to the backfill. Marston load theory, as cited by Moser (1990), recognizes that the amount of load taken by a pipe is affected by the relative movement between the backfill and the natural soil, as settlement of both the backfill and pipe occurs.

Marston proposed that the weight of the backfill was partly resisted by frictional shear forces at the walls of the trench which developed with time. He conservatively ignored the apparent cohesion of the soil when enforcing equilibrium of vertical forces to derive his solution.

A completely rigid pipe will attract load, and so the vertical force acting on the pipe may be expressed as:

$$W_d = C_d \gamma B^2 \qquad 2\text{-}1$$

where W_d = load on rigid pipe
C_d = load coefficient
γ = unit weight of backfill
B = width of trench

The load coefficient is an exponential function of the coefficient of friction ($\mu = \tan\delta$) between the natural soil and the backfill and the coefficient of lateral earth pressure, K, as well as the depth of soil cover, H and the width of the trench, B. Both soil parameters were empirically derived by Marston and were found to vary with the types of soil and backfill.

The coefficient of friction was observed to vary from 0.3 to 0.5, which corresponds to values of the angle of friction between the backfill and the natural soil, δ, ranging from 17 to 27^o. Little variation was found in the lateral earth pressure coefficient, K, with observed values ranging from only 0.33 to 0.37. If K is taken to be equivalent to K_o, the lateral earth pressure coefficient at rest, and assuming that Jaky's expression for K_o applies, then these prescribed values would be typical for a cohesionless material with a friction angle of approximately 40^o. Interestingly, Marston's experiments included saturated clay, which produced the highest K value and the lowest μ.

Since the soil parameters, μ and K do not vary significantly, it can be said that the load coefficient is largely a function of the relative depth of cover above the pipe. The coefficient for a rigid pipe, C_d, is approximately 0.85 at H/B equal to 1 and increases to 1.5 for a H/B ratio of 2. Effectively, the proportion of backfill weight (γBH) felt by the pipe decreases from 85% to 75% as the cover height increases from one times the trench width to twice the trench width. At greater cover heights, the load on the pipe is more dependent on soil type. At heights of cover greater than 10B, C_d is almost constant.

The load on a flexible pipe can be approximated if the relative stiffness of the pipe to the soil fill at the side of the pipe is reasonably estimated. If it can be assumed that these stiffnesses are equal, then the load can be proportioned on the basis of area, i.e.

$$W_c = (W_d / B)\, D \qquad 2\text{-}2$$

where W_c = load on flexible pipe
D = outer pipe diameter

Marston's theory is a useful tool which is limited however, as it does not properly appreciate pipe-soil interaction or arching within the backfill for non-rigid pipes and is not readily amenable to variations in the properties of the backfill or natural soil over the depth of the trench. Sladen and Oswell (1988) suggested the chief limitations of Marston theory were firstly the simplifying assumptions concerning the geometry of the failure prism and the uniformity of vertical stresses within the prism, and secondly the lack of consideration of the stiffness of the backfill soil. Marston's theory was extended by Kellogg (1993) to include sloping trenches.

Molin (1981) found that the vertical soil pressure, w, above a pipe in an infinitely wide trench (e.g. under embankment fill) increased with the stiffness of the pipe and so proposed that the average pressure at crown level could be expressed by;

$$w = Cq_o \qquad 2\text{-}3$$

where q_o = pressure at crown level <u>without</u> a pipe
C = load factor (minimum value of 1), and is given by,

$$C = \frac{36S_r(20S_r + 1)}{(12S_r + 1)(36S_r + 1)} \qquad 2\text{-}4$$

and S_r = stiffness ratio = 8S/E′

where S = stiffness of pipe = EI/D^3

and E′ = horizontal modulus of soil reaction as defined by the Iowa equation (MPa) (refer section 2.7.4)

This equation for C is a design approximation of the theoretical cases of full slip between the pipe and the soil and no slip. Full slip gives rise to maximum C values or the greatest pressures above pipes (Crabb and Carder, 1985).

Molin's expressions have little influence on flexible pipes as a pipe stiffness, S, of 50 kPa is required to override the mininimum C value of unity, assuming a relatively low soil stiffness of 5 MPa. This pipe stiffness value exceeds common flexible pipe stiffnesses.

The German pipe design method in "Abwassertechnischen Vereinigung e.V."(ATV Code, 1984) allows calculation of pipe loads for all types of pipe installations and incorporates the effects of pipe stiffness and the variation of soil moduli in the vicinity of the pipe. The method is semi-empirical although the basis of the method is similar to Marston theory.

Jeyapalan and Hamida (1988) provided an overview of the German approach and showed that the Marston loads are always greater. Assuming that the German approach leads to the correct loads, Jeyapalan and Hamida concluded that even for relatively stiff, vitrified clay pipes (Marston is based on the assumption of a rigid pipe), Marston theory is particularly conservative "for small pipes backfilled with well-compacted granular material". Loads may be overestimated by 100%.

The general expression for the load on a pipe is:

$$W_{GDM} = C_d L \gamma B D \qquad 2\text{-}5$$

where W_{GDM} = load at the pipe crown calculated by the German design method

L = load re-distribution coefficient

C_d = Marston load coefficient

Coefficient, L, depends on soil moduli in the vicinity of the pipe, the ratio of stiffness of the pipe to the side fill and the geometry of the buried pipe installation.

2.2.2 The Influence of Live Loading on Backfill Surface

Pipes that have been buried at shallow depths will be subjected to the loads imparted by traffic. Traffic must include construction plant since, during construction, the pipe is most susceptible to damage; protection afforded by backfill cover height may be incomplete and overlying pavements may yet to be completed. After construction, pipelines underlying roads, railways or airport runways will experience live loading.

Traffic imparts a local loading, which has most impact when the traffic direction is transverse to the longitudinal axis of the pipeline. Pneumatic tyres which transmit axle loads have an almost elliptical footprint on a road surface. Pavement engineers approximate the footprint to a uniformly loaded, rectangular patch. An example of the pattern of loading from a transport vehicle (A14) used by design engineers is illustrated in Figure 2-1a (after NAASRA, 1976).

Simple load distributions have been promoted in the past for the purpose of pipe design, based on the assumption of elastic backfill behaviour. For example, the Standard, AS/NZS 2566.1 (1998), “Buried Flexible Pipelines, Part 1: Structural Design”, allows load spreading of concentrated road vehicle loads at a rate of 0.725 times the cover height. The surface patch plan dimensions increase by 1.45H at the pipe crown and accordingly the imparted vertical stress is much reduced. For this rate of load spreading, the 400 kPa surface loading from the dual wheels of the T44 vehicle reduces rapidly with depth below the surface, z, as indicated in Figure 2-1b. In this Figure, B is the surface footprint width, i.e. 200 mm.

More accurate appreciation of the influence of wheel loading can be realized through numerical analyses. For example, Fernando, Small and Carter (1996) employed a Fourier series to simulate uniform loading over a rectangular patch area and were able through this technique to achieve 3D simulation with a 2D finite element model of the soil and pipe. However the limitation of this approach is that the loaded soil must remain elastic.

2.3 THE NATURE OF BURIED PIPE INSTALLATIONS

Pipeline construction has certain characteristics leading to the formation of zones of soil of different strengths and stiffnesses within what is essentially a homogeneous backfill soil material. Compactive effort is restricted by the geometry of the trench and the sensitivity of the installed flexible pipe to compaction of material around it. Typical zones and the terminology used to describe these zones are given in Figure 2-2. The terminology of ASTM D2321-89 for the installation of thermoplastic pipe is adopted in this thesis.

The zones lead to the definition of the structural zone for a pipe and its backfill. The structural backfill extends from the base of the bedding to a maximum of 300 mm above the pipe. In this zone, granular material is strongly preferred over other soils for ease of compaction, high earth pressure response and stability when saturated and confined. For economic reasons other materials have been accepted for situations where loads are low to moderate (Molin, 1981, Janson and Molin, 1981). The bedding provides the vertical soil support.

The lateral support zone is unlikely to be uniform; many authors have commented on the difficulty in compacting underneath the pipe in the haunch zone and have subsequently suggested using crushed rock backfills which need little compaction (e.g. Webb, McGrath and Selig, 1996, Rogers, Fleming, Loeppky and Faragher, 1995, and Rogers, Fleming and Talby, 1996) and cementitious slurries.

Nevertheless, ASTM D2321-89 allows the use of plastic soils with liquid limits up to 50%.

The influence of the natural soil forming the trench walls on the lateral soil support has been addressed by Leonhardt, as cited by McGrath, Chambers and Sharff (1990). The effective sidefill stiffness is given by $\Omega E'$, where Ω is Leonhardt's correction factor on the the modulus of soil reaction, E', as defined in the Iowa formula (see section 2.7.4):

$$\Omega = \frac{1.662 + 0.639(B/D - 1)}{(B/D - 1) + [1.662 - 0.361(B/D - 1)]\dfrac{E'}{E_3}} \qquad 2\text{-}6$$

where E_3 = the Young's modulus of the natural soil forming the trench

B = the width of the trench

D = pipe diameter

When E' is much less than E_3, the trench walls are effectively rigid. If the ratio of trench width to pipe diameter is 2, then the effective modulus for pipe support is 2.3 times E'. As E' approaches the value of E_3, Ω is reduced as illustrated in Figure 2-3. Less influence is apparent for a wider trench and the correction factor may be ignored for a trench width to pipe diameter ratio of 5 or greater.

The final backfill material may or may not be the same as the pipe embedment zone material, depending on the economics of the construction which will be influenced chiefly by the trench geometry and the suitability of the excavated material.

2.4 PRIMARY MODEL OF BEHAVIOUR

The primary model for the design of flexible pipes is the ‘thin elastic ring’ (Prevost and Kienow, 1994). Solutions for the maximum moments and deflection of the ring for a variety of loading regimes are given in published stress tables in structural engineering textbooks.

A typical stress distribution that is assumed in the soil surrounding a buried pipe is shown in Figure 2-4. This assumed stress distribution is an attempt to include the effects of soil-structure interaction. The pipe is loaded at its crown by the backfill weight and traffic, if the burial is shallow. A uniform vertical pressure is assumed at crown level. The pressure at the crown is resisted partly by the soil reaction from the foundation or bedding for the pipe. Further support is afforded in a flexible pipe system by lateral backfill pressure, which is generated as the pipe deflects under the vertical load (refer Figure 2-5 for an illustration of pipe deflection). If the deflection of the sides of the pipe is considerable, earth pressures may approach passive pressure levels. The distribution of the side reaction is commonly assumed to be parabolic, but this is an arbitrary assumption, which may not follow necessarily from a rigorous study of the mechanics of the problem.

The level of lateral earth pressure response depends also on the nature of the backfill and its level of compaction, as well as the stiffness of the side walls of a trench (if the pipe sits in a trench rather than in an embankment fill). Therefore, it should be readily appreciated that the backfill and its construction are vital to the performance of a flexible pipe. Unfortunately designers have in the past placed too much attention on the structural properties of the pipe. Crabb and Carder (1985) demonstrated the importance of sidefill compaction in their experiments. Rogers et al. (1995), stated that soil stiffness rather than the stiffness of the pipe dominates the design of profile wall drainage pipe. McGrath, Chambers and Sharff (1990) supported this statement succintly by designating the design problem as “pipe-soil interaction” rather than “soil-structure interaction”.

Knowing the pressure distribution around the pipe, moments, stresses and deformations may be evaluated assuming "ring behaviour" applies, i.e. a loss in vertical diameter is compensated by an increase of the same magnitude in the horizontal diameter, such that the deformed shape is elliptical. Generally the maximum moment in the pipe is given by the expression (after Prevost and Kienow, 1994):

$$M = mWR \qquad 2\text{-}7$$

where W = transverse uniform load on ring's section at crown level
R = ring radius
m = moment coefficient based on ring theory

The transverse uniform load can be derived from Marston's theory, however the resultant load above the pipe may not be uniform and the inclusion of the effects of external live load in the load term, W, was considered by the authors to be "fraught with uncertainty".

Deflection of the pipe may be expressed as:

$$\Delta_i = \frac{Wd_r}{S} \qquad 2\text{-}8$$

where Δ_i = pipe deflection in direction, i.
W = total transverse load on ring at crown level
S = stiffness of pipe = EI/D^3
d_r = deflection coefficient in the direction being considered

The deflection coefficient varies with the direction being considered and the pressure on the pipe, e.g. for a uniform pressure of w on the pipe, the deflection in the x direction, Δx is given by;

$$\frac{\Delta x}{D} = \frac{wd_x}{S} \qquad 2\text{-}9$$

where w = uniform pressure on pipe crown $= \frac{W}{D}$

The pipe initially tends to deform as an ellipse as shown in Figure 2-5. Therefore it has been a common assumption that the lateral diametric increase of the pipe is equal to the vertical diametric reduction. Since the horizontal modulus of soil reaction, E′, is defined as the force per unit length along the pipe to cause a unit displacement, then the side thrust on the pipe may be expressed in terms of the lateral deformation. Subsequently, lateral deflections may be determined separately for the vertical and lateral load components using the ring equations available in structural texts, and the total lateral deflection is estimated by algebraic summation. This process leads to the Spangler or Iowa equation, viz.:

$$\frac{\Delta x}{D} = D_l K_s \left[\frac{w/S}{1 + 0.0076(E'/S)} \right] \qquad 2\text{-}10$$

where D_l = deflection lag factor

K_s = modified bedding constant

w = average pressure above the pipe crown

E' = horizontal modulus of soil reaction (MPa)

The deflection lag factor is unity for short term loading. For sustained loading, D_l increases with time due to consolidation effects arising from the lateral soil pressures developed beside the deflecting pipe (Howard, 1985).

Prevost and Kienow suggested that the bedding support angle, which is illustrated in Figure 2-2, could be taken to be 90° with little danger of significant error in determining moments and deflections, giving rise to a value of K_s of 0.012.

When stiffness of the pipe is expressed by the parameter pipe stiffness, PS, a property which may be derived experimentally, the following equation results:

$$\frac{\Delta x}{D} = D_1 K \left[\frac{w}{0.149PS + 0.061E'} \right] \qquad 2\text{-}11$$

where K = bedding constant (≈ 0.1 for a bedding angle of 90°)

PS = pipe stiffness $= \dfrac{S}{0.0186}$ (refer section 2.7.2)

In summary, the Iowa formula is based on three major limiting assumptions:

(1) The vertical deflection is equivalent to the horizontal deflection

(2) The deformation of the pipe is elliptical

(3) The horizontal modulus of soil reaction is constant for the backfill material.

The application of a horizontal modulus of soil reaction assumes that there is no soil support or soil stresses until deflections commence. However the placement of the pipe leads to in-situ soil stresses which effectively increase the lateral resistance available. In Sweden, an alternative expression to the Iowa equation has been used, which allows for an initial lateral resistance due to the at rest earth pressures in the backfill (Molin, 1981).

Watkins (1988) re-arranged the Iowa equation to express the ratio of pipe deflection to vertical soil strain above the pipe, ε' (= w/E'). Assuming the vertical and horizontal deflections at small strains are equal and that K is 0.1, the Iowa equation becomes;

$$\left(\frac{\Delta y}{D}\right)\left(\frac{1}{\varepsilon'}\right) = \frac{R_s}{80 + 0.61R_s} \qquad 2\text{-}12$$

where $R_s = \dfrac{E'}{S}$

Watkins argued that the pipe deflection can not exceed the soil deformation, so the left hand side of the equation, $\left(\frac{\Delta y}{D}\right)\left(\frac{1}{\varepsilon'}\right)$, should not exceed unity. However at values of ring stiffness ratios (R_s) greater than 200, this can occur. From extensive testing of flexible pipes, Watkins found an alternative, and more satisfactory, empirical expression:

$$\left(\frac{\Delta y}{D}\right)\left(\frac{1}{\varepsilon'}\right) = \frac{R_s}{30 + R_s} \qquad 2\text{-}13$$

A visual comparison of equations 2-12 and 2-13 is provided in Figure 2-7. The comparison suggests that the Iowa equation overestimates delections of more flexible pipes (higher R_s values) and may tend to slightly underestimate the deflections of less flexible pipes

The assumption of elliptical pipe deformations in the above equations has been found to be reasonable at relatively small deflection levels only. The deflection estimates from these equations generally become non-conservative as strains increase (Howard, 1985, Cameron, 1990 and Sargand, Masada and Hurd, 1996). Rogers (1987) found that elliptical deformations were associated only with poor sidefill or surround support.

Valsangkar and Britto (1978) tested the applicability of ring compression theory for flexible pipes buried in trenches, largely through centrifuge tests. If ring theory is applicable then membrane compression stresses should dominate and flexural stresses should be insignificant. The study concluded that for pipes in narrow trenches, where the side cover is less than or equal to one diameter, the use of simple ring theory could not be justified. Therefore the Iowa equation should not be applied for ratios of trench width to pipe diameter (B/D) of 2 or less.

2.5 OTHER DESIGN APPROACHES AND CONSIDERATIONS

Moore (1993) set out the design considerations for plastic pipe enveloped in uniform soil. He recommended determination of the vertical pressure above the pipe, p_v, in a deep embankment, simply by summation of the product of unit weights of layers above the crown by their thickness. To determine the same pressure for a pipe installed in a trench, the coefficient of friction between the trench wall and backfill, μ, was needed to apply simple arching theory as follows:

$$p_v = \gamma B\left[\frac{1-e^{-2K_o\mu H/B}}{2K_o\mu}\right] \qquad \text{2-14}$$

where H and B are the depth and width of the trench, respectively. Equation 2-14 assumes a uniform backfill material.

The horizontal pressure, p_h, beside the pipe was based on the "at rest" coefficient of earth pressure, K_o, and p_v. The pressures in the soil were determined on planes sufficiently far away from the pipe, which was suggested to be a minimum of one pipe diameter from the circumference (refer Hoeg 1968).

Moore converted the vertical and horizontal pressures immediately surrounding the pipe, to isotropic and deviatoric stress components, defined as $p_m = (p_v + p_h)/2$ and $p_d = (p_v - p_h)/2$, respectively. These pressures are illustrated in Figure 2-8. Uniform circumferential hoop stress arises under isotropic loading, which will cause circumferential shortening and may lead also to significant flexural stresses. Designs must provide adequate strength and stiffness to resist these stresses. The deviatoric stress set results in elliptical deformation. From this combination of deformations it can be seen that the vertical diametric strain should exceed the horizontal diametric strain, provided p_v is greater than p_h.

Moore provided equations for the stresses, thrusts and pipe deflections based on elastic behaviour and thin ring theory. The radial stress on the pipe for the isotropic loading is given by:

$$\sigma_o = A_m p_m \qquad 2\text{-}15$$

where p_m = the mean stress, $(p_v + p_h)/2$

and A_m = an arching coefficient

$$= \frac{2(1-\nu_s)E_p A}{E_p A + 2G_s r}$$

with ν_s = Poisson's ratio for the soil

G_s = shear modulus of the soil

r = radius of the pipe

A = cross sectional area of the pipe

E_p = the elastic modulus of the pipe

If A_m for the pipe-soil system is less than unity, the pipe is regarded as flexible and positive arching can occur.

Hoeg 's (1968) equivalent expression for the arching coefficient for points on the pipe circumference was as follows:

$$A_m = (1 - a_1) = \left[1 - \frac{(1-2\nu_s)(C-1)}{(1-2\nu_s)C+1}\right] \qquad 2\text{-}16$$

where C = compressibility ratio

= "compressibility of the structural cylinder relative to that of a solid soil cylinder"

The compressibility ratio was defined by the equation:

$$C = \left(\frac{1}{2(1-\nu_s)} \right) \left(\frac{M_s (1-\nu_p^2)}{E_p} \right) \left(\frac{D}{t} \right) \qquad 2\text{-}17$$

where M_s = the constrained or 1D modulus of the soil
E_p = Young's modulus of the pipe material
ν_p = Poisson's ratio of the pipe material
D = the average pipe diameter
t = the pipe thickness

The deviatoric component of the pressures surrounding the pipe, p_d, causes further radial stress in the pipe, $\sigma_{rd\theta}$, as well as shear stress, $\tau_{d\theta}$. Both pipe stresses are functions of p_d and position along the pipe circumference, as given by the angle, θ, which is defined in Figure 2-8. The expressions for the radial and shear stresses are:

$$\sigma_{rd\theta} = A_{d\sigma} p_d \cos 2\theta \qquad 2\text{-}18$$

and

$$\tau_{d\theta} = A_{dr} p_d \sin 2\theta \qquad 2\text{-}19$$

where p_d = deviatoric stress = $(p_v - p_h)/2$

In the equations above, $A_{d\sigma}$ and A_{dr} are functions of the relative stiffness of the pipe to the surrounding soil as well as the bond developed between the pipe and the soil. Hoeg (1968) provided theoretical solutions for these two factors, for both a smooth and a rough soil-pipe interface. The expressions for these two factors for a perfectly rough interface, and along the pipe circumference, are provided in the following equations:

$$A_{d\sigma} = -(1 - 3a_2 - 4a_3) \qquad 2\text{-}20$$

$$A_{dr} = (1 + 3a_2 + 2a_3) \qquad 2\text{-}21$$

where;

$$a_2 = \frac{(1-2\nu_s)(1-C)F - 0.5(1-2\nu_s)^2 C + 2}{[(3-2\nu_s)+(1-2\nu_s)C]F + (2.5 - 8\nu_s + 6\nu_s^{\,2})C + 6 - 8\nu_s}$$

$$a_3 = \frac{[1+(1-2\nu_s)C]F - 0.5(1-2\nu_s)C - 2}{[(3-2\nu_s)+(1-2\nu_s)C]F + (2.5 - 8\nu_s + 6\nu_s^{\,2})C + 6 - 8\nu_s}$$

In the equations above, F is the flexibility factor which "relates the flexibility of the structural cylinder to the compressibility of a solid soil cylinder". Hoeg (1968) defined factor F with the equation:

$$F = \left(\frac{(1-2\nu_s)}{4(1-\nu_s)}\right)\left(\frac{M_s(1-\nu_p^2)}{E_p}\right)\left(\frac{D}{t}\right)^3 \qquad 2\text{-}22$$

The maximum radial stress, σ_{rd}, occurs at the pipe springline and the minimum stress is located at both the crown and the base of the pipe. The maximum shear stress, τ_d, occurs at the crown and the base of the pipeline, while the minimum shear occurs at the pipe springline.

The thrusts and moments arising from the pipe stresses for the deviatoric and isotropic external stress sets depend upon the position of the point under consideration on the pipe circumference. Of particular interest are the thrusts at the crown and the springline, N_{crown} and N_{spring}, and the corresponding moments. These moments and thrusts can be derived from thin shell theory:

$$N_{crown} = \sigma_o r + \left[\frac{\sigma_{rd}}{3} - \frac{2\tau_d}{3}\right] r \qquad 2\text{-}23$$

$$N_{spring} = \sigma_o r - \left[\frac{\sigma_{rd}}{3} - \frac{2\tau_d}{3}\right] r \qquad 2\text{-}24$$

$$M_{crown} = \left[\frac{\sigma_{rd}}{3} - \frac{\tau_d}{6}\right] r^2 \qquad 2\text{-}25$$

$$M_{spring} = -\left[\frac{\sigma_{rd}}{3} - \frac{\tau_d}{6}\right] r^2 \qquad 2\text{-}26$$

Deflections may be determined from the pipe stresses by considering the components of the external stresses, w_o, due to the isotropic loading and $w_{d\theta}$, due to the deviatoric loading as follows;

$$w_o = \frac{\sigma_o r^2}{EA} \qquad 2\text{-}27$$

$$w_{d\theta} = \left[\frac{(2\sigma_{rd} - \tau_d) r^4}{18EI}\right] \cos 2\theta = w_{d\,max} \cos 2\theta \qquad 2\text{-}28$$

$$w_\theta = \left(w_o + w_{d\,max} \cos 2\theta\right) \qquad 2\text{-}29$$

The changes in diameter of the pipe in the vertical and horizontal directions, ΔD_V and ΔD_H respectively, may then be formulated as;

$$\Delta D_V = 2\left(w_o - w_{d\,max}\right) \qquad 2\text{-}30$$

$$\Delta D_H = 2\left(w_o + w_{d\,max}\right) \qquad 2\text{-}31$$

Moore (1993) demonstrated with a case example that this theory provided far superior predictions of deflections than those produced by the Iowa equation and gave estimates of radial stresses, which reasonably matched those measured.

Webb, McGrath and Selig (1996) emphasized the importance of hoop stiffness on pipe performance. Low hoop stiffness permits pipe deformation, which aids the shedding of load to the surrounding soil. Hoop stiffness is defined as:

$$H = \frac{2E_p A}{D} \qquad 2\text{-}32$$

where A = cross sectional area of the pipe wall per unit length

E_p = elastic tensile modulus of the pipe material

The mean circumferential hoop contraction stress, σ_H, is related to hoop stiffness by the equation:

$$\sigma_H = \varepsilon_H H \qquad 2\text{-}33$$

where ε_H = hoop contraction strain
= average of the strains of the inner and outer walls at a point on the pipe

The deviatoric element of the pressure about the pipe causes non-uniform hoop thrust and thus differential strain, across the pipe section. If the pipe has low cross-sectional stiffness and the differential strain is high, buckling failure may occur.

Moore (1993) provided recommendations for estimating the critical thrust to initiate buckling. A simple but conservative estimate was given as:

$$N_{cr} = \frac{12E_pI}{D^2} \qquad 2\text{-}34$$

2.6 PIPE DEFORMATIONS DURING BACKFILLING

Compaction effort in the lateral support zone must be limited to protect the pipe. Compaction of the side backfill leads to greater soil support but can distort and uplift a flexible pipe before it is loaded. Furthermore, if the soil support is such that the pipe can no longer deform laterally when it is loaded vertically, the risk of buckling or overstressing the pipe wall in the vicinity of the crown is greatly increased. As suggested by Webb, McGrath and Selig (1996), the initial pipe deflection is beneficial, provided it is not excessive. The initial deformation is solely due to the pipe stiffness or lack of it, while subsequent deformations depend more on the sidefill stiffness.

Zoladz, McGrath and Selig (1996) witnessed pipe uplift in laboratory trials simulating trench conditions with hard trench walls. Rogers, Fleming and Talby (1996) reported similar experiences but also pointed out that the raising of the side backfill should be conducted simultaneously on both sides of the pipe to avoid non-symmetric distortion of the pipe.

Cameron (1990 and 1991) conducted a series of laboratory trench tests in a soil box with braced walls, using a poorly graded sand and spirally-wound, profiled uPVC pipes ranging from 250 to 525 mm diameter. Pipe stiffnesses (S) varied from 0.9 to 2.0 kPa. Cameron found that the average vertical diametric strain of the pipe after completion of the backfill could be related to the final cover height, divided by the nominal pipe diameter and the density index (%) of the sidefill (see Figure 2-9). Initial vertical expansion of the pipe is countered by subsequent backfilling and compaction to complete the design cover height. For the range of cover heights, pipes and compaction levels in the study, Cameron found that installation caused verical 'diametric strains', generally ranging between ± 1%.

2.7 SPECIFIC MATERIAL AND DESIGN CONSIDERATIONS

2.7.1 Deflection Criteria

Deflections are expressed as 'strains' relative to the internal diameter of the pipe i.e. 'diametric strains'. Vertical strains dominate design considerations.

Considerable debate has occurred over what should be the acceptable deflection limits for flexible pipes. Entwined in this debate is the short and long term (loading) stiffness of the pipe material and the relevance of these values to the deflection limits. Numerous authors have reported that pipes have been distorted by 10 to 20% and still continue to perform adequately. Rogers et al (1995) suggested that a deflection limit of 5% represents a factor of safely of 4 for steel corrugated culverts which might buckle at 20% strain. They recommended a 5% deflection limit for the construction period only.

Janson and Molin (1981) proposed to design in the short term for 5%, with an acceptance of 'local exceedances' to 7.5%. Over the long term, diametric strains which consider creep should be designed to be less than 15%. Petroff (1984) advocated a value of 7.5% for "overall stability", while recognizing that 20% was in certain circumstances acceptable.

Schluter and Shade (1999) suggested that a 7.5% limit represented a quarter of the deflection at which collapse or curvature reversal was commonly believed to occur, thereby providing a generous safety margin. However, the same authors found that reverse curvature could be induced in HDPE pipes (high density polyethylene) at diametric strains less than 30% and as low as 22%. Pipes constructed of uPVC went into reverse curvature after 30% strain.

2.7.2 Pipe Material Design Parameters

Pipe stiffness is usually evaluated by testing a length of pipe in a parallel plate compression test (eg. ASTM D2412-96a). This test overcomes uncertainties in material properties and the difficult analysis of unusual pipe wall configurations, such as spirally-wound, profiled pipe. The test provides the short term pipe stiffness, as a testing rate of 12.5 ± 0.5 mm of deflection per minute is specified. Pipe stiffness terms may be derived from the test as follows;

$$PS = \frac{F}{L\Delta y} = \frac{EI}{0.149R^3} = \frac{EI}{0.0186D^3} \qquad 2\text{-}35$$

and so

$$S = \frac{EI}{D^3} = 0.0186(PS) \qquad 2\text{-}36$$

where F = load applied at the crown of the pipe

L = length of pipe

and Δy = the vertical deflection of the pipe

The rate of loading in the parallel plate test obviously does not relate to long term loading of a buried pipe. A constant rate of loading is specified as thermoplastics are known to experience stress relaxation after being loaded. In other words plastics shed load over time and the apparent modulus is reduced as stress is reduced while the deformation remains constant. Schluter and Shade (1999) reported that the increase of modulus for an increase by a factor of 100 of the rate of loading was only 6.5% for uPVC, but was 56% for HDPE.

Some researchers (e.g. Rogers et al., 1995) have questioned the applicability of pipe stiffness determined by this method to the design of buried pipe installations. Joekes and Elzink (1985) pointed out the variations posssible in evaluating stiffness with respect to time. The ASTM parallel plate compression test takes minutes to perform, while in Germany and Belgium, tests to evaluate short term stiffness were performed over 24 hours. From observations of deflections with time of a 30 kilometer length of PVC sewer pipe in Europe, Joekes and Elzink concluded that the long term stiffness of pipes should be based on 42 day tests, extrapolated to a two year stiffness. However they acknowledged that for good installations with well compacted granular backfill material, one month may be appropriate.

The parallel plate test as specified by ASTM is conducted at a temperature of 23°C. It is well recognised that the engineering properties of thermoplastics are sensitive to temperature. Schluter and Shade (1999) found that HDPE was far more sensitive to temperature than uPVC. Parallel plate testing between temperatures of 24 and 60°C produced losses of stiffness of 22% and 122% for uPVC and HDPE, respectively.

When applying the Iowa formula to estimate deflections, it would seem that backfill stiffness is considerably more important in controlling pipe deformations, to the extent that little if any pipe stiffness is required. Jeyapalan and Abdelmagid (1984) warned designers that a minimum pipe stiffness is indeed necessary to ensure the pipe deforms essentially as an ellipse, as assumed by the pipe deflection formulae. The pipe must be able to carry part of the load and promote arching in the soil.

Therefore sufficient stiffness is needed to avoid both collapse of the crown when the sidefill material is soft or loose and rectangularization of the cross-section when the sidefill material is dense (Rogers, 1987 and Howard, 1985).

2.7.3 Profile Wall Pipes

Sargand, Masada and Hurd (1996), studied the influence of seven wall profiles on the performance of plastic (PVC and HDPE) pipes, ranging in diameter from 450 to 900 mm. Both laboratory and field tests were conducted. Soil pressures were monitored. The authors warned that certain combinations of pipe profiles and backfill materials could result in poor compaction of the soil in the vicinity of the pipe, resulting in excessive pressure on the exposed ribs.

It was concluded that the ratio of the average spacing of the ribs to the maximum particle size of the backfill soil should be less than 0.6 or greater than 2.6, so that particles could either fit between the ribs or bridge against a few ribs. Particle size distributions of the backfill material were not discussed.

2.7.4 Modulus of Soil Reaction or Bulk Soil Modulus

Estimates of pipe deflections are sensitive to E′. In the Iowa formula, Spangler defined E′ as the product of the modulus of passive resistance of soil at the side of the pipe, e, (units of pressure/length) by the radius of the pipe, R. As recognized by Spangler, the maximum soil stiffness at the *passive* soil condition can only be achieved at high levels of soil strain, which in practice may not be reached. In essence, the Spangler soil modulus is tied to the circumferential strain of the pipe and is not the usual definition of soil modulus, i.e.:

$$E' = \frac{p}{\varepsilon_\theta} \qquad \text{2-37}$$

where p = pressure against the pipe from the side soil

ε_θ = circumferential pipe strain = $\Delta x/R$ for axial symmetry

From backanalyis of culverts, Spangler gave typical values of E′ for sands, ranging between 2.4 and 8.3 MPa.

Watkins and Spangler (1958), as cited by Singh and Pal (1990), suggested that E′ values are reasonably constant for a given soil and state of compaction. However they recognised that the pipe size also had an influence, presumably because of the difference in soil side strains in the one installation for different pipes, leading to different levels of soil resistance.

McGrath, Chambers and Sharff (1990) rightly stated that the semi-empirical modulus of soil reaction, E′, was not a true material property and that there was no one practical test available to evaluate it. However, they, along with Watkins (1988), proposed that E′ could be approximated by the constrained soil modulus of a vertically loaded soil in a consolidometer. The constrained or one-dimensional modulus of soil, M, may be defined as:

$$M = \frac{\sigma'_v}{\varepsilon_v} = \frac{E_s(1-\nu_s)}{(1+\nu_s)(1-2\nu_s)} \qquad 2\text{-}38$$

where σ'_v = effective vertical stress

ε_v = vertical soil strain

E_s = Young's modulus of the soil

and ν_s = Poisson's ratio of the soil

The secant modulus over the stress range was advocated by both groups of researcher to define the average sidefill stiffness. The use of the constrained modulus does incorporate the non-linear behaviour of the soil under load.

Howard (1977) reported typical values of modulus of soil reaction, which varied with. soil type and composition, as well as the degree of soil compaction (refer Table 2-I). The moduli were deduced from laboratory tests by the US Bureau af reclamation on a range of pipe diameters and materials, and were complemented by field test data. For clean sands (less than 12% fines), the reported E′ values ranged between 6.9 and 20.7 MPa with level of compaction varying from “slight” to “high”.

Jeyapalan and Abdelmagid (1984) pointed out to pipe designers the need to evaluate soil modulus in the conventional soil mechanics way, giving careful consideration to the influence of the levels of both stresses and strains. In a backfilled trench, stresses in both directions will be influenced by stresses developed during compaction of the backfill, arching of the backfill above the pipe, external surface loads and the associated pipe deformations. The problem is further complicated by the fact that in narrow trench conditions, the natural soil will contribute to E′, as stresses from pipe deformations can extend to a distance of at least two and one-half diameters from the pipe wall [Barnard (1957)]. Nevertheless approximations to elastic behaviour may be made within small ranges of external stress.

Moore (2001) reported on research in the USA, which lead to a revised outlook on E' and its replacement with Young’s modulus, E_s, which was recognized to vary with the level of vertical stress. Since the suggested values of E_s varied also with dry density ratio, the correspondence between density index and dry density ratio from Table 2-I has been used in compiling Table 2-II of the E_s values reported by Moore (2001) for sands and gravels.

Assuming a Poisson’s ratio of 0.25 for the soil, the constrained modulus, M, may be evaluated according to equation 2-38. The resulting values have been provided in Table 2-III. It is evident that for moderate levels of vertical stress, the constrained modulus is expected to range between approximately 12 and 25 MPa, for moderately dense sand (I_D between 55 and 70%).

2.7.5 Design Charts and Current Design Approaches

Gumbel and Wilson (1981) considered the earth pressures at the depth of the pipe installation without the pipe being present. Vertical and horizontal earth pressures, p_v and p_h, respectively, were divided into isotropic and deviatoric (“distortional”) stresses, p_z (or p_m) and p_y (or p_d), similar to Moore’s later work (1993). The pipe was then imposed on the soil. The effects of the two components of the soil pressure on the pipe were considered separately, using a plane strain, elastic analysis.

A pipe subjected to isotropic compressive stress, p_z, experiences uniform hoop thrust and uniform contractive deflection, N_z and δ_z, respectively. The deviatoric components of the earth pressure, vertical compressive stress, p_y, and horizontal tensile stress, $-p_y$, produce positive and negative hoop thrusts and deflections, N_y and δ_y, which vary around the pipe. The pattern of deflection is elliptical. Maximum hoop thrust and expansion occur at the springline.

Additional deviatoric values of thrust and deflection occur at large pipe deflections, which are due to the secondary effect of isotropic pressure loading on a non-circular pipe. In other words, the superposition of the effects of the isotropic and deviatoric components of earth pressure is not valid at large deflections as the isotropic pressure is acting on a pipe which is increasingly becoming less circular. Accordingly Gumbel and Wilson made corrections to N_y and δ_y.

The extent of the uniform hoop thrust component carried by the pipe is affected by arching within the backfill soil. An elastic arching factor, α, was introduced, such that αp_z represented the amount of pressure carried by the pipe in ring compression. This factor varies between zero and unity.

Gumbel and Wilson defined a relative soil stiffness parameter, Y, which was the ratio of the plane strain soil modulus, E_s*, to the pipe stiffness, S, rather than the modulus of soil reaction, E′. The plane strain soil modulus was defined by:

$$E_s^* = \frac{E_s}{\left(1 - \nu_s^2\right)} \qquad 2\text{-}39$$

For a constant soil stiffness, as the pipe stiffness increases, Y decreases, and the pipe takes more of the vertical earth pressure, i.e. less arching occurs. Gumbel and Wilson proposed that α depended largely on parameter Y.

Intuitively it would seem more reasonable to impose an arching factor on the distortional component so that as the vertical pressure is reduced, the horizontal pressure is increased.

Crabb and Carder (1985) suggested that for design purposes it was reasonable to assume no arching, although it was acknowledged that arching could be significant for flexible pipe. For example, at $Y > 10^3$ for a pipe with a diameter to wall thickness ratio of 20, and $Y > 10^5$ for a ratio of 500, arching will be significant.

Design charts were constructed by Gumbel and Wilson for a range of values of $[p_y/(\alpha p_z)]$, a ratio which represented the level of uniformity of the pressures acting on the pipe. Each chart gave deflection estimates and buckling limits for various levels of relative soil/pipe stiffness. An example of a design chart is provided in Figure 2-10 for $p_y/(\alpha p_z)$ of 0.6, which Gumbel and Wilson believed represented a reasonable upper limit to expected levels of distortional loading.

For a given value of the stiffness ratio, Y, the deflection of a pipe varies almost linearly with the level of the distortional load on the pipe. The secondary effects mentioned previously cause the departure from linearity at high levels of deflection. The solid lines which cross the pipe deflection lines and which are labelled, F = 1, 2 or 3, represent load limits for buckling for safety factors of 1, 2 or 3. Buckling is an important consideration at high values of Y, i.e. for relatively low pipe stiffness.

Gumbel and Wilson also recognised "ring strength" as a design consideration, however it was considered to be unlikely to be important for most flexible pipes, which are capable of sustaining large strains. Ring strength is concerned with yield of the pipe material due to the action of hoop and flexural stress.

The analysis indicated that for less uniform earth pressure (high values of $p_y/(\alpha p_z)$), deflection considerations were more important than buckling. Buckling was likely to occur at relatively less deflection for more uniformly loaded pipes. Furthermore, buckling became a more dominant design factor as the relative soil/pipe stiffness (Y) increased above 10^3.

Crabb and Carder (1985) verified the approach of Gumbel and Wilson through experiments in a test pit on instrumented pipes of various materials, which were 300 mm in diameter. They found however that the proposed buckling limits were conservative.

According to Dhar, Moore and McGrath (2002), AASHTO have adopted the Iowa equation (equation 2-11) for flexural pipe deflections and also a pipe deflection term which accounts for shortening due to the developed hoop forces. The two pipe deflections are summed to give the vertical deflection, while the lateral deflection is the hoop force deflection less the deflection due to bending. The Iowa equation as used by AASHTO has been re-arranged into the following form:

$$\varepsilon_y = \left[\frac{D_l K w}{(EI)_p / R^3 + 0.061M} \right] \qquad 2\text{-}40$$

where $\varepsilon_y = \Delta y/(2R)$ = vertical diametric strain (deflection),

Δy = vertical pipe deflection

R = pipe radius

w = the pressure above the pipe crown (assumed uniform)

M = one dimensional soil modulus (refer equation 2-38)

$(EI)_p$ = flexural stiffness of the pipe

It is of interest that the constrained modulus of the soil has replaced Spangler's modulus of soil reaction.

The average deflection of the pipe due to hoop force includes an arching term which expresses how much load reaches the pipe. The overall expression for the deflection or strain is:

$$\varepsilon_H = \frac{F_{VA}\, w}{\left((EA)_p \big/ R + 0.57M \right)} \qquad 2\text{-}41$$

where, ε_H = average diametric pipe strain due to hoop force

$(EA)_p$ is the cross-sectional stiffness of the pipe

F_{VA} is a vertical arching factor, given as:

$$F_{VA} = 0.76 - 0.71\left(\frac{S_H - 1.17}{S_H + 2.92} \right) \qquad 2\text{-}42$$

where, S_H = a hoop stiffness factor = $(MR)/(EA)_p$

Equations 2-40 and 2-41 appear to be incongruous as the load reaching the pipe is different – an arching factor is not applied in the first equation. Another apparent failing of this approach is the legitimacy of the summation of deflections as the hoop deflections should be controlled by the isotropic pressure around the pipe, while the flexural deflections are generated by the deviatoric pressure (see Figure 2-8). Nonetheless, Dhar et al. (2002) reported good correspondence between three laboratory pipe tests under simulated embankment loading, and the AASHTO approach.

Design charts for buried flexible pipes subjected to vehicular loading have been established by Katona (1990), based on his finite element program CANDE. Safe cover heights were given for corrugated HDPE pipes for a range of standard (AASHTO) truck loadings. Katona analysed the responses of six pipe geometries, when buried with a soil cover of 0.3, 0.6 and 0.9 m. The level of compaction of the backfill material and hence the soil stiffness was a further variable in the analysis. The influence of a stiff but thin pavement (150 mm) was considered also.

The features and assumptions of Katona's analysis were:

- The pipe was represented by linear elastic beam-column elements
- Short term stiffness of the pipe material was adopted

- Non-linear soil stiffness (hyperbolic model)
- Trench construction was not considered (uniform soil envelope)
- Fully rough interface between pipe and soil
- Soil self weight included
- Plane strain analysis with approximated wheel loads (using Holl's 1940 solution)

Traffic was assumed to pass across the longitudinal axis of the pipe. Only single vehicle passes were considered, appropriate to construction traffic.

The criteria for design included a factor of safety of two for both the short term hoop strength and the critical buckling pressure. A deflection limit was imposed of 7.5% of the internal diameter of the pipe, while flexural strains were limited to 5%.

A good, slightly conservative comparison was achieved with limited experimental data from AASHTO H20 loading of a 600 mm diameter pipe in sandy, silty clay. Load levels were varied in the experiment and the influence of fair and good backfill compaction was obtained.

Within the bounds of the analyses, it was found that deflection generally controlled the design. Pipe stiffness was seen to be important in helping to control deflection, strain and buckling, while the cross-sectional area of the pipe was successful in controlling hoop stress. With respect to the installation, it was shown that both the depth of cover and the level of backfill compaction were essential considerations for the pipe design. If either the quality of compaction or the depth of cover is increased, the design criteria are more easily met.

Figures 2-11 to 2-15 have been reproduced from Katona (1990), which serve to indicate the basis of his more conservative design table for safe cover heights. The design parameters on the vertical scales have been normalised so that the design criteria is breached at values greater than unity. Figure 2-15 provides further design data from Katona for pipes protected by a thin pavement. It is evident from this

Figure that a thin but stiff pavement layer causes a major reduction in design parameters and hence in the required cover height.

2.8 SUMMARY OF THE CHAPTER

The methods available for design have sprung from elastic thin ring theory. It has been shown in this Chapter that the long standing Iowa equation is inadequate for calculation of pipe deflections. The work of both Gumbel and Wilson (1981) and Moore (1993) have promoted more rational design approaches, with appreciation of stresses and strains within the pipe. These approaches have tackled industry concerns for flexible pipe over observed non-elliptical deformations and the threat of pipe buckling. Moreover they have challenged the concept of soil reaction modulus promulgated by the Iowa equation, in favour of rational elastic theory and associated material properties.

The availability of Finite Element Analyses (FEA) has seen numerous recent attempts to understand the soil-pipe interaction model. Although it was an early attempt, program CANDE (Katona, 1990) was particularly notable, as it was purpose built for this particular interaction problem. Limiting aspects of CANDE were the assumption of a uniform soil envelope and the approximation of a 3D loading pattern in a 2D finite element analysis. This thesis extends the finite element modelling of soil-pipe interaction to take account of these two aspects.

In summary, the design of pipes subjected to external loading suffers from a number of uncertainties, as follows:

a) The amount of load that reaches the pipe in a deep pipe embedment

b) The amount of load that reaches the pipe due to trafficking of a backfilled surface, paved or non-paved

c) The variations of density that can occur in the various zones about a pipe installation

d) The contribution of the natural soil in a trench installation to the stiffness of the surround soil

e) Deformations of the pipe prior to loading due to the backfilling process

f) Appropriate material properties for thermoplastics for the rate of loading and the temperature

g) The applicability of pipe stiffness tests to pipe installations

h) The bond that exists between the pipe and the surrounding soil

i) Acceptable deflection criteria for pipes

The remainder of this thesis is not concerned with items a), e), h) and i), as the particular problem that is addressed is the live loading of profiled pipes in shallow trenches. The ribbed profile of the pipes could be expected to provide a rough interface with the granular soil in the installation, therefore item h) was not explored further. The initial deformations of the pipe due to installation were recognized as being of some importance, however the investigation of this effect (item e) will have to await further research by others. With respect to item f), the influence of temperature on the properties of the pipe material was not investigated.

The next Chapter reviews constitutive modelling of soils for incorporation into finite element analyses (FEA) of soil-pipe interaction.

2.9 REFERENCES TO THE CHAPTER

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TABLE 2-I. Typical values of E′ (MPa) after Howard (1977)

USCS[1]	***Degree of Compaction***	***dumped***	***slight***	***moderate***	***high***
	Level of Standard Compaction		**R_D[2] < 85%**	**R_D = 85-95%**	**R_D > 95%**
	Density Index		**I_D[3] < 40%**	**I_D = 40-70%**	**I_D > 70%**
	Coarse/Fines				
CH, MH or CH-MH (LL[4] > 50%)	< 25% coarse	0.3	1.4	2.8	6.9
CL, ML or CL-ML (LL < 50%)	> 25% coarse	0.7	2.8	6.9	13.8
GM, GC, SM, SC	> 12% fines	0.7	2.8	6.9	13.8
GW, GP, SW, SP	< 12% fines	1.4	6.9	13.8	20.7
crushed rock		6.9	20.7	20.7	20.7

[1] UCSC = "Unified Soil Classification Symbol"
[2] R_D = dry density ratio = ratio of target dry density to maximum dry density for the compactive effort
[3] I_D = density index (%) for a clean granular (coarse) material
[4] LL = liquid limit

TABLE 2-II. Typical values of Young's modulus, E_s (MPa), after Moore (2001)

Density Index I_D[5]	Dry Density Ratio, R_D	Vertical Stress Level (kPa)					
		7	**35**	**70**	**140**	**275**	**410**
40%	85%	3.2	3.6	3.9	4.5	5.7	6.9
55%	90%	8.8	10.3	11.2	12.4	14.5	17.2
70%	95%	13.8	17.9	20.7	23.8	29.3	34.5

TABLE 2-III. Typical values of constrained modulus, M[6] (MPa), after Moore (2001)

Density Index I_D[7]	Dry Density Ratio, R_D	Vertical Stress Level (kPa)					
		7	**35**	**70**	**140**	**275**	**410**
40%	85%	3.8	4.3	4.7	5.4	6.8	8.3
55%	90%	10.6	12.4	13.4	14.9	17.4	20.6
70%	95%	16.6	21.5	24.8	28.6	35.2	41.4

[5] Inferred values of density index from Table 2-I
[6] Assumes a Poisson's ratio of 0.25
[7] Inferred values of density index from Table 2-I

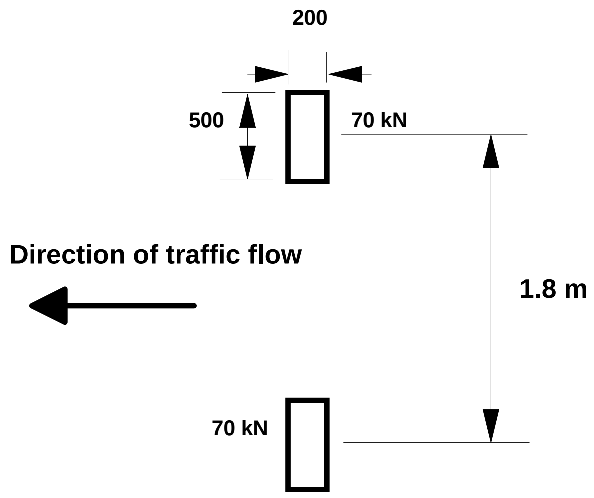


Figure 2-1a. Design Transport Vehicle Loading, A14, rear axle loading (NAASRA-1976)

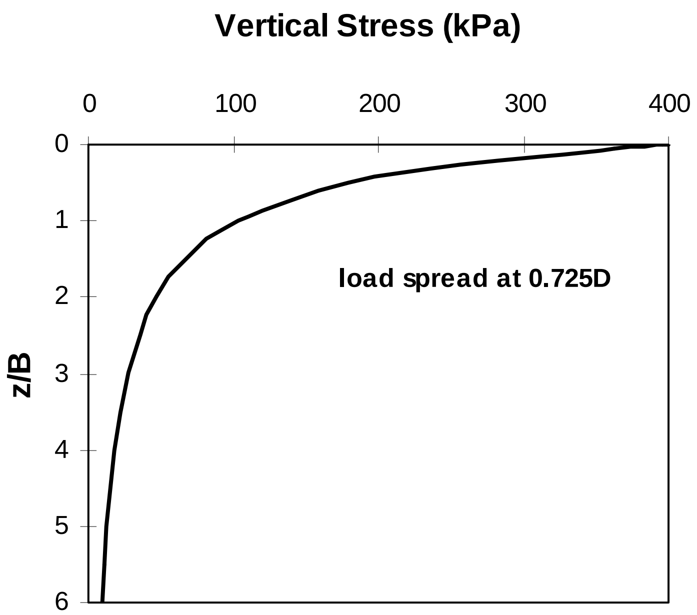
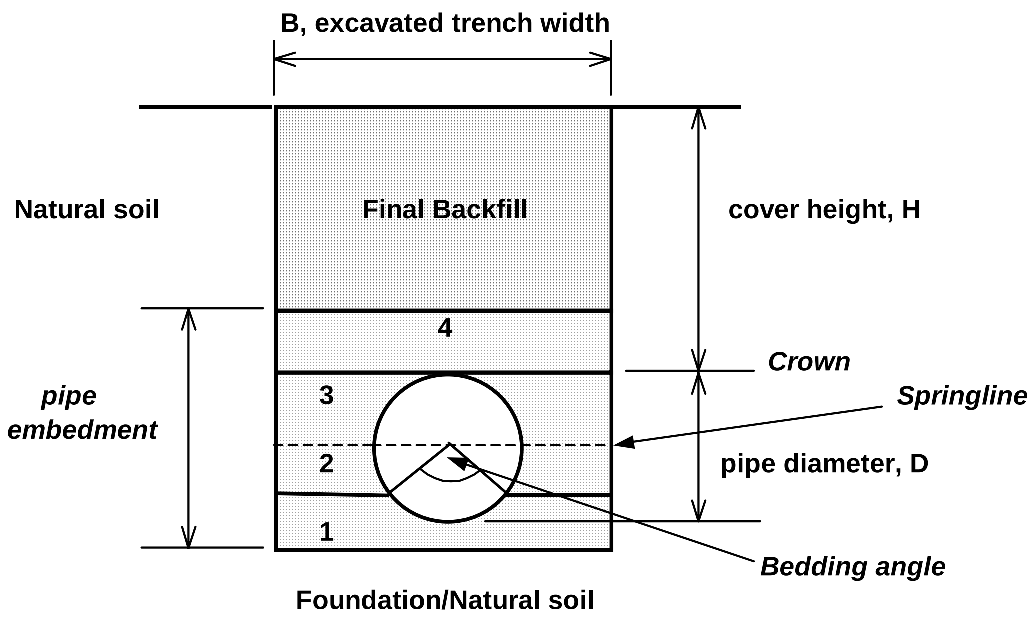


Figure 2-1b. Assumed reduction in T44 vertical pressure with depth according to AS3725-1989

Figure 2-1. Road vehicle loading patterns



Zone	Description	Level of Compaction
1	Bedding	Well compacted and usually shaped to receive the pipe to distribute the load support.
2	Haunching	Poorly compacted, particularly in a narrow trench. Rodded or tamped at best.
3 and 4	"Initial Backfill"	Moderately compacted.
3	Springline to crown	Moderately compacted by rodding or tamping.
4	Approx 150 to 300 mm above the pipe	Moderately compacted by rodding or tamping or a few passes of a vibrating plate.

Figure 2-2. The zones within a backfill

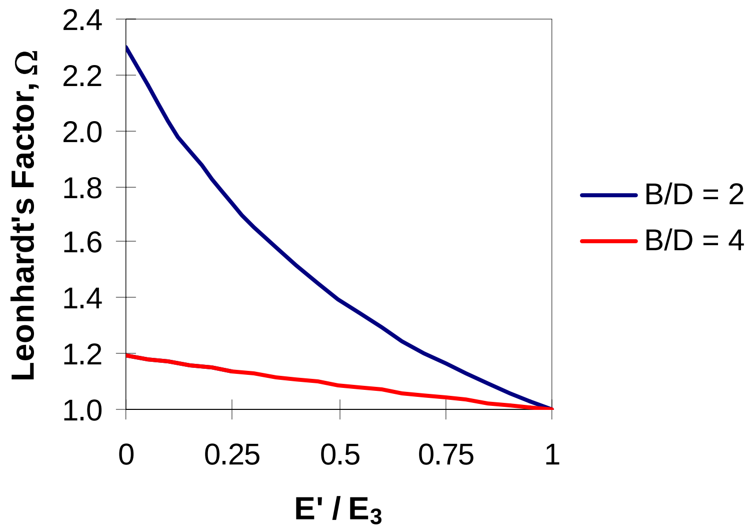


Figure 2-3. Leonhardt's correction factor on modulus for two trench geometries

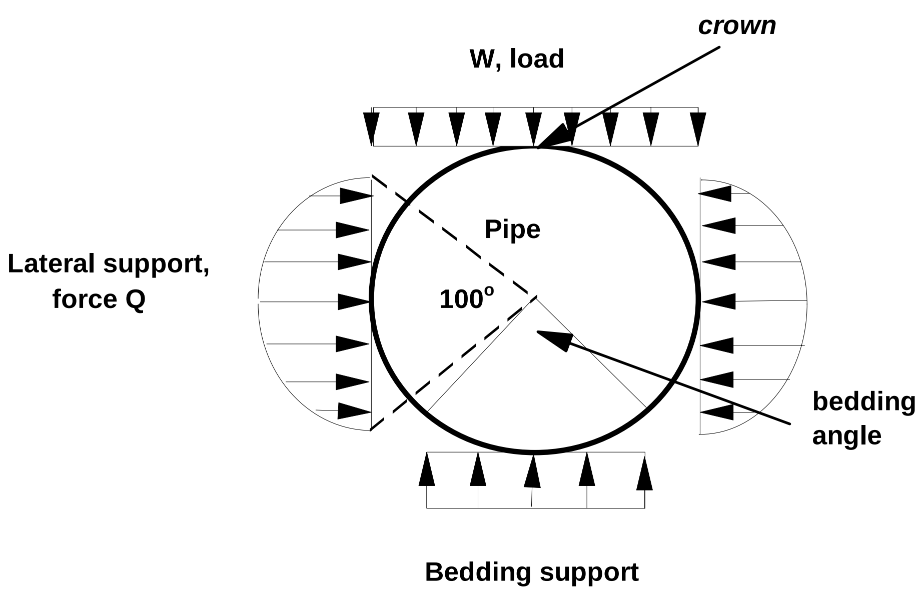


Figure 2-4. Commonly assumed pipe pressure distributions

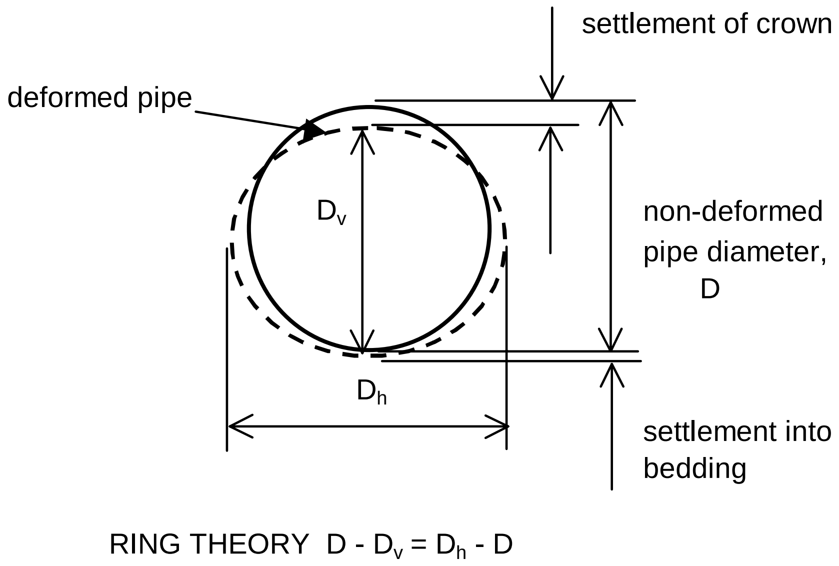


Figure 2-5. Exaggerated pipe deformation

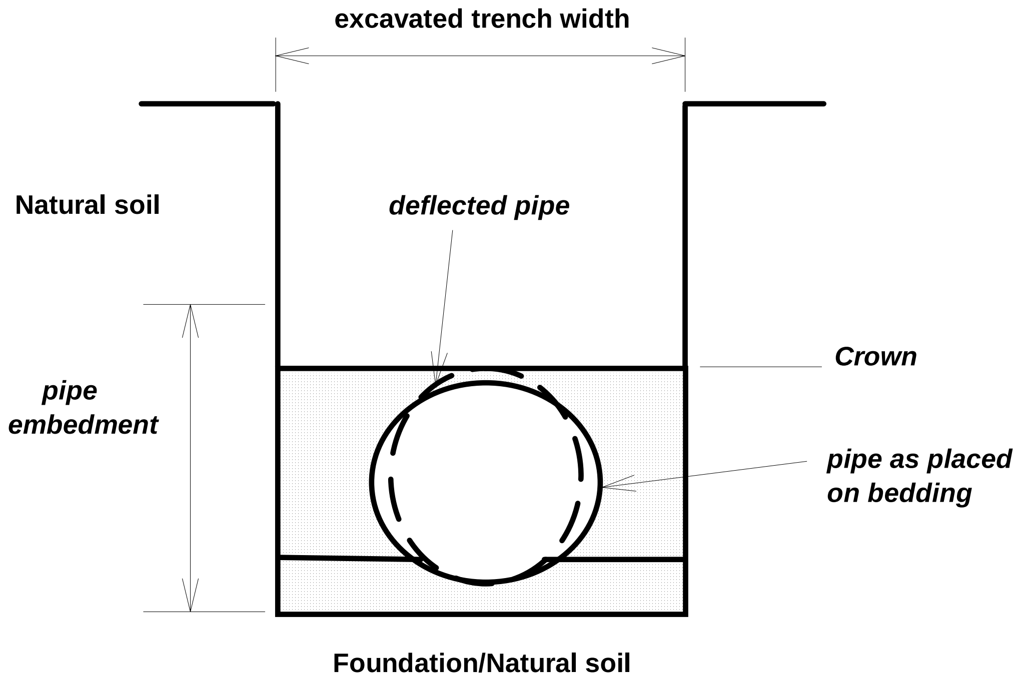


Figure 2-6. Initial potential pipe distortions due to compaction

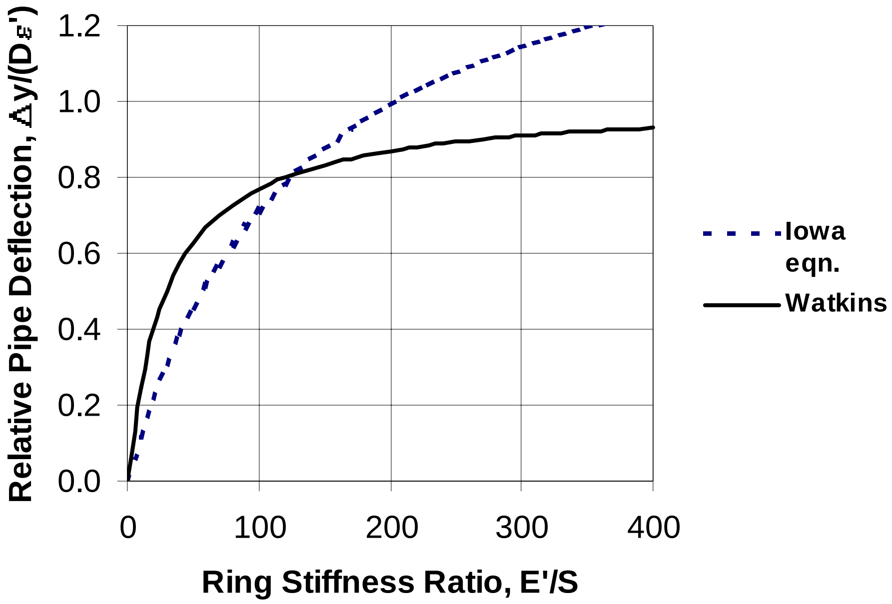


Figure 2-7. Comparison of the Iowa equation and Watkins's (1988) empirical expression

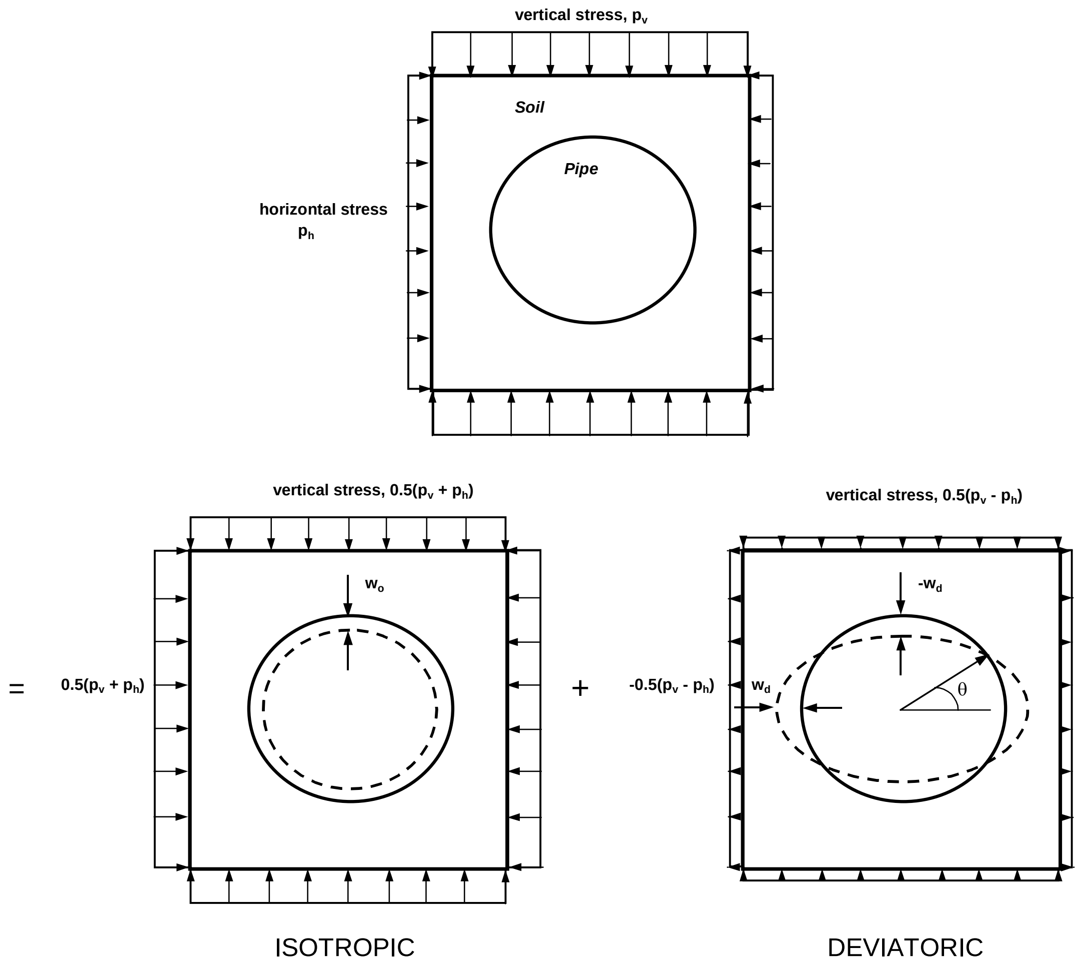


Figure 2-8. The response of a flexible thin pipe to the isotropic and deviatoric components of external loading (after Moore 1993).

Note: the dashed line represents the deflected pipe

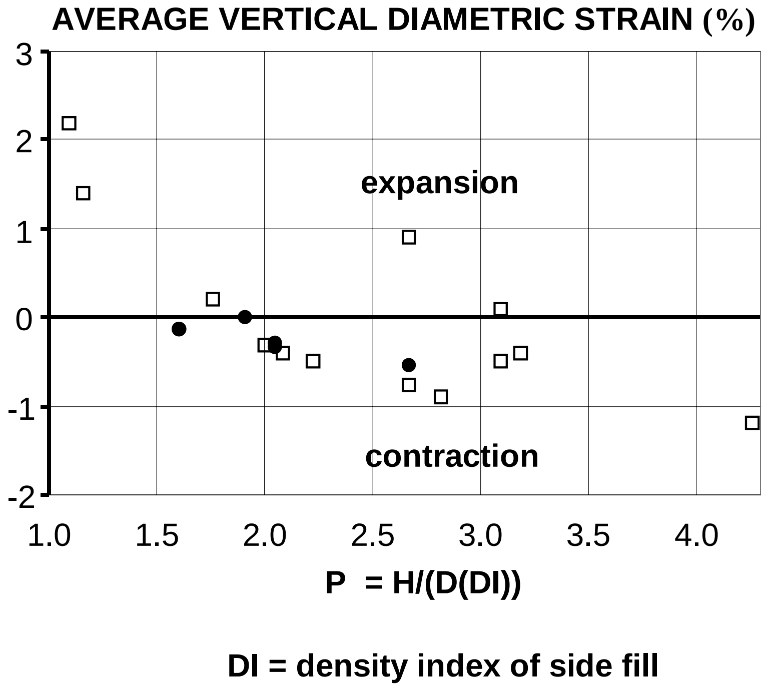


Figure 2-9. The variation of installation deformation with backfill and sidefill compaction and pipe diameter

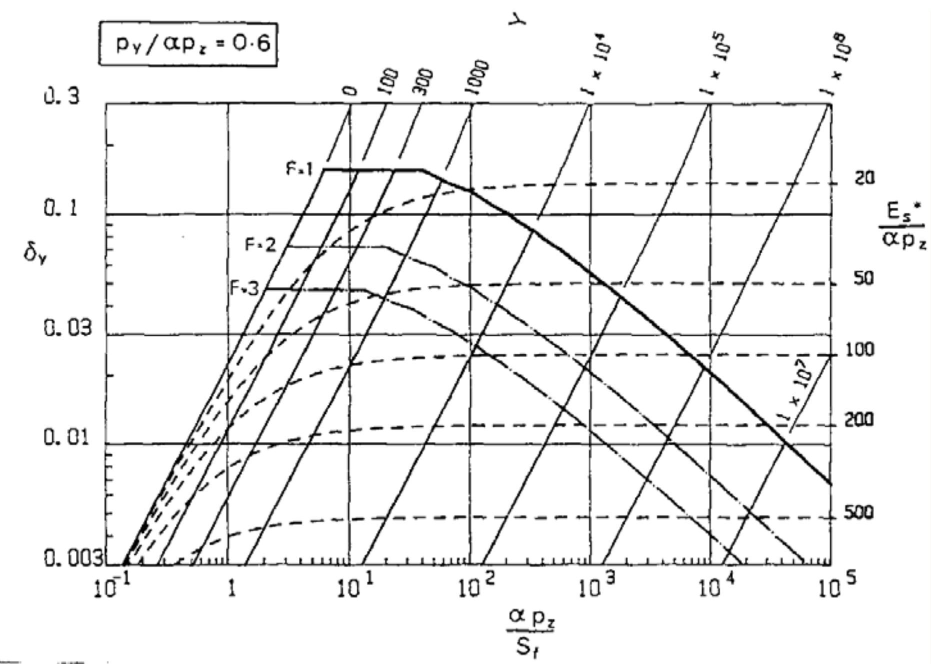


Figure 2-10. An example of a design chart for buried pipes from Gumbel and Wilson (1981)

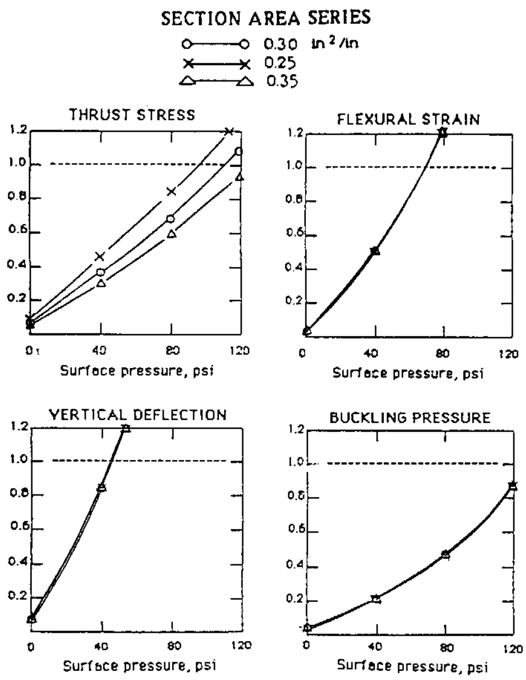


Figure 2-11. Influence of corrugated section on design criteria

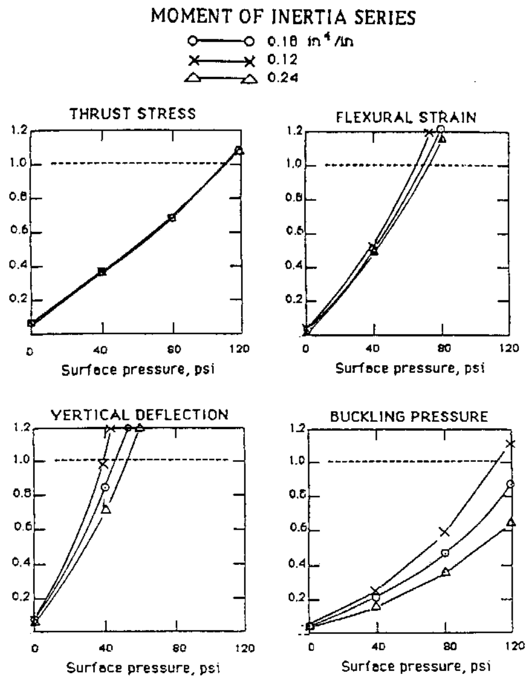


Figure 2-12. Influence of moment of inertia on design criteria

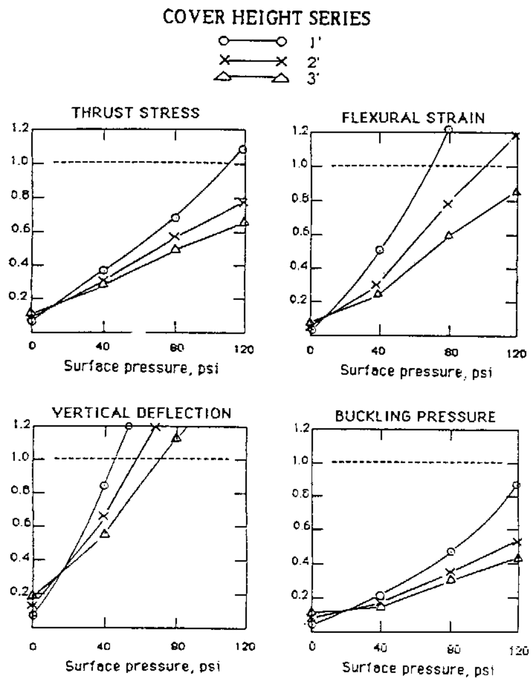


Figure 2-13. Influence of soil cover height on design criteria

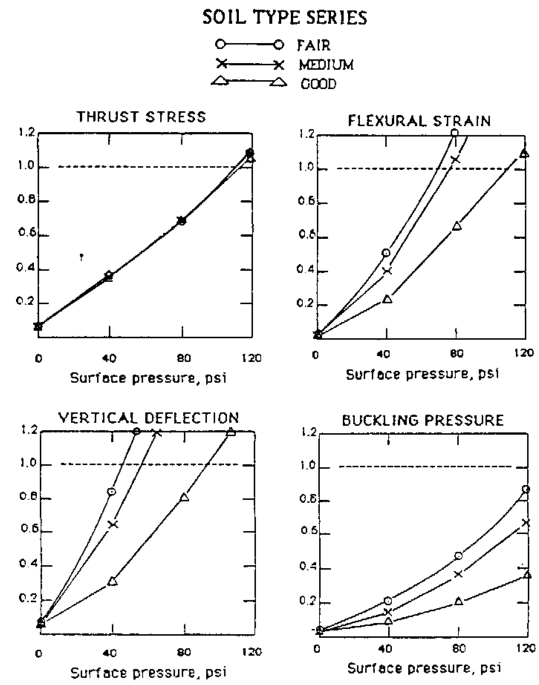


Figure 2-14. Influence of soil type on design criteria

Figures 2-11 to 2-14. The influences of pipe cross-sectional area, moment of inertia and soil cover height and soil type on buried pipe performance (Reference: Katona 1990)

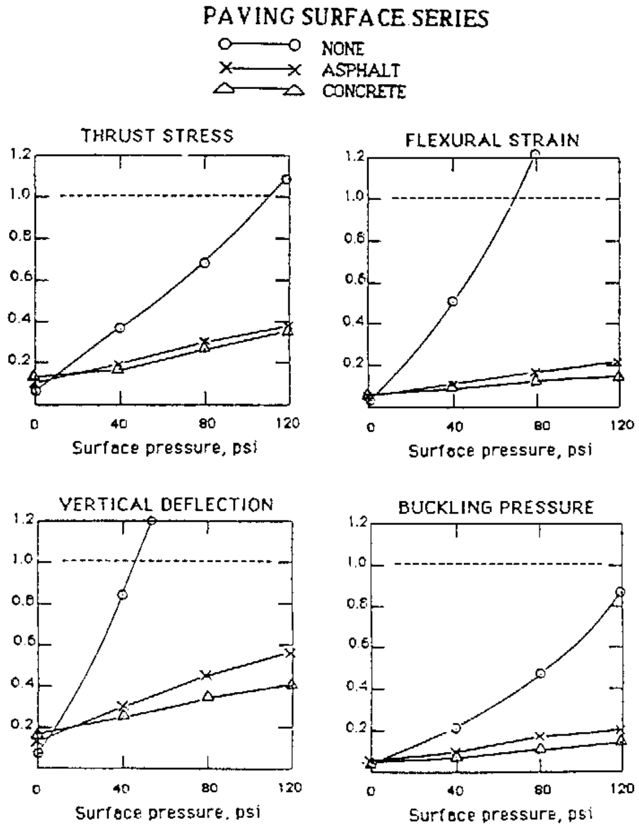


Figure 2-15. Influence of a thin pavement on buried pipe performance (Katona 1990)