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Finite Element Analysis of Deeply Buried Flexible Pipes

Sarah Elkhatib
GHD Pty Ltd, Perth, Western Australia

Summary Residue underdrains in tailings storage facilities are commonly buried under high fills. Flexible plastic pipes are normally used, the material being either PVC or HDPE. The performance of flexible pipes under large heights of tailings is relatively unknown. Guidelines for the design of buried flexible pipelines are presented in Australian Standard 2566.1. However, these are mainly for shallow burial of pipes. The main limitation of existing design methods is that the effect of arching is ignored. Many pipe designers use the prism load routinely even when considerable arching occurs. This often leads to conservative designs which are not cost-effective. A new bauxite residue disposal area to be developed at Worsley in Western Australia was used as a case study. The residue underdrains will be buried under 70 m of tailings and therefore subjected to very high overburden pressures. Due to the limitations of existing design methods, finite element analysis (FEA) was used to model the pipes. FEA can simulate the complex interaction between the pipe and soil more accurately than other methods. Several factors which influence the structural performance of flexible pipes were examined and the results are presented in this paper. The advantages of using FEA over conventional design methods are discussed.

1. INTRODUCTION

A network of slotted pipes is generally required for the collection and removal of liquor from residue storage facilities. The residue underdrains are commonly flexible pipes buried under high fills.

Buried flexible pipelines can be designed using various methods available in the literature. However, most of the conventional design methods have limitations. They are primarily for shallow burial of pipes, and so are inappropriate for residue storage applications. A more appropriate procedure for simulating the complex interaction between the pipe and soil is finite element analysis (FEA).

A new bauxite residue storage facility at Worsley in Western Australia was used as a case study. The residue underdrains, which will ultimately be buried under 70 m of tailings, were designed using FEA.

This paper presents the results of the modelling carried out and discusses the influence of several factors on the structural performance of the deeply buried pipes. The results from FEA are compared to ones calculated using conventional design methods and a discussion of their limitations is provided.

2. OVERVIEW OF WORSLEY ALUMINA

The Worsley Project lies in the Darling Ranges in the south-west of Western Australia. It is located 200 km south of Perth and 50 km east of Bunbury on the headwaters of the Augustus River. The production of alumina at Worsley is expected to double from 1999 to 2000. Consequently, this will lead to an increase in residue production that will exceed the storage capacities of the existing disposal areas. This identified the need to develop another facility to store the residue.

The two general areas used for the disposal of residue are the Northern Valley and the Southern Valley. The Northern Valley contains the existing five bauxite residue disposal areas (BRDA 1, 2, 3, 4, and 4X), as shown in Figure 1. The new residue disposal area, named BRDA 5, is located in the Southern Valley.

BRDA 5 will be the largest of all the residue disposal areas when developed to its ultimate stage. It will be approximately 70 m in depth, occupy an area of 450 hectares and store 185 million m^3 of residue.

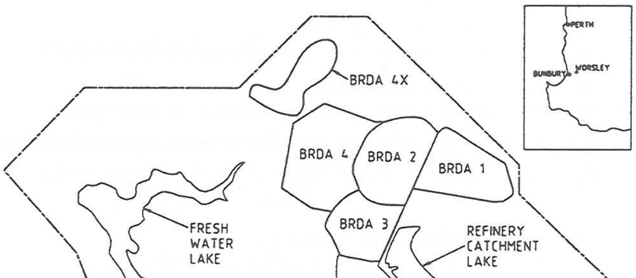


Figure 1: Site plan of the Worsley Alumina Refinery

The underdrainage blanket on the floor of BRDA 5 consists of the layers illustrated in Figure 2. A clay blanket of low permeability overlies the foundation and is designed to prevent seepage of contaminated liquor into the groundwater stream. The clay blanket is overlain by a highly permeable gravel blanket and a network of residue underdrains. At ultimate development, the underdrains will total approximately 75 km in length. The pipes are designed to collect and remove liquor, thereby increasing residue strengths, and reducing the pressure head across the clay liner, and hence minimising contaminant leakage. The pipes are surrounded by a gravelly backfill. The trench backfill and the clay fill are referred to as the embedment and native soils respectively.

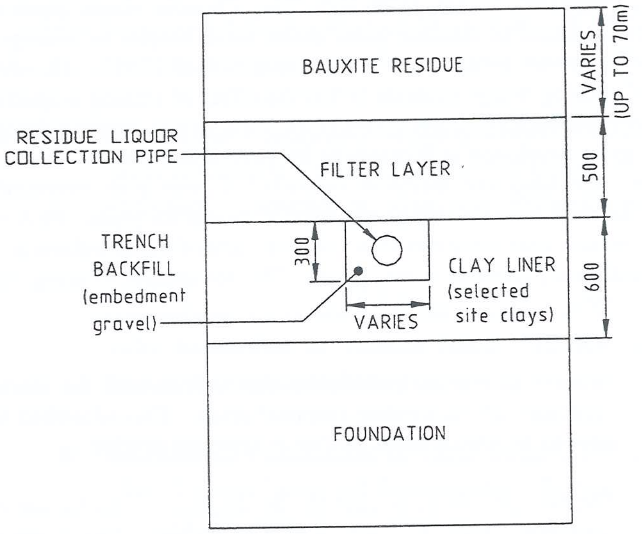


Figure 2: Illustration of underdrainage system of BRDA 5

3. NOTATION AND UNITS

The following symbols are used in this paper.

B = trench width [m]
C' = soil cohesion [kPa]
D = Mean diameter of pipe [m]
D_e = external diameter of pipe [m]
D_L = deflection lag factor [-]
E_e = Young's modulus of embedment soil [kPa]
E_n = Young's modulus of native soil [kPa]
E_p = modulus of elasticity of pipe material [kPa]
E_e' = modulus of soil reaction of embedment soil [kPa]
E_n' = modulus of soil reaction of native soil [kPa]
I = moment of inertia of the pipe wall [m^4/m]
K = pipe bedding constant [-]
P_{cr} = critical buckling load of pipe [kPa]
R = mean radius of pipe [m]
SN = ring-bending stiffness of pipe ($= EI/D^3$) [kN/m/m]
T = pipe wall thickness [m]
W = load per unit length of pipe [kPa]
Δx = horizontal deflection of pipe [m]
Δy = vertical deflection of pipe [m]
ϕ = soil friction angle [degrees]
γ = unit weight of soil [kN/m^3]
ν = Poisson's ratio [-]

4. DESIGN CRITERIA

For buried flexible pipes, the main failure modes which need to be considered are excessive deflection, circumferential bending strain and wall buckling (Moser, (1)).

4.1 Deflection

Deflections caused by the weight of the overlying fill can be the governing criterion in the design of buried pipelines. Longitudinal deflection is usually not of concern. With careful placement of the bedding, the pipe does not sag. Ring deflection, however, is of great concern as it can lead to reversal of curvature of the pipe. It can cause leaks, reduce flow and also contribute to incipient collapse of the ring. Deflection limits vary for different pipe materials. For high-density polyethylene (HDPE) pipes, the recommended limit is 7.5%.

4.2 Bending Strain

When a pipe deflects under load, bending strains are induced in the pipe wall. Strain is related to deflection and so most pipe manufacturers specify installation techniques which will limit deflection and thus limit the strain. Poor installation can lead to localised deformations, which can influence the performance of the pipe. The recommended strain limit for HDPE pipes is 4%.

4.3 Buckling

Buckling may govern the design of pipes which have low stiffness and are subjected to high soil pressures. The actual pressure on the pipe can be compared with the allowable buckling pressure to obtain a factor of safety against buckling. Australian Standard 2566.1 (2) recommends a factor of safety of 2.5 be adopted against buckling.

The above parameters indicate that deflections and loads on pipes are of most importance. Limiting deflections can often limit strains and limiting the load on a pipe increases the factor of safety against buckling. As a result, these two parameters were the main ones investigated.

5. EXISTING DESIGN METHODS

One of the most important properties of flexible pipes is their ability to induce arching. Arching is the load redistribution which occurs when the pipe stiffness is different to that of the soil. The stiffness of most flexible pipes is less than the soil stiffness and so the overburden load (or prism load) is redistributed away from the pipe and into the adjacent soil. Theoretically, the load on a pipe will only be equal to the prism load if the pipe and soil have the same stiffness.

Several methods exist for the design of buried pipes, as discussed below. One of the major limitations in these conventional methods is the neglect of arching.

5.1 Spangler's Iowa State Formula

The Iowa State Formula was developed by Spangler (3) in 1941. Spangler incorporated the effects of the surrounding soil on the pipe's deflection. He modelled the vertical pressure on the pipe as a uniform distribution and the lateral pressures as parabolic distributions. Through analysis he derived the following formula:

$$\Delta x = \frac{D_L K W r^3}{E_p I + 0.061 E_e' r^3} \qquad (1)$$

The deflection lag factor, D_L, accounts for the fact that as a result of soil consolidation at the sides of the pipe, additional load may be expected on the pipe with time. A value ranging from 1.5 to 2.5 for D_L was originally proposed. However, due to the inherent conservatism in the formula, Moore (4) suggested that a value of 1.0 be used, but with the prism load rather than a load reduced because of arching. The bedding constant, K, varies with the bedding angle. However, a value of 0.1 is often assumed in the calculation.

Spangler did not go beyond the determination of the horizontal deflection. He assumed that the vertical deflection was always equal to the horizontal one. However, contrary to Spangler's assumption, in the case of very flexible pipes, the vertical deflection departs considerably from the horizontal one (Prevost and Kienow, (5)). Spangler's original work involved steel pipes with stiffness values significantly higher than that of HDPE. Therefore, use of Spangler's equation to predict Δx of HDPE pipes would yield unrealistic results (Jeyapalan and Abdelmagid, (6)).

The other inaccuracy in the Iowa state formula is the modulus of soil reaction, E_e'. This is a semi-empirical parameter required for input into the formula. Unfortunately, E_e' is not a true soil parameter and cannot be measured either in a laboratory or in the field. Efforts to develop tests to evaluate it have been largely unsuccessful. The most widely used E_e' values were developed by Howard (7) in 1977. Howard used Spangler's formula for back-calculating E_e' from field data. However, his values do not reflect the change in soil stiffness that occurs with increasing depth of fill.

5.2 Critical Buckling Loads

The use of Luscher (8) buckling theory is common in the design of buried plastic pipes. This approach used the modulus of soil reaction to characterise the soil support. The uncertainties associated in using this modulus are as discussed above. Luscher's buckling theory is a simplified analysis of the soil-pipe system as it ignores shear deformations in the soil.

A more rigorous approach considers the soil to behave as a continuum (Moore and Selig, (9)). For a flexible pipe supported by uniform ground of modulus, E_n, continuum buckling theory suggests that the critical buckling load can be calculated using the following formula:

$$P_{cr} = \frac{(E_p I)^{1/3} (E_n)^{2/3}}{D} \qquad (2)$$

In contrast to the spring model, continuum theory employs modulus parameters with real physical meaning.

5.3 Australian Standard 2566.1

AS 2566.1 outlines guidelines for the design of buried flexible pipelines. The calculation of deflections is based on the Iowa formula. However, E_e' in the Iowa formula is replaced by an effective combined modulus which accounts for both the embedment and native soils.

Other limitations include the use of the empirical parameter E_e' and the assumption that Δx and Δy are equal, as discussed for the Iowa formula.

The calculation of the allowable buckling pressure using AS2566.1 is based on continuum buckling theory. However, the soil modulus, E_n, is replaced by an effective combined modulus.

5.4 Finite Element Analysis

FEA has been developed as a numerical method to solve complex problems in continuum mechanics. It permits a reasonably accurate treatment of the soil-pipe interaction problem. The soil mass around a pipe is set up as a mesh of soil elements and each element can be assigned different properties. Various loading conditions, sub-surface conditions and structural properties can be modelled mathematically, which is an advantage over physical testing of such structures.

Soil stiffness is defined using Young's modulus and Poisson's ratio, both which can be readily obtained using laboratory testing. The three design criteria that need to be checked can generally be obtained directly from FEA programs, eliminating the need to use inappropriate formulae. FEA also incorporates the effects of arching, yielding more realistic results regarding the interaction of the pipe and soil system.

6. DESIGN OPTIONS

Several design options were considered to obtain the most optimal design. The main variables were:

- the trench width;
- the embedment soil stiffness; and
- the pipe stiffness.

7. FEA MODELLING

The large array of pipes at Worsley and the magnitude of the loads they will be subjected to justifies the use of rigorous design procedures to ensure an adequate design is adopted.

Due to its advantages over conventional pipe design methods, FEA was used to model the complex soil-pipe system. The software used for the analysis is called FLAC (Fast Lagrangian Analysis of Continua).

7.1 Overview of FLAC

FLAC is a two-dimensional finite element program which was developed in 1986. It simulates the behaviour of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. Materials are represented by elements which form a grid. Each element behaves according to a prescribed linear or non-linear stress/strain law in response to the applied forces or boundary conditions.

7.2 FLAC Model

7.2.1 Geometry

Figure 3 shows the finite element mesh used in the analysis. Only one-half of the pipe was modelled due to symmetry.

FLAC is limited by a maximum mesh size. Accordingly, the vertical and lateral extents of the model are limited, and so the mesh size should be selected carefully in order to keep it within FLAC's capability without compromising the accuracy of the results. Therefore, the vertical depth of up to 70 m was simulated by assuming an appropriate residue height above the pipe and by substituting the remainder of the residue by an equivalent uniform surcharge load. The height above the pipe was large enough to ensure that the equivalent surcharge above this height has the same effect as the actual residue. The lateral extent of the mesh away from the pipe was also chosen such that the vertical boundary does not interfere with the stress conditions close to the pipe.

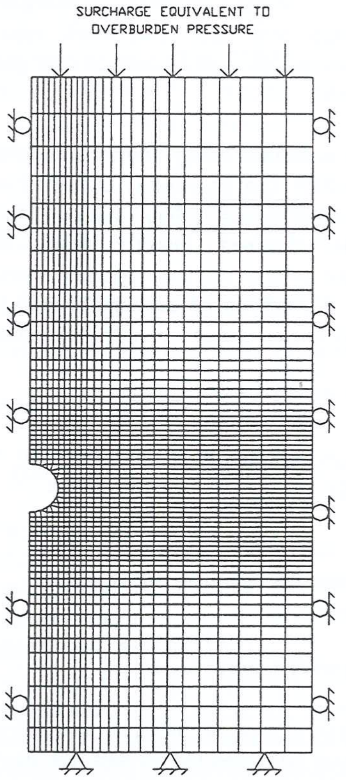


Figure 3: FEA mesh and applied boundary conditions used in the analysis

7.2.2 Soil Properties

Soil properties required for input into FLAC include the unit weight, cohesion, friction angle and bulk and shear moduli. The bulk and shear moduli were calculated from Young's modulus and Poisson's ratio. Table 1 shows the properties used for the analysis. When a different trench backfill was to be examined, only Young's modulus was varied. The remaining properties do not vary significantly for the materials considered. Also, they do not have a significant influence on the pipe's performance criteria and so were kept constant for simplicity.

Table 1: Soil parameters used in the analysis

Zone	γ (kN/m^3)	c' (kPa)	ϕ' (°)	E (MPa)	ν
Residue (unconsolidated)	20	0	35	20	0.3
Filter	20	0	35	20	0.25
Trench Backfill (embedment soil)	20	0	35	Varies (10-60)	0.25
Clay Liner (native soil)	17	5	32	60	0.3
Foundation	14.5	25	30	40	0.3

The primary materials considered for the trench backfill included lateritic gravel (E_e = 20–30 MPa) and crushed basalt (E_e = 60 MPa). Intermediate values were used in the analysis to account for slightly different materials that may be available on site and to establish relationships between soil stiffness and pipe performance criteria.

7.2.3 Pipe Properties

The pipe was modelled as a series of structural beam elements which were assumed to behave in a linear elastic manner. The parameters required for establishing the stiffness matrix of the pipe were Young's modulus, moment of inertia through the pipe wall and the cross-sectional area of the pipe wall.

Several pipe diameters were examined depending on the area that the pipe is servicing. The trends observed were consistent and so only results from the HDPE pipes with an external diameter of 100 mm are presented. Several pipe stiffnesses were considered and these are presented in Table 2. For all the pipes, E_p = 337 MPa.

Table 2: Properties of pipes used in the analysis

Pipe No.	t (mm)	I (m^4/m)	SN (kN/m/m)
1	2.5	1.3×10^{-9}	0.47
2	2.9	2.0×10^{-9}	0.72
3	3.3	3.0×10^{-9}	1.12
4	4.0	5.3×10^{-9}	2.03
5	5.0	1.0×10^{-8}	4.09
6	6.6	2.4×10^{-8}	9.91
7	10.0	8.3×10^{-8}	38.52

Note that whilst it was apparent that some of these pipes were too thin-walled to withstand high loads, they were included in the analysis to establish a relationship between the pipe/soil stiffness ratio and the pipe's structural performance.

7.2.4 Boundary Conditions

The boundary conditions specified are illustrated in Figure 3. Nodes lying on the bottom horizontal boundary were restricted from movement in both the horizontal and vertical directions. Symmetry is simulated by allowing only vertical movement of the nodes lying on the pipe centre-line. The nodes on the right vertical boundary were restricted from horizontal movement.

8. RESULTS AND DISCUSSION

8.1 Trench Installations

The installation of pipes in trenches can have a significant influence on the design of a buried pipe. Several factors need to be considered for pipes installed in trenches. These include the width of the trench and the stiffness of the trench backfill.

The effects were examined by modelling various trench widths, ranging from 1.6 pipe diameters to 4 pipe diameters. The soil embedment modulus, E_e, was also varied between 10 MPa and 60 MPa. Note that when $E_e = 60$ MPa, the model is effectively without a trench as $E_n = 60$ MPa also. This simulates an embankment type installation. The same pipe was used in all the models. This was a 100 mm HDPE pipe with a stiffness of 9.9 kN/m/m.

8.1.1 Deflections Using FLAC

Figure 4 illustrates the relationship between the trench widths and vertical deflections of the pipe for various embedment soil moduli. It shows that deflections are sensitive to both criteria, especially at lower values of soil modulus. A soft backfill does not provide as much support for the pipe enabling the pipe to deflect more vertically and laterally. This emphasises the importance of careful selection of the trench backfill material and the compaction effort.

The vertical deflections also increase as the trench width increases. The trench walls, where an increase in soil stiffness is encountered (from the embedment to the native soils), are further away from the pipe and so the influence of the stiffer native soil in supporting the pipe is reduced.

Note that except for $E_e = 10$ MPa, all the cases considered resulted in deflections less than 7.5%.

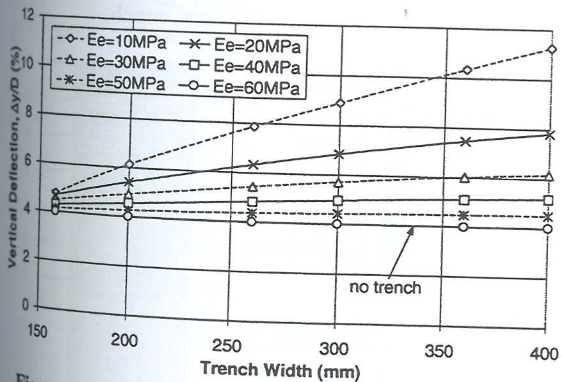


Figure 4: Effects of trench width and backfill on vertical deflections

The horizontal deflections were also calculated and they showed that deflection ratios (ie $\Delta y/\Delta x$) were always greater than unity for the soil and pipe combinations considered. For stiffer embedment materials, the deflection ratio was found to be higher than for softer materials. The ratios ranged from 1.5 (for $E_e = 10$ MPa and $B = 4D_e$) up to 5 (for $E_e = 50$ MPa and $B = 1.6D_e$). This confirms that the Iowa formula for calculating deflections is not adequate, as it assumes that the vertical and horizontal deflections are equal.

Note that the trends observed are only for cases where the native soil is stiffer than the embedment soil. If the embedment material were stiffer, then a wider trench would result in smaller deflections. Such cases were considered in the analysis to account for the variability of the soils on site. However, those results are not presented.

8.1.2 Deflections Using Conventional Methods

The vertical deflections obtained from FLAC were compared to those calculated using both Spangler's formula and AS2566.1. The results are illustrated in Figure 5. The results are shown for a soft backfill ($E_e = 10$ MPa) and a stiff backfill ($E_e = 50$ MPa).

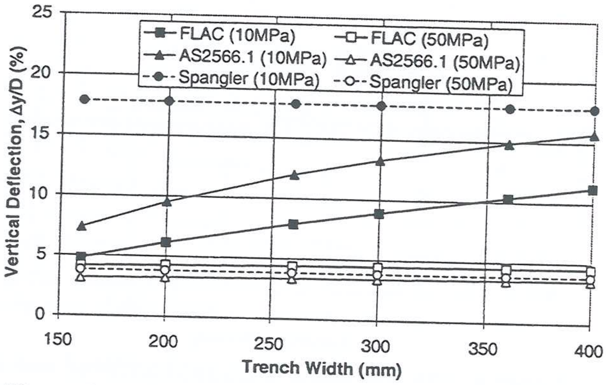


Figure 5: Comparison of FLAC with conventional methods

For a soft backfill, Spangler's equation significantly overestimates deflections as it does not account for the support provided by the stiff native soil. AS2566.1 appears to follow the same trend as FLAC, however, the deflections are still overestimated for the soft backfill case. Note that Spangler's equation gives a constant value regardless of the trench width. The results from FLAC and AS2566.1 appear to converge with Spangler's results as the trench width increases. This is a result of the native soil having less influence on the behaviour of the pipe as the trench width increases. Note that for $E_e = 10$ MPa, even the narrowest trench does not satisfy the deflection constraints when calculated using conventional methods.

For a stiff backfill, both Spangler's equation and AS2566.1 underestimate deflections. However, the difference is not as significant as for the soft backfill, and they were all less than 7.5%.

8.1.3 Strains

Bending strains were calculated for each of the cases considered. Trends observed indicate that strains increase as the embedment soil stiffness decreases. However, the strains were always less than 4%. HDPE pipe can tolerate high strains resulting from its deformation because of its ductility, and so pipe strain was not the governing criterion in the design of the pipes.

8.1.4 Buckling

Whilst Figure 4 indicates that with respect to deflections, ideally a narrow trench with stiff backfill is the best combination for design (only if the native soils are stiffer than the embedment soils), this has to be compromised with the buckling criteria of the pipe. Figure 6 illustrates the relationship between the vertical load on the pipe and trench widths for various embedment moduli.

As the flexible pipe undergoes vertical deflection, the soil above it tries to follow the pipe downwards. However, the soil's movement is resisted by shear resistances along the trench walls. Movement along these slippage surfaces is resisted by the soil's shear resistance, which brings about an active state in the soil above the pipe. Through this action, part of the weight of the backfill is carried over into the trench walls.

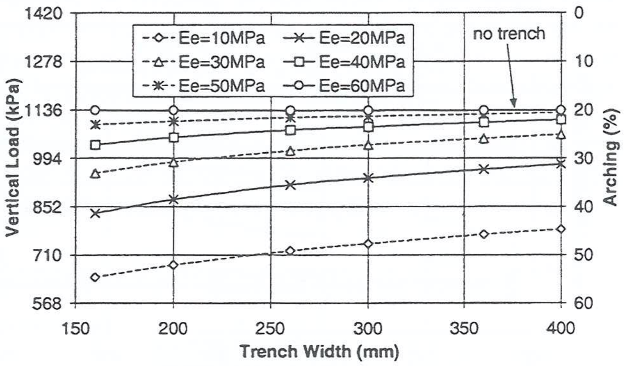


Figure 6: Effects of trench width and backfill on arching

Figure 6 indicates that the embedment soil stiffness significantly influences the load on the pipe. As the soil stiffness decreases, more load is redistributed away from the pipe. A softer soil results in higher pipe deflections and more soil compression. This mobilises an active state in the backfill and load is shed to the stiffer soils.

Arching in excess of 50% was observed. The critical buckling loads were calculated using continuum buckling theory and then compared to the reduced loads on the pipe to obtain factors of safety against buckling. The cases with soft backfill (ie 10 MPa and 20 MPa) failed the buckling criteria. Cases with a trench width greater than $3.5D_e$ also failed.

Factors of safety calculated using the prism load rather than the reduced load, were generally always less than 2.5, and so failed the buckling performance criteria. Note that using the prism load will always result in lower factors of safety, and hence, AS2566.1 will lead to conservative designs.

8.1.5 Stress and Displacement Contours

Figures 7(a) and 7(b) show the vertical stress distributions and vertical displacement contours for the two different pipe installations: one in a trench ($E_e = 20$ MPa and 200 mm trench width) and the other without a trench ($E_e = 60$ MPa).

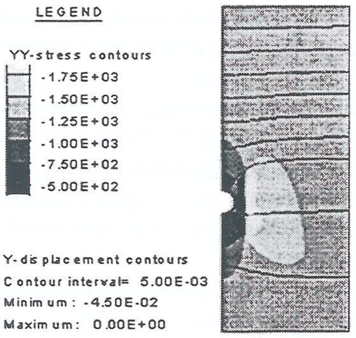
Figure 7(a) illustrates how load is distributed to the stiffer native soil. The lowest stresses occur at the pipe crown where the greatest displacements occur. Soil compression is significant above the pipe, also assisting in load redistribution.

Figure 7(b) illustrates the case without a trench. Minimum stresses occur at the pipe invert and crown. However, a significant load increase is observed in the soil directly adjacent to the pipe. Note also that vertical displacements of the soil are almost uniform across the model.

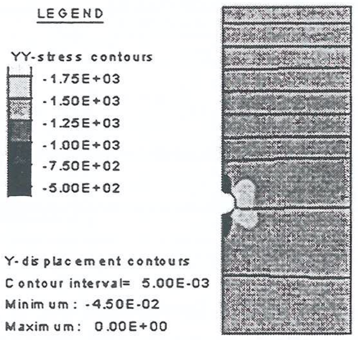
8.1.6 Vertical Loads on Pipe

Figure 8 shows the vertical stresses in the soil across a plane above the pipe for a few of the cases with trenches and the case without a trench. The minimum stresses occur at the pipe crown. Also shown is the prism load at approximately 1420 kPa (equivalent to 71 m of overburden height). The stress at the crown of the pipe is approximately 600 kPa, indicating that arching in excess of 50% occurs. A local maximum occurs at the pipe springline. This varies for the different trench widths. Away from the pipe, the stresses then reduce until the trench wall is encountered. A significant increase is observed due to the sudden increase in soil stiffness. The increase exceeds the prism load to counteract the load reductions above the pipe. The stresses then decrease gradually and approach the prism load further away from the pipe.

Note that Spangler's assumption of uniform vertical pressures on the pipe is incorrect.



(a) Pipe in 200 mm trench



(b) Pipe without trench

Figure 7: Stress and displacement contours for different pipe installations

The case without the trench exhibits different trends. Whilst the stress above the pipe crown is also at approximately 600 kPa, it increases significantly over the radius of the pipe, and reaches a maximum (in excess of the prism load) at the pipe springline. The stresses then gradually decrease until the prism load is reached.

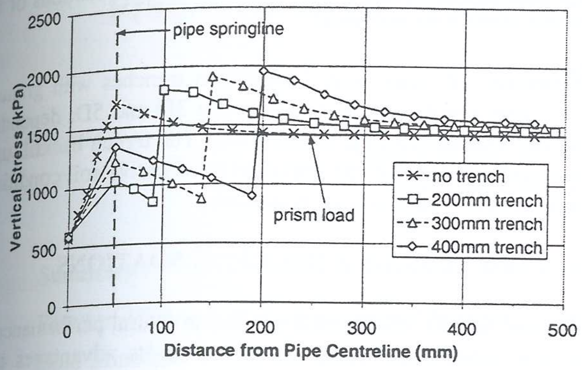


Figure 8: Vertical stresses acting on a plane above the pipe

8.2 Pipe and Soil Stiffness

The stiffness of a pipe and soil system plays an important role in the design of flexible pipes and is the main variable influencing arching around a pipe.

Several pipe stiffnesses were considered in the design of the residue underdrains. The embedment soil was also varied to determine the best combination of pipe and trench backfill for the underdrainage system. The trench width used for this study was kept constant at 200 mm.

8.2.1 Deflections Using FLAC

Figure 9 illustrates the relationship between pipe stiffness and deflections for various soil moduli. As would be expected, an increase in pipe stiffness results in smaller vertical deflections of the pipe. Also, an increase in the embedment soil modulus results in smaller deflections.

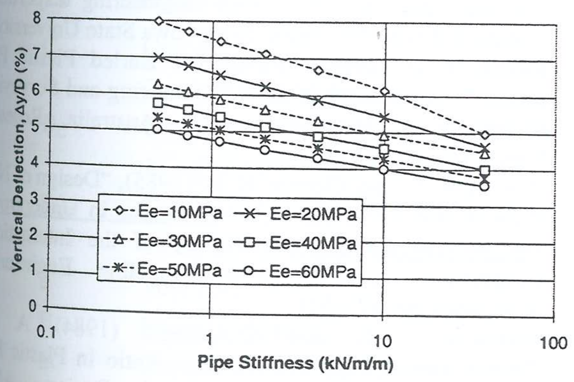


Figure 9: Effects of pipe and soil stiffness on deflections

Horizontal deflections were also calculated for all the cases. The ratio of vertical to horizontal deflection of the pipe increases significantly with the decrease in pipe stiffness and increase in soil stiffness. Again, the deflection ratios were greater than unity for all the cases considered.

8.2.2 Deflections Using Conventional Methods

The vertical deflections calculated by FLAC were compared to those calculated using conventional methods. The results are illustrated in Figure 10. The same trends were observed as for the previous case. For soft backfills, both the conventional methods overestimate deflections significantly, resulting in deflection in excess of 7.5%. For stiff backfills, both methods underestimate deflections, but not significantly.

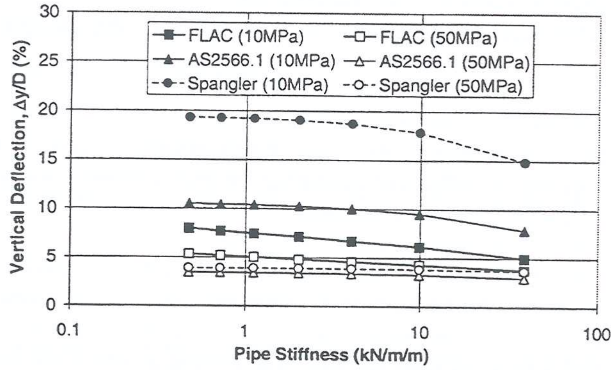


Figure 10: Comparison of FLAC with conventional methods

8.2.3 Strains

The strains were calculated for all the cases modelled and they were always less than the recommended limit of 4% for HDPE pipes, and so this was not a governing factor in the selection of the pipes.

8.2.4 Buckling

Figure 11 illustrates the relationship between arching and pipe stiffness for various soil moduli. As the pipe stiffness increases the pipe attracts more load. This is a result of the smaller deflections of the pipe. When a pipe deflects, the soil directly above it is mobilised. As the stiffness of a pipe increases, it deflects less and consequently, less backfill is mobilised, resulting in more load on the pipe.

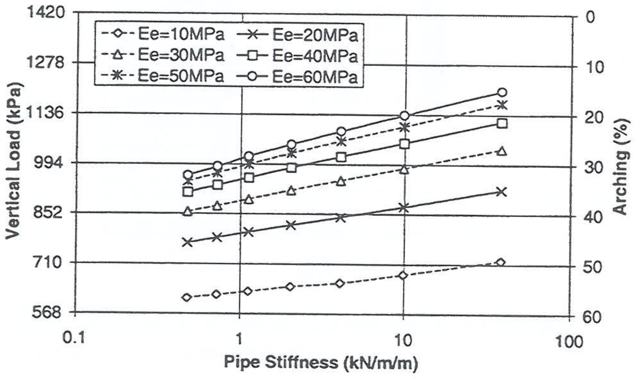


Figure 11: Effects of pipe and soil stiffness on arching

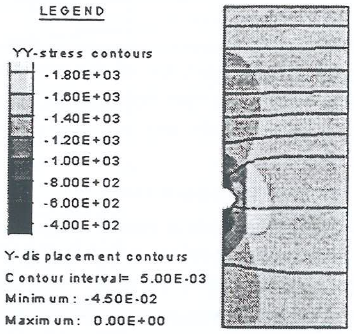
For low values of soil modulus (ie 10 MPa), arching in excess of 50% was observed. Figure 11 indicates that arching is more dependent on the surrounding soil than the pipe stiffness.

Based on the loads shown in Figure 11, the factors of safety against buckling were calculated. The thin-walled pipes, although subjected to lesser loads, generally failed the buckling criteria because their critical buckling pressures were too low. However, the three stiffest pipes were capable of withstanding the loads imposed on them.

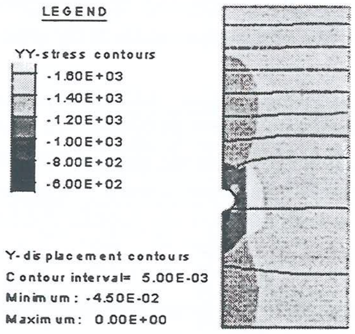
Calculating factors of safety using the prism load, rather than the reduced load, resulted in almost all the pipes failing the buckling criteria. This indicates that AS2566.1 would require stiffer pipes to be specified, leading to conservative designs.

8.2.5 Stress and Displacement Contours
Figures 12(a) and 12(b) show the vertical stress contours in the soil elements and their displacements. The figures illustrate the effects of increasing the pipe stiffness. All other properties were kept constant.

The stress contours show the load redistribution away from the pipe. Whilst most of the displacement contours appear to be the same, the one directly above the pipe is different. The stiffer pipe prevents the soil from settling as much and hence load transferral to the native soil is not as significant.



(a) Pipe stiffness = 0.47 kN/m/m



(b) Pipe stiffness = 10.0 kN/m/m

Figure 12: Stress and displacement contours for different stiffness pipes

It is important to recognise that the pipe stiffness plays a vital role in the successful design of flexible pipe-soil interaction systems. The field performance of a pipe would depend heavily on its ability to carry a portion of the load and encourage positive arching.

8.3 Adopted Design at BRDA 5

The detailed design adopted for BRDA 5 is not presented here due to its complexity. Pipes with different diameters were used depending on the area they are servicing. The pipe stiffness and material also vary depending on the location of the pipe in the valley. Obviously, the deepest sections of the valley require stiffer pipes.

Generally, all pipes were installed in trenches with granular fill. The trench widths vary between 3D and 5D, depending on the conditions of the native soils. The trench backfill used was well-graded lateritic gravel in good native soil conditions and crushed basalt in poor conditions.

9. CONCLUSIONS & RECOMMENDATIONS

Various factors which influence the structural performance of HDPE pipes were examined. Due to its advantages over conventional design methods, FEA was used for the analysis. The general trends observed indicate that the design of pipes is not simple. Several factors need to be considered. Whilst one variable may have a positive influence on pipe deflections, it may have a negative influence on wall buckling. All the criteria need to be considered for an optimal design to be reached.

Conventional design methods do not have sufficient variables to account for the complex interaction between the pipe, the granular backfill and the in-situ soil, and so can only provide a crude approximation of a pipe's performance. Such methods should only be used within the limits set by the assumptions made in their development.

10. REFERENCES

1. Moser, A. P. (1990), "Buried Pipe Design", McGraw-Hill.
2. AS/NZS 2566.1:1998, "Buried Flexible Pipelines - Part 1: Structural Design, Australian/New Zealand Standard.
3. Spangler, M. G. (1941), "The Structural Design of Flexible Pipe Culverts", Iowa Engineering Experiment Station Bulletin 153, Ames, Iowa: Iowa State University.
4. Moore, I. D. (1989), "Review of Buried Plastic Pipe Design", Department of Civil Engineering and Surveying, University of Newcastle, NSW, Australia, Research Report No. 039.05.89.
5. Prevost, R. C. and Kienow, K. K. (1985), "Design of Non-Pressure Very Flexible Pipe", Advances in Underground Pipeline Engineering, Proceedings of the International Conference on Underground Pipeline Engineering, Wisconsin, pp. 527-541.
6. Jeyapalan, J. K. and Abdelmagid (1984), A. M., "Importance of Pipe Soil Stiffness Ratio in Plastic Pipe Design", Pipeline Materials and Design, ASCE Convention, San Francisco, California, pp. 48-66.
7. Howard, A. K. (1977), "Modulus of Soil Reaction Values for Buried Flexible Pipe", Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, GTI.
8. Luscher, U. (1966), "Buckling of Soil-Surrounded Tubes", Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 192, No. 6, pp. 211-228.
9. Moore, I. D. and Selig, E. T. (1990), "Use of Continuum Buckling Theory for Evaluation of Buried Plastic Pipe Stability", Buried Plastic Pipe Technology, ASTM STP 1093, pp. 344-359.