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FINITE ELEMENT CALCULATIONS OF STRESSES AND DEFORMATIONS IN BURIED FLEXIBLE PIPES

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A Thesis

**Submitted to the School of Graduate Studies and Research
Under the Supervision of**

Dr. Erman Evgin

**in Partial Fulfilment of the Requirements for the Degree of
Master in Applied Sciences in Civil Engineering**

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ABSTRACT

The objectives of this thesis are (1) to measure the characteristics of an interface between a granular soil and a Polyvinyl Chloride (PVC) surface, (2) to use those characteristics as input parameters for interface elements in a finite element analysis, and (3) to evaluate the effects of interface strength on the behaviour of buried flexible pipe.

The work is divided into four main parts. First, a comprehensive literature survey was done for the behaviour of interfaces and for some well-known analytical techniques available for underground flexible pipe structures. Second, an experimental program involving an interface between a PVC plate and stone dust was undertaken. The effects of various parameters such as surface roughness, number of load cycles, normal stress and relative density were investigated. Third, numerical analyses were done in order to evaluate the effects of interface elements on the behaviour of a 1500-mm Fibre Reinforced Plastic (FRP) pipe and a 900-mm PVC pipe. Finally, the results of the numerical analyses were compared with those of well known analytical methods.

The experiments on interface were performed using the Cyclic Three-Dimensional Interface (C3DI) device. It was found that the shear strength of the interface is not significantly influenced by the surface roughness of the PVC plate with the normal stress varying from 100 to 300 kPa. The results also showed that the ratio of the interface friction angle to the soil friction angle is about 0.67.

The numerical analyses using the PLAXIS finite element code showed that the interface elements are necessary to predict accurately the deformations of the buried pipes. In addition, the shear strength of the interface between the pipes and the backfill material does not have a significant influence on the deformations of the pipes; however, it has an effect on the earth pressure distributions.

Analytical methods such as the modified Iowa formula by Watkins (1958) and the elastic solution by Høeg (1968) gave good approximation of the deformation of the buried pipes.

However, they do not consider the effect of the installation practice, which has a significant influence on the deformations of the pipes and the earth pressure distributions. The finite element method seems to be a better tool in that it allows considering the installation practice and it avoid several assumptions involved in the analytical methods.

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CHAPTER 1

INTRODUCTION

1.1 General

Pipes are important lifelines of modern urban infrastructure. They are essential to distribute water, dispose wastewater, and manage stormwater. Since the early years of the 20th century in North America, cast iron pipes were extensively used to build water distribution systems, and concrete pipes were used to build sewer and drainage systems. The average life of these conventional pipes is about 50-70 years; therefore, the infrastructures of most cities are near the end of their design life. In addition, the municipalities have to deal with the extension of their systems to keep up with the constant population growth. In order to replace or install new pipelines, the technology of today brought new material types providing good alternatives, and often better products.

Materials technology has evolved the most among all aspects of pipelines engineering. Many new materials such as plastic resins appeared on the market. According to Björklund (1996) in his article about the future use of plastic pipe, thermoplastic pipes account for 90% of the market for water distribution in the Nordic countries, and 75% for drains and sewers applications. There are three main types of plastic pipe used in water-industry applications: polyethylene (PE), polyvinyl chloride (PVC), and fibre-reinforced plastic (FRP). Polyethylene is characterised by its density. High density (HDPE) is strong, low density (LDPE) is relatively weak but highly ductile, and the medium density (MDPE) gives good compromise of strength and toughness. PVC is a thermoplastic and it is stronger than polyethylene, allowing thinner sections and reducing both weight and cost. However, it is more brittle and less tolerant of site handling. FRP pipes are relatively lightweight, and can be easily tailored to specific applications. They are also relatively easy to handle, but are susceptible to impact. The popularity of plastic pipes is due to constant improvements being made in manufacturing technologies and advantages over the conventional concrete and ductile iron pipes in terms of light-weightness, cost efficiency and long term chemical stability.

1.2 Statement of the problem

Although there are many research papers and reports about the behaviour of buried plastic pipes, there is limited experimental data about the behaviour of interfaces between soils and plastic pipes. An interface is a thin layer of soil next to a contact surface where stresses are transferred from one medium to another. It is well known that flexible pipes derive their strength and stability from the surrounding soil. Therefore, the interaction between the pipes and the surrounding soil should be taken into account in the analysis of buried plastic pipes.

1.3 Objectives of the Investigation

The first objective is to measure the characteristics of an interface between a granular soil and a PVC surface. The second objective is to use those characteristics as input parameters for interface elements in a finite element code, and evaluate their effects by comparing the results with an analysis without interface elements. The finite element analyses were carried out for a buried 1500-mm FRP pipe and a 900-mm PVC pipe taking into account the stages of construction and the compaction process.

1.4 Scope of the Investigation

A comprehensive literature survey was done first to review some well-known analytical techniques available for underground flexible pipe structures. The experimental part of the study involved PVC-stone dust interfaces. This type of soil is widely used as backfill for buried pipes. Tests were conducted using a cyclic three-dimensional interface (C3DI) apparatus (Fakharian and Evgin, 1996). A direct shear type soil container was used with the C3DI apparatus. The relationship between the shear strength, the normal stress, the number of cycles and the surface roughness was determined. A tester from Hommel industry was used to determine the surface roughness of the PVC surfaces.

Two types of stone dust were used for the interface tests. The engineering properties of these soils were determined from a series of triaxial tests (ASTM D2850), sieve analysis (ASTM D422), and minimum and maximum density tests (ASTM D4253 and D4254).

The finite element analysis was performed with the PLAXIS finite element code for soil and rock. In order to obtain realistic results, the analysis was based on the experimental data and numerical results of Mohri and Kawabata (1995). The experimental curve of the deformation of a 1500 mm fibre-reinforced plastic (FRP) pipe was reproduced with PLAXIS using the soil properties from the Variable Moduli Model of Nelson and Baron (1971) and the parameters provided in the paper of Mohri and Kawabata (1995). In addition, a comparison was done between an analysis with interface elements and an analysis without interface elements. Finally, the effect of the measured shear strength values of a PVC-soil interface on the calculated deformations of a buried pipe was evaluated.

1.5 Outline of the Thesis

Chapter 2 presents a review of literature on the analytical methods used to calculate the loads acting on buried flexible pipes and their deformations. Failure modes of plastic pipes and soil-structure interaction behaviour are discussed. In addition, a summary of the field experiment and numerical analysis of Mohri and Kawabata (1995) is presented.

Chapter 3 is a presentation of the capabilities of the PLAXIS finite element code for soil and rock. This section reviews the features included in PLAXIS with an emphasis on the interface element.

Chapter 4 explains the testing procedure including the C3DI apparatus and the materials such as PVC and stone dust. The engineering properties of the stone dust are defined with triaxial tests and sieves analyses. Finally, the test results are presented and discussed.

Chapter 5 describes the Hardening-Soil model, which is the soil model used throughout this study with the PLAXIS finite element code. The input parameters for the Hardening-Soil model are determined from the calculated stress-strain curves using the Variable Moduli Model of Nelson and Baron (1971), which is used by Mohri and Kawabata (1995) in their numerical analysis. Therefore, the Variable Moduli Model is described and applied in this chapter.

Chapter 6 presents the two-dimensional finite element analysis, using PLAXIS, of the buried FRP pipe, which was also analysed by Mohri and Kawabata (1995). The results of both analyses are compared. Analyses with and without interface elements are presented in order to point out the effect of interface elements on the deformations of the pipe and on the earth pressure distributions around the pipe. In addition, Chapter 6 describes the second two-dimensional finite element analysis of the PVC buried pipe. The interface strength is changed throughout this analysis in order to find out its effect on the deformations and on the earth pressure distributions around the pipe.

Chapter 7 presents an evaluation of current analytical methods used to predict pipe deformations and stress distributions. Comparisons are made among the results of these methods, the results of the finite element analyses and the field data of Mohri and Kawabata (1995).

Chapter 8 presents a summary, discussions and conclusions.

CHAPTER 2

LITERATURE REVIEW

In the early years of the 20th century, the newly established Iowa Engineering Experiment Station, directed by Anson Marston, began a theoretical and experimental research on structural failure of drainage pipes. The research goals were to establish a method for estimating the load and its distribution on the pipe, and a method for determining the supporting strength of the pipe. The research led to the well-known Marston-Spangler theory, which is still the current design method for most cities in North America.

In the 60's, other researchers such as Burns and Richard (1964) and Høeg (1968) worked on more rigorous theoretical solutions for buried conduits. They extended the original work by Mindlin to an elastic circular pipe placed in a deep infinite elastic media and subjected to vertical soil pressure (Masada, 1996).

The development of computers in the last few decades increased the evolution of sophisticated numerical analysis methods. As a result, analytical tools such as finite element codes are now available for engineers and researchers to calculate stresses and deformations of buried conduits.

2.1 General theories and experiments

2.1.1 Classes of underground conduits

Underground pipes are divided into two major classes for the purpose of load computation. These classes are ditch conduit and projecting conduit and are based on the construction methods that influence the distribution of the load on the pipe. The projecting conduit class is subdivided into positive projecting conduit and negative projecting conduit. Figure 2.1 illustrates the general types of installation.

Ditch conduit: The ditch conduit is defined as a pipe installed in a relatively narrow trench in undisturbed soil, and then backfilled.

Projecting conduit: The projecting conduit is defined as a pipe installed above the natural ground surface or in a relatively narrow and shallow trench, and then covered with an embankment.

Positive projecting conduit: This subdivision of the projecting conduit includes a conduit that is installed in shallow bedding with its crown projecting above the natural ground surface.

Negative projecting conduit: Conduit that is installed in relatively narrow and shallow trench with its crown remaining below the natural ground surface.

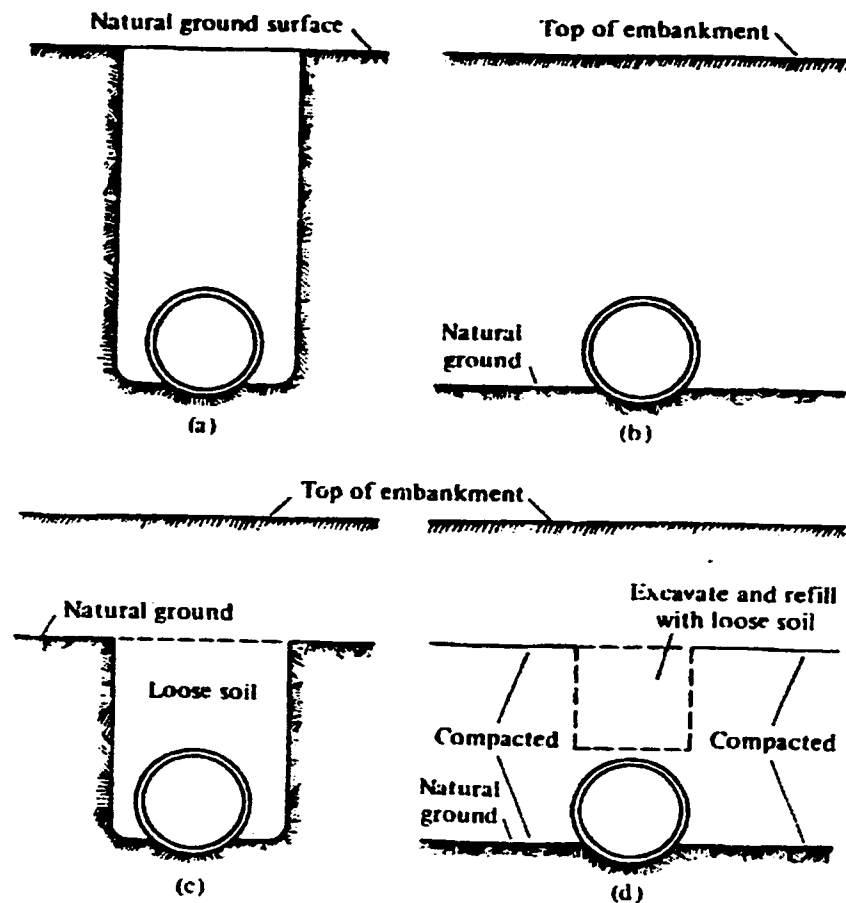


Figure 2.1: a) Trench b) Positive projecting c) Negative projecting d) Imperfect ditch (Spangler, Handy, 1982).

Imperfect ditch conduit or induced trench conduit: This is a special case of the negative projecting conduit. In fact, it is an artificially made negative projecting conduit. First, a certain height of well-compacted embankment covered the pipe. Second, a narrow ditch is excavated over the pipe and backfill with loose soil. Finally, the embankment is completed to its final height.

2.1.2 Buried pipe installation in trench

Arching effect

Load on a buried pipe is not exactly equal to the weight of the soil over the pipe, which is called the soil prism. The weight of the soil prism is found to be the pipe outside diameter times the height of earth above the pipe times the unit weight of the earth. The load acting on a pipe depends on the movement of the soil prism relative to the soil on the sides. In the case of ditch conduit, the backfill material and the pipe have a tendency to settle downward relative to the sides of the trench. This relative movement mobilises shearing forces, which act upward, along the sides of the trench. These shearing forces, associated with horizontal forces from the sides of the trench, reduce the vertical load on the pipe. This phenomenon is called the arching effect and it is as permanent as any other form of shear resistance (Petroff, 1990). See section 2.2.4 for a mathematical description.

Terzaghi summarised theories of arching, and did experimental works using a deflecting trapdoor in the base of a soil bin. In addition, based on the trapdoor experiments, Janssen assumed that the shearing planes that occur when the door is deflected downward are vertical (Bulson, 1985). He also assumed that the vertical pressure on the yield element is equal to the difference between the weight of the soil above the element and the frictional resistance along the side of the element.

Load on ditch conduit

Marston applied the arching analysis of Janssen, replacing the trapdoor width by the width of the ditch, in order to determine the vertical load on a pipe buried in a narrow trench. From the free body diagram shown in Figure 2.2, and taking the cohesion as zero, the formula for vertical load on top of the conduit was developed:

$$V = \gamma B_d^2 \frac{1 - e^{-2K\mu'(h/B_d)}}{2K\mu'} \quad (2.1)$$

V : Vertical load on any horizontal plane in the backfill (N/m)

γ : Unit weight of the backfill material (N/m³)

B_d : Horizontal width of the ditch at the crown (m)

h : Distance from the ground surface to any horizontal plane in the backfill (m)

K : Ratio of active lateral pressure to vertical pressure (Rankine's ratio)

ϕ : Friction angle of the fill material (°)

ϕ' : Friction angle between the fill material and the sides of the ditch (°)

μ : Coefficient of internal friction of the fill material ($\tan \phi$)

μ' : Coefficient of friction between the fill material and the sides of the ditch ($\tan \phi'$)

$$\text{Rankine's ratio: } K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} = \frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2(45^\circ - \frac{\phi}{2})$$

Wetzel proposed that the value of K should be 0.5 for loose fill and 1.0 for dense fill (Bulson, 1985). Christensen (1967), who undertook a further study of the value of K , proposed the following formula:

$$K = (1 + 2 \tan^2 \phi)^{-1} = \frac{1 - \sin^2 \phi}{1 + \sin^2 \phi'}$$

The portion of the total load that is carried by the pipe depends on the rigidity of the pipe. In the case of a rigid pipe, the side fills may move downward relative to the soil prism causing the pipe to sustain the entire load V . In the case of a relatively flexible pipe, the soil prism may move downward relative to the side fills, because of the deflection of the pipe, causing the pipe to sustain a reduced load. As a result, Marston determined the load of a rigid ditch conduit by the following formula, which is used with the computation diagram in Figure 2.3.

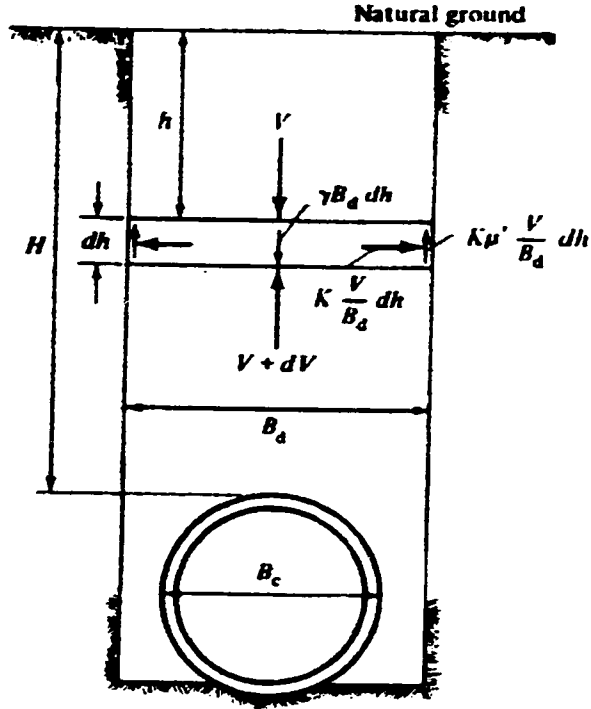


Figure 2.2: Free body diagram for ditch conduit (Spangler and Handy 1982).

$$W_c = C_d \gamma B_d^2 \quad C_d = \frac{1 - e^{-2K\mu'(H/B_d)}}{2K\mu'} \quad (2.2)$$

The formula for the relatively flexible pipe is the following:

$$W_c = C_d \gamma B_d B_c \quad (2.3)$$

B_c : Outside diameter of the conduit (m)

There is still a controversy about the differentiation between rigid, semi-flexible and flexible pipes. The ASTM standards define flexible pipes as pipes that deflect more than 2% of their respective diameter without any sign of structural distress.

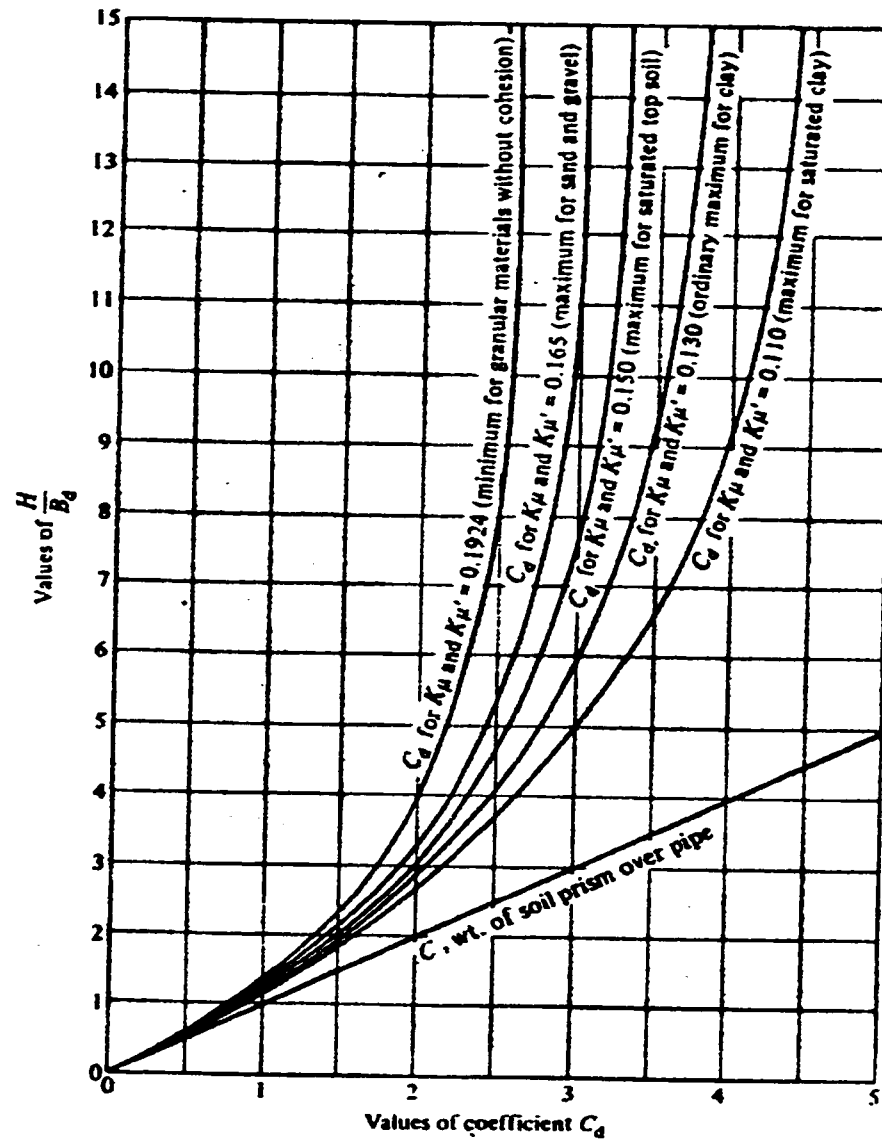


Figure 2.3: Computation diagram for C_d (Spangler and Handy 1982).

These two formulas (2.2, 2.3) give the maximum load on any particular pipe in service. Experiments and field observations have demonstrated that the pipe may not reach the maximum load for a long period of time. In fact, the load keeps building up for an extended period of time after the completion of the backfill (Spangler and Handy 1982).

Load on ditch conduit with sloping side

The width of the ditch (B_d) in the previous formulas is the width of a normal trench with vertical sides. In the case of trenches with sloping side, the parameter B_d must be the width of the trench at the crown of the pipe (Schlick, 1932).

2.1.3 Buried pipe installation in embankment

Positive projecting conduit

As mentioned before, the crown of a positive projecting conduit projects over the natural ground surface. The distance from the pipe's crown to the ground surface is expressed by the term pB_c , where p is the projection ratio. The planes where the shear forces act in a positive projecting conduit are assumed to be the vertical planes extending upward from the sides of the conduit. Figures 2.4 and 2.5 illustrate two patterns of settlements that influence load on positive projecting conduits.

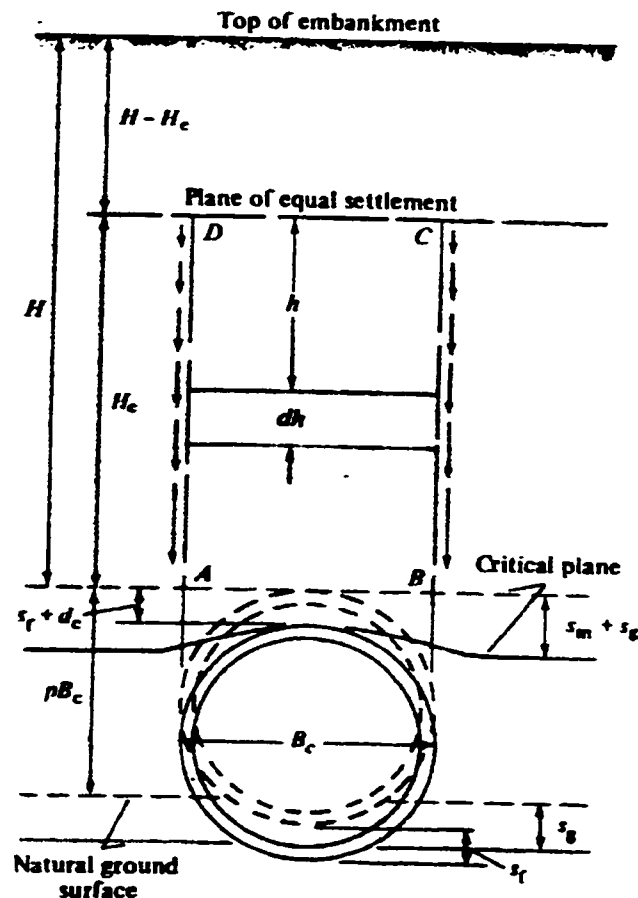


Figure 2.4: Settlements that influence load on projecting conduit (Spangler and Handy 1982).

The rigidity of the pipe has a large effect on the vertical load applied on the pipe. The magnitude and direction of the movement of the soil prism ABCD (Figures 2.4 and 2.5) relative to the side fills, are influenced by the settlements of the natural ground, the settlement of the side fills and the deformation of the pipe. These settlements and the pipe's deflection are combined in a settlement ratio defined in equation 2.4. The values of settlement ratio for design purposes are presented in Table 2.1.

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \quad (2.4)$$

- r_{sd} : Settlement ratio
 s_m : Settlement of the side fill of height pB_c
 s_g : Settlement of the natural ground surface adjacent to the conduit
 s_f : Settlement of the conduit into its foundation
 d_c : Pipe's deflection

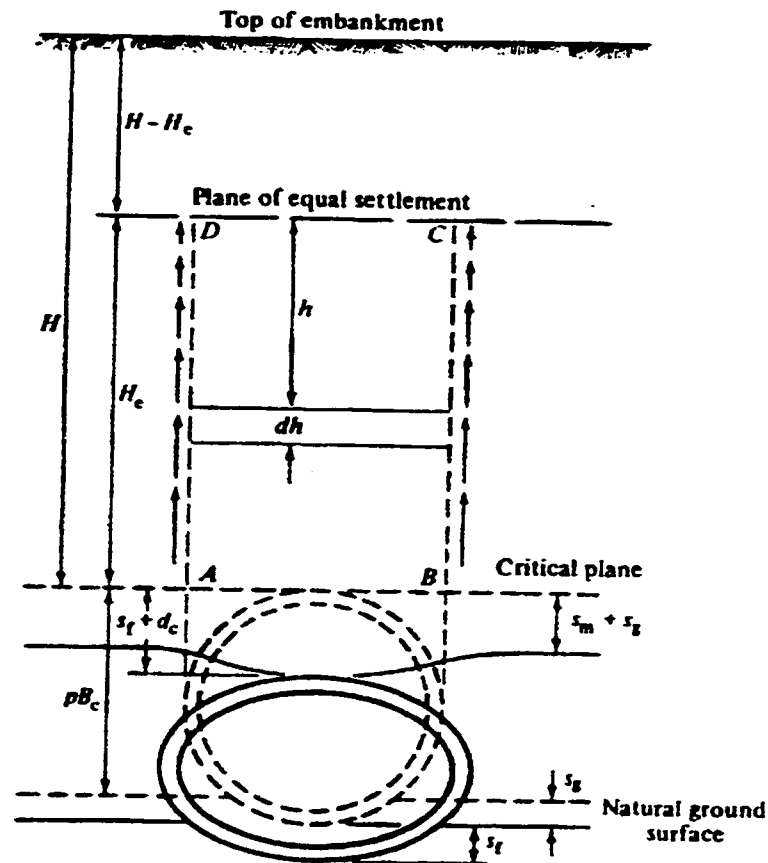


Figure 2.5: Settlements that influence load on projecting conduit (Spangler and Handy 1982).

Table 2.1: Design values of settlement ratio (Spangler and Handy 1982).

Conditions	Settlement ratio
Rigid culvert on foundation of rock	+1.0
Rigid culvert on foundation of ordinary soil	+0.5 to +0.8
Rigid culvert on foundation of material that yield with respect to adjacent natural ground	0.0 to +0.5
Flexible culvert with poorly compacted side fills	-0.4 to 0.0
Flexible culvert with well-compacted side fills	-0.2 to +0.2

A critical plane is related to the settlement ratio. The critical plane is a horizontal plane passing through the crown as shown in Figure 2.5. After the completion of the backfill, the critical plane moves downward. If it moves below the crown, then the settlement ratio is positive and the vertical load on the pipe is greater than the weight of the soil prism. On the other hand, if the critical plane settles less than the crown, then the settlement ratio is negative and the vertical load on the pipe is reduced.

In the case of projecting conduit, the vertical shear planes may not extend up to the top of the embankment (Fig. 2.5). The horizontal plane where the vertical shear planes extend is called the plane of equal settlement. Over this plane, the soil prism above the conduit settles at the same rate as the side fills (Spangler and Handy, 1982). When the height of the plane of equal settlement above the crown, which is designated as H_e , is less than the total height of the embankment H , then this is called the incomplete ditch or incomplete projection condition. If H_e is greater than or equal to H , then this is called the complete ditch or complete projection condition.

Marston derived formulas for the vertical load on positive projecting conduit. For the complete ditch or projection condition the formula is

$$W_c = C_c \gamma B_c^2 \quad C_c = \frac{e^{\pm 2K\mu(H/B_c)} - 1}{\pm 2K\mu} \quad (2.5)$$

C_c : Load coefficient for positive projecting conduit. Figure 2.6 presents a computation diagram for C_c .

All other parameters in Equation 2.5 have the same meaning as defined in Equation 2.1. The plus sign is used for the complete projection condition and the minus sign is used for the complete ditch condition.

The load coefficient for the incomplete ditch projection condition is:

$$C_c = \frac{e^{\pm 2K\mu(H/B_c)} - 1}{\pm 2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c} \right) e^{\pm 2K\mu(H_e/B_c)} \quad (2.6)$$

Again, the plus sign is used for incomplete projection condition and the minus sign is used for the incomplete ditch condition.

Marston also develop a formula for evaluating H_e :

$$\left[\frac{1}{2K\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c} \right) \pm \frac{r_{sd}P}{3} \right] \frac{e^{\pm 2K\mu(H_e/B_c)} - 1}{\pm 2K\mu} \pm \frac{1}{2} \left(\frac{H_e}{B_c} \right)^2 \pm \frac{r_{sd}P}{3} \left(\frac{H}{B_c} - \frac{H_e}{B_c} \right) e^{\pm 2K\mu(H_e/B_c)} - \frac{1}{2K\mu} \cdot \frac{H_e}{B_c} \mp \frac{H}{B_c} \cdot \frac{H_e}{B_c} = \pm r_{sd}P \frac{H}{B_c} \quad (2.7)$$

The only unknown parameter in this equation is H_e , then the equation is solved by trial and error. The plus signs are used for the incomplete projection condition and the minus signs are used for the incomplete ditch condition. Wastlund and Eggwertz proposed another method to find H_e (Bulson, 1985).

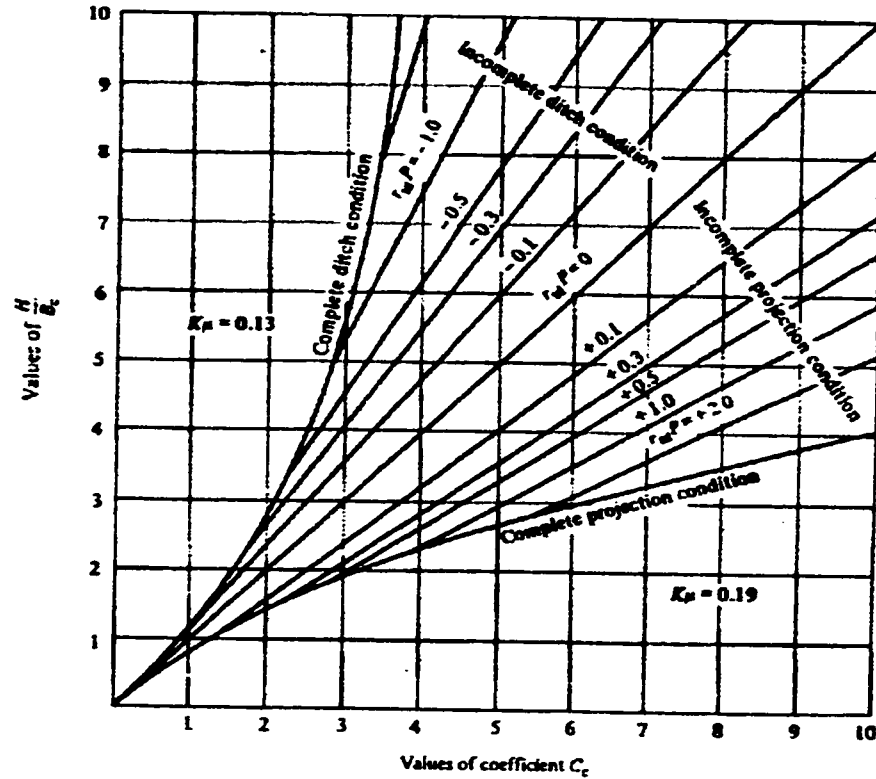


Figure 2.6: Computation diagram for C_c (Spangler and Handy 1982).

Imperfect ditch conduit

In the imperfect ditch conduit, illustrated in Figure 2.7, the objective is that the soil prism above the pipe will settle more than the side fills. The imperfect ditch system includes a second projection ratio, which is designated by p' . The projection ratio p' is defined as the depth of the ditch divided by its width. In addition, the critical plane is now the horizontal plane in the trench backfill material at the level of the compacted backfill surface. The settlement ratio is given by the following formula:

$$r_{sd} = \frac{s_g - (s_d + s_f + d_c)}{s_d} \quad (2.8)$$

s_d : Settlement of the fill in ditch within height $p'B_c$

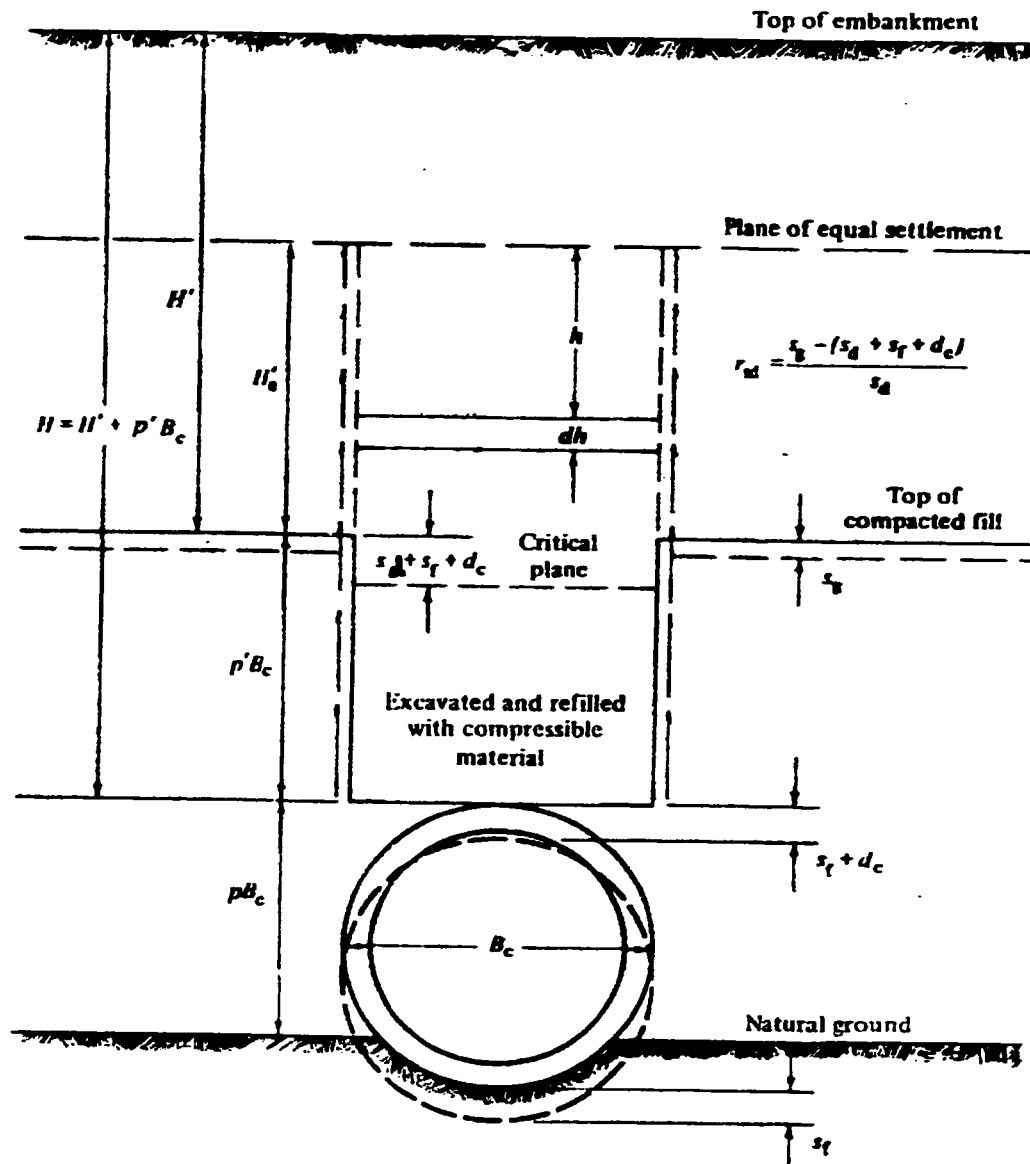


Figure 2.7: Settlements that influence load on imperfect ditch conduit (Spangler and Handy 1982).

Spangler and Handy (1982) stated that "...it is tentatively recommended that this ratio [the settlement ratio] be assumed to lie between -0.3 and -0.5 for the purpose of estimating loads."

The formula for the calculation of the vertical load on imperfect ditch conduit is

$$W_c = C_n \gamma B_c^2 \quad (2.9)$$

C_n : Load coefficient for imperfect ditch conduit.

Figure 2.8 presents a computation diagram for the load coefficient for imperfect ditch conduit and negative projecting conduit C_n when $p'=0.5$. Other diagrams when $p'=1.0$, 1.5, and 2.0 can be found in Spangler and Handy (1982).

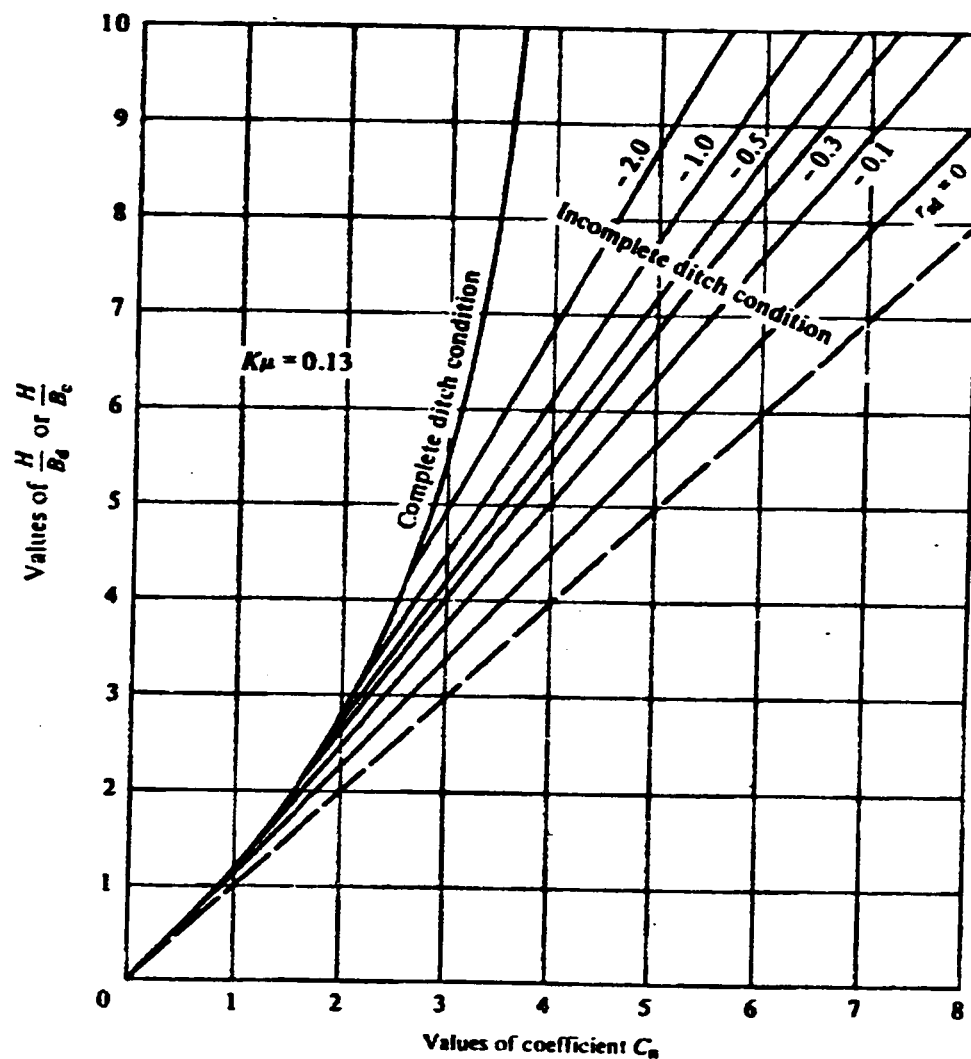


Figure 2.8: Computation diagram for C_n (Spangler and Handy 1982).

Negative projecting conduit

The load on negative projecting conduit is determined using the same procedure as for the imperfect ditch conduit. Spangler (1950) presented the complete theory on loads on negative projecting conduits.

Conduits in wide trenches

Research on the trench theory undertaken by Schlick (1932) led to the definition of a transition width. The transition width is defined as the width at which the load calculated from the projecting theory is equal to the load calculated from the trench theory. Figure 2.9 shows curves for transitional-width ratio. The transition width is a function of the depth of cover H , and becomes smaller as the depth of cover increases. In the application of transitional width concept, it is suggested to use the product of the settlement ratio and the projection ratio ($r_{sd}p$) equal to 0.5 (Spangler and Handy, 1982).

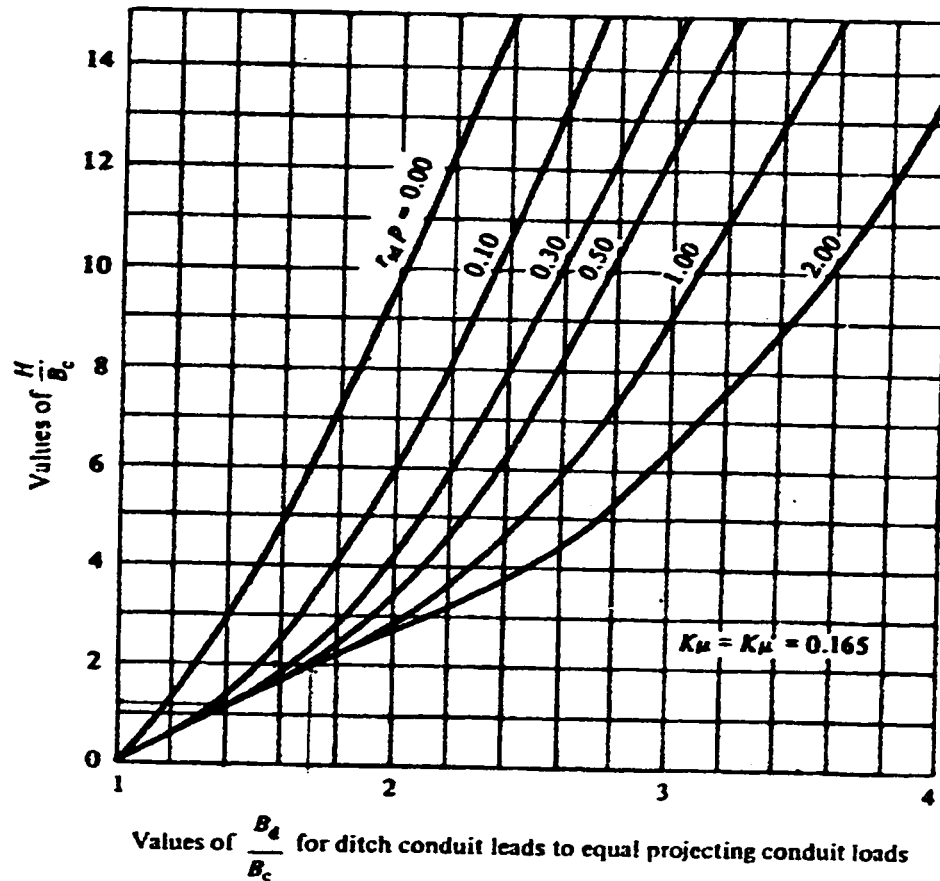


Figure 2.9: Curves for transitional width ratio (Spangler and Handy 1982).

2.1.4 Surface loads

Surface concentrated load

Wheel loads from trucks, airplanes or trains cause concentrated loads on buried pipes. The AASHTO Bridge Specification has a simplified procedure for determining the distribution of load in the ground resulting from concentrated loads on the ground surface. In this procedure, the load is assumed to attenuate with increasing depth at an angle of 41° with the vertical in each direction. The American Concrete Pipe Association (ACPA) design practice distributes the earth pressure at the crown on a larger area in the longitudinal direction of the pipe because of the beam strength of the pipe in this specific direction. It is also possible to use Boussinesq formulas to determine the distribution of the load in the ground. The American Water Works Association has its design approach for flexible pipe (AWWA C950).

Distributed surface surcharge loads

The distributed surface surcharge loads may be calculated as an equivalent additional layer of soil.

2.1.5 lowa deflection formula

A flexible pipe may be defined as a conduit that will deflect at least 2% without any sign of failure or cracks in normal loading conditions (Uni-Bell, 1993). In fact, the flexible pipe develops its load-carrying capacity from its flexibility. As explained in the previous sections, the capability of the pipe to deflect under load causes the pipe to avoid a major portion of the total vertical load. In addition, it develops passive soil support at the sides of the pipe when entering in flattening mode.

Pipe stiffness

The resistance of a flexible pipe to applied loads is related to the pipe stiffness, which is measured according to ASTM D 2412: "Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading". The equation that defines the pipe stiffness is

$$PS = \frac{F}{\Delta Y} = \frac{EI}{0.149r^3} \quad (2.10)$$

For a solid pipe wall of unit length, the moment of inertia is

$$I = \frac{t^3}{12} \quad (2.11)$$

PS : Pipe stiffness

F : Applied load

ΔY : Measured change of the vertical inside diameter

r : Mid-wall radius

EI : Stiffness factor

I : Moment of inertia

t : Thickness of the pipe

Spangler's deflection formula (Spangler, 1941)

Spangler first established some relationships to define the capability of a flexible pipe to resist ring deflection when not buried in the soil. There are three relations:

$$\Delta Y = \frac{0.149Fr^3}{EI} \quad (2.12)$$

$$\Delta X = \frac{0.136Fr^3}{EI} \quad (2.13)$$

$$\Delta X = 0.913\Delta Y \quad (2.14)$$

The symbols are the same as for equation 2.10 excepted for ΔX , which is the change in horizontal diameter.

Second, Spangler incorporated the effects of the surrounding soil on the pipe's deflection. In order to do that, Spangler used the Marston theory for the determination of the load applied on the pipe. He assumed the vertical load and reaction load uniformly distributed. He also assumed that the horizontal pressure is distributed parabolically over the middle 100°, and that the maximum unit pressure on the side fills is equal to the modulus of passive resistance of the backfill material multiplied by one-half the horizontal deflection of the pipe. Based on Figure 2.10, Spangler derived his formula for deflection of buried pipes.

$$\Delta X = D_l \frac{KW_c r^3}{EI + 0.061er^4} \quad (2.15)$$

- D_l : Deflection lag factor
 K : Bedding constant (function of a bedding angle, Table 2.2)
 W_c : Marston's load per unit length of pipe
 r : Mean radius of the pipe
 E : Modulus of elasticity of the pipe material
 I : Moment of inertia of the pipe wall per unit length of pipe
 e : Empirical modulus of passive resistance of the side fill further modified by Watkins
 ΔX : Horizontal deflection

The original Iowa formula includes three empirical constants: K , D_l and e . The bedding constant, K , is related to the angle of the vertical support of the pipe, ψ , or the uniform soil reaction on the pipe, P , due to the overburden pressure, W . The deflection lag factor, D_l , considers the consolidation of the side fills with time. The magnitude of the deflection lag factor for a conservative design practice should be 1.5, as recommended by Spangler (1941). The experience of Spangler had shown that deflections could increase by as much as 30% over 40 years (Uni-Bell, 1993). The empirical constant e will be discussed later on.

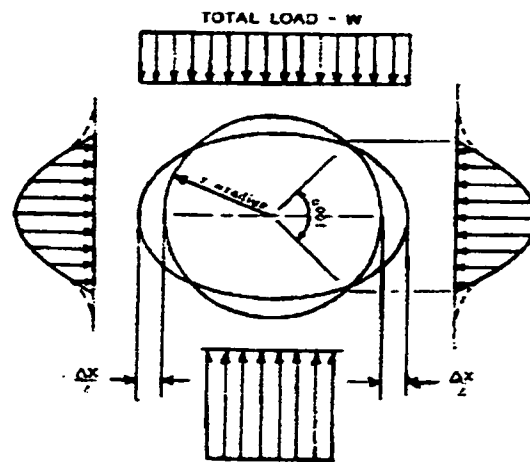


Figure 2.10: Basis of Spangler's derivation of the Iowa formula (Uni-Bell, 1993).

Table 2.2: Value of bedding constant K (Uni-Bell, 1993).

Bedding angle ($^{\circ}$)	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

2.1.6 Modified Iowa deflection formula

In 1955, Watkins investigated the value of the modulus of passive resistance and found that it could not be a true property of the soil. As a result, Watkins defined a new parameter named the modulus of soil reaction, $E' = er$. Meyerhof and Barnard proposed a coefficient of lateral earth pressure as an alternative (Bulson, 1985). From the work of Watkins, the modified Iowa formula was written:

$$\Delta X = D_i \frac{KW_e r^3}{EI + 0.061E'r^3} \quad (2.16)$$

Table 2.3 presents average values of the modulus of soil reaction, E' , proposed by Howard (1977). The accuracy of the theoretical deflection using the values of Howard is +/- 2%.

Table 2.3: Average values of modulus of soil reaction, E' (Uni-Bell, 1993).

Soil type-pipe bedding material (Unified Classification System ^a) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch			
	Dumped (2)	Slight, <85% Proctor, <40% relative density (3)	Moderate, 85%-95% Proctor, 40%-70% relative density (4)	High, >95% Proctor, >70% relative density (5)
Fine-grained Soils (LL > 50) ^b Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer. Otherwise use E' = 0			
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with less than 25% coarse- grained particles	50	200	400	1,000
Fine-grained Soils (LL < 50) Soils with medium to no plasticity, CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained Soils with Fines GM, GC, SM, SC ^c contains more than 12% fines	100	400	1,000	2,000
Coarse-grained Soils with Little or no Fines GW, GP, SW, SP ^c contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection ^d	±2	±2	±1	±0.5

^aASTM Designation D 2487, USBR Designation E-3.

^bLL = Liquid limit.

^cOr any borderline soil beginning with one of these symbols (i.e. GM-GC, GC-SC).

^dFor ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

Note: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m³) (ASTM D 698, AASHTO T-99, USBR Designation E-11). 1 psi = 6.9 kPa.

SOURCE: "Soil Reaction for Buried Flexible Pipe" by Amster K. Howard, U.S. Bureau of Reclamation, Denver, Colorado. Reprinted with permission from American Society of Civil Engineers.

2.1.7 Modified Iowa deflection formula by Greenwood and Lang

The Marston-Spangler theory for flexible pipe was intended to predict the deflection of flexible steel pipes under overburden pressures. Some well-known researchers such as Spangler (1941), Watkins (1958) and Howard (1977) have done several experiments, which have proven that the theory works. However, the modified Iowa formula does not take into account the stages of construction and the pipe-soil interface behaviour in the calculation of the total deflection of a pipe.

The parameters that affect pipe deflection are the following:

- Pipe stiffness
- Soil resistance (type, density, modulus, moisture content)
- Applied loads
- Trench configuration (geometry and embedment)
- Haunch support
- Construction stages
- Time
- Temperature
- Variability in construction procedures and in soil characteristics

The effect of time, temperature and variability in input parameters are not considered in this thesis. Spangler's Iowa formula does not take into account the non-elliptical deformation and the initial deformation due to construction stages. As it is demonstrated in chapters 7 and 8, the vertical elongation of a flexible pipe is significant. It is so significant that the earth pressure at the completion of the backfill may not cause the pipe to extend horizontally. The non-elliptical deformation is a result of the non-uniform earth pressure around the pipe. Greenwood and Lang (1990) found out that the non-uniformity is a function of soil type, degree of compaction, split embedment and pipe stiffness.

Greenwood and Lang (1990) also presented a modified Iowa formula, which is more complete than the original formula. Their modified Iowa formula includes the work of Leonhardt (1972-79) who developed a factor to consider the soil resistance of the native soil. The factor, which is applied to the soil resistance parameter in the modified Iowa formula of Watkins, is a function of the trench width to pipe diameter ratio and the embedment (backfill soil around the pipe) modulus to native soil modulus. A pipe-soil interaction coefficient, which is an empirical factor, is added to the soil resistance term to reflect the behaviour of flexible pipes in the field. The modified Iowa formula of Greenwood and Lang (1990) is presented below.

$$\Delta X = \frac{K(\gamma H)}{EI / r^3 + 0.0615 C_I E'} - \delta_{vo} \quad (2.17)$$

- Δx : Horizontal deformation
 K : Bedding factor (From Table 2.4)
 γ : Unit weight of the backfill
 H : Height of the backfill above the pipe
 E : Modulus of elasticity of pipe material
 I : Moment of inertia of the pipe wall per unit length of pipe
 r : Mean radius of the pipe
 E' : Watkins' modulus of soil reaction
 δ_{vo} : Elongation due to compaction of the side fills
 D : Pipe diameter
 C_I : Pipe-soil interaction coefficient defined by Greenwood and Lang (1990)

$$C_I = a \left(\frac{EI}{1250 \cdot D^3} \right)^b$$

- a, b : Parameters provided in Table 2.5

ζ : Leonhardt relationship

$$\zeta = \frac{1.662 + 0.639(B/D - 1)}{(B/D - 1) + [1.662 - 0.361(B/D) - 1]E_2 / E_3}$$

E_2 : Soil modulus of the embedment

E_3 : Soil modulus of the native soil

B : Trench width

Table 2.4: Bedding factor values (Greenwood and Lang, 1990)

Soil group	Range of fines %	Backfill standard Proctor density			
		>95	85-95	70-84	<70
Clean gravel	<5	0.083	0.083	0.083	0.083
	5-12	0.096	0.096	0.083	0.083
Dirty gravel	12-50	0.103	0.103	0.096	0.083
Clean sand	<5	0.103	0.103	0.096	0.083
	5-12	0.103	0.103	0.096	0.083
Dirty sand	12-50	0.103	0.103	0.096	0.083
		0.103	0.103	0.096	0.083
Inorganic clay and silt	>50	0.103	0.103	0.096	0.083

Table 2.5: Values of parameters a and b for the pipe-soil interaction coefficient (Greenwood and Lang, 1990)

Backfill standard Proctor density	a	b
>95	1.24	0.180
85-95	0.938	0.245
70-84	0.643	0.353
<70	0.456	0.436

2.1.8 Watkins soil-strain theory

Watkins theory starts from the Iowa formula written in dimensionless ratios:

$$\frac{\Delta Y}{D} = \frac{PR_s}{E_s AR_s + B} \quad (2.18)$$

- P : Vertical pressure at the top of the pipe
 R_s : Stiffness ratio: $12(E_s D^3 / E_f^3)$
 E_s : Slope of stress-strain curve for soil in one-dimensional consolidation test: P / ϵ
 ϵ : Vertical soil strain
 A, B : Empirical constants including D_f and K in the Iowa formula
 ΔY : Measured change in vertical diameter
 D : Outside diameter

The equation 2.18 can be rewritten as

$$\frac{\Delta Y}{D \epsilon} = \frac{R_s}{AR_s + B} \quad (2.19)$$

The empirical constant B also includes the value of E_s in equation 2.19. However, as discussed below, it is not necessary to calculate the values of A and B .

Figure 2.11 is a graph based on empirical data, which gives the ring deflection factor $((\Delta Y / D) / \epsilon)$ as a function of stiffness ratio. Watkins observed that usually the deflection factor approaches the unity because the stiffness ratio is usually greater than 300. As a result, the ring deflection becomes about as much as the side fills settlement. It is then possible to evaluate the pipe's deflection from the vertical soil strain in the fill. Figure 2.12 presents vertical soil strain values as a function of soil compressibility and soil pressure.

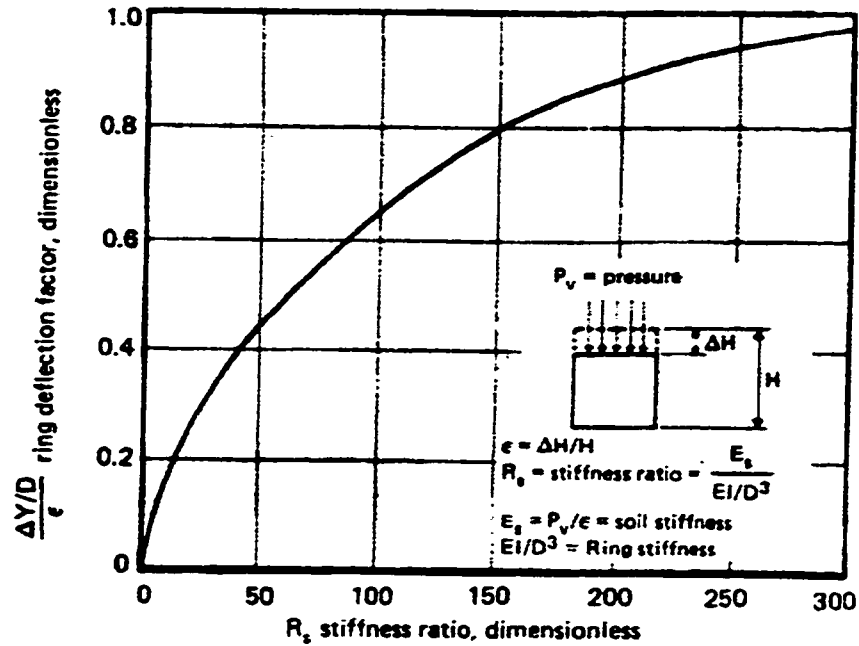


Figure 2.11: Ring deflection factor as a function of stiffness ratio (Moser, 1990)

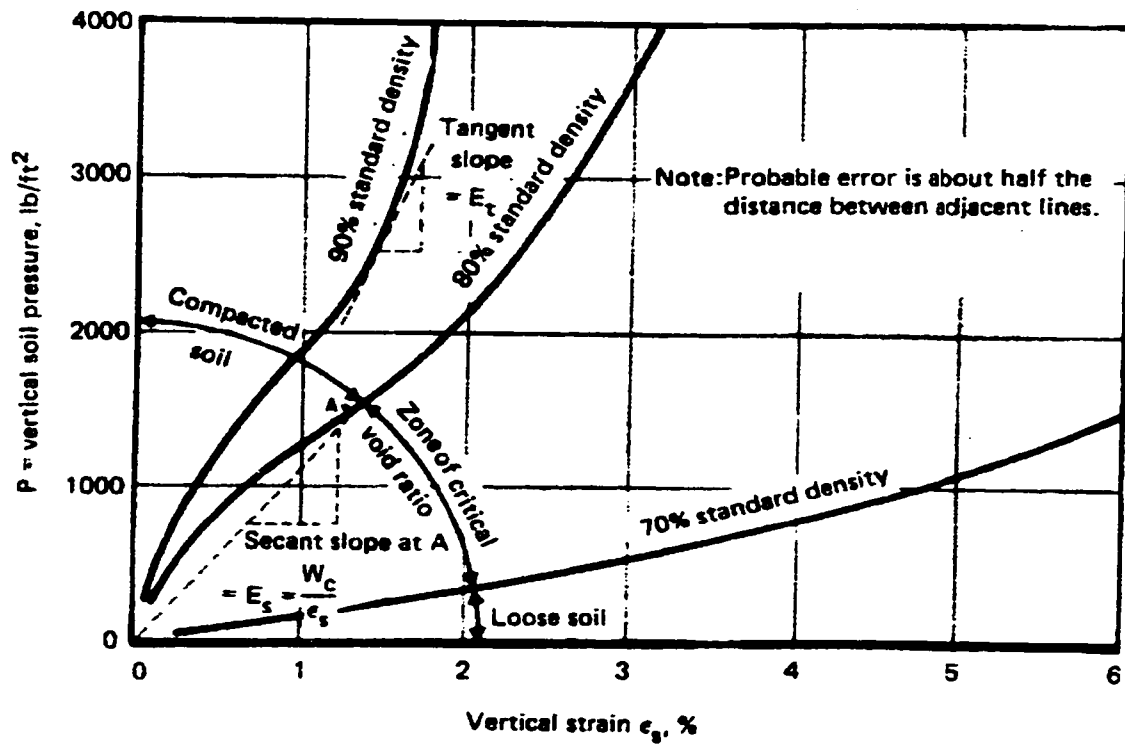


Figure 2.12: Vertical stress strain data for typical trench backfill (Moser, 1990)

2.1.9 Elastic solution by Burns and Richard

Burns and Richard (1964) published a theoretical solution for an elastic pipe placed in an infinite elastic medium and subjected to vertical and horizontal load. The Iowa formula is also a theoretical solution, but Spangler developed it on the basis of several field observations. Figures 2.13 and 2.14 illustrate the elastic pipe/soil problem considered by Burns and Richard. The solution was based on the condition of full-bond or free-slip at the soil/pipe interface. In both conditions, it was assumed that no gap takes place at the pipe/soil interface. A summary of their solution is presented below:

$$\text{Extensional flexibility ratio: } UF = 2B \frac{M^* R}{EA} = (1 + K) \frac{M^* R}{EA} \quad (2.22)$$

$$\text{Bending flexibility ratio: } VF = 2C \frac{M^* R^3}{6EI} = (1 - K) \frac{M^* R^3}{6EI} \quad (2.23)$$

$$M^* = \frac{E^* (1 - \mu)}{(1 + \mu)(1 - 2\mu)}$$

$$B = \frac{1}{2}(1 + K) = \frac{1}{2} \left(\frac{1}{1 - \mu} \right) \quad C = \frac{1}{2}(1 - K) = \frac{1}{2} \left(\frac{1 - 2\mu}{1 - \mu} \right) \quad K = \frac{\mu}{1 - \mu}$$

Full-Bonding interface case:

$$P_r = p \{ B[1 - a_o^*] - C[1 - 3a_2^* - 4b_2^*] \cos 2\theta \} \quad (2.24)$$

$$T_{r\theta} = p \{ C[1 + 3a_2^* + 2b_2^*] \sin 2\theta \} \quad (2.25)$$

$$v = \frac{pR}{2M^*} \left\{ \left[1 - a_2^* + \left(\frac{2C}{B} \right) b_2^* \right] \sin 2\theta \right\} \quad (2.26)$$

$$w = \frac{pR}{2M^*} \{ UF[1 - a_o^*] - VF[1 - a_2^* - 2b_2^*] \cos 2\theta \} \quad (2.27)$$

$$N = pR \{ B[1 - a_o^*] + C[1 + a_2^*] \cos 2\theta \} \quad (2.28)$$

$$M = pR^2 \left\{ \frac{CUF}{6VF} [1 - a_o^*] + \frac{C}{2} [1 - a_2^* - 2b_2^*] \cos 2\theta \right\} \quad (2.29)$$

$$a_o^* = \frac{UF - 1}{UF + (B/C)} \quad a_2^* = \frac{C(1 - UF)VF - (C/B)UF + 2B}{(1 + B)VF + C(VF + 1/B)UF + 2(1 + C)}$$

$$b_2^* = \frac{(B + CUF)VF - 2B}{(1 + B)VF + C(VF + 1/B)UF + 2(1 + C)}$$

Free slip interface case:

$$P_r = p\{B[1 - a_o^*] - C[1 - 3a_2^{**} - 4b_2^{**}]\cos 2\vartheta\} \quad (2.30)$$

$$v = \frac{pR}{6M^*}\{VF + (C/2B)UF[1 + 3a_2^{**} - 4b_2^{**}]\sin 2\vartheta\} \quad (2.31)$$

$$w = \frac{pR}{2M^*}\left\{UF[1 - a_o^*] - \frac{2}{3}VF[1 - a_2^{**} - 2b_2^{**}]\cos 2\vartheta\right\} \quad (2.32)$$

$$N = pR\left\{B[1 - a_o^*] + \frac{C}{3}[1 + a_2^{**} - 4b_2^{**}]\cos 2\vartheta\right\} \quad (2.33)$$

$$M = pR^2\left\{\frac{CUF}{6VF}[1 - a_o^*] + \frac{C}{3}[1 - a_2^{**} - 4b_2^{**}]\cos 2\vartheta\right\} \quad (2.34)$$

$$a_2^{**} = \frac{2VF - 1 + 1/B}{2VF - 1 + 3/B} \quad b_2^{**} = \frac{2VF - 1}{2VF - 1 + 3/B}$$

- I : Moment of inertia of the pipe
- R : Mean radius of the pipe
- A : Area of the section of the pipe wall per unit length
- μ : Poisson's ratio for medium
- E : Young's modulus for material in cylinder wall
- E^* : Young's modulus of the medium
- M : Bending moment in pipe wall
- N : Thrust in pipe wall
- P_r : Radial stress
- T_ϑ : Tangential stress
- p : Applied vertical boundary pressure
- r, ϑ, z : Cylindrical coordinates
- w : Radial displacement in medium
- v : Tangential displacement

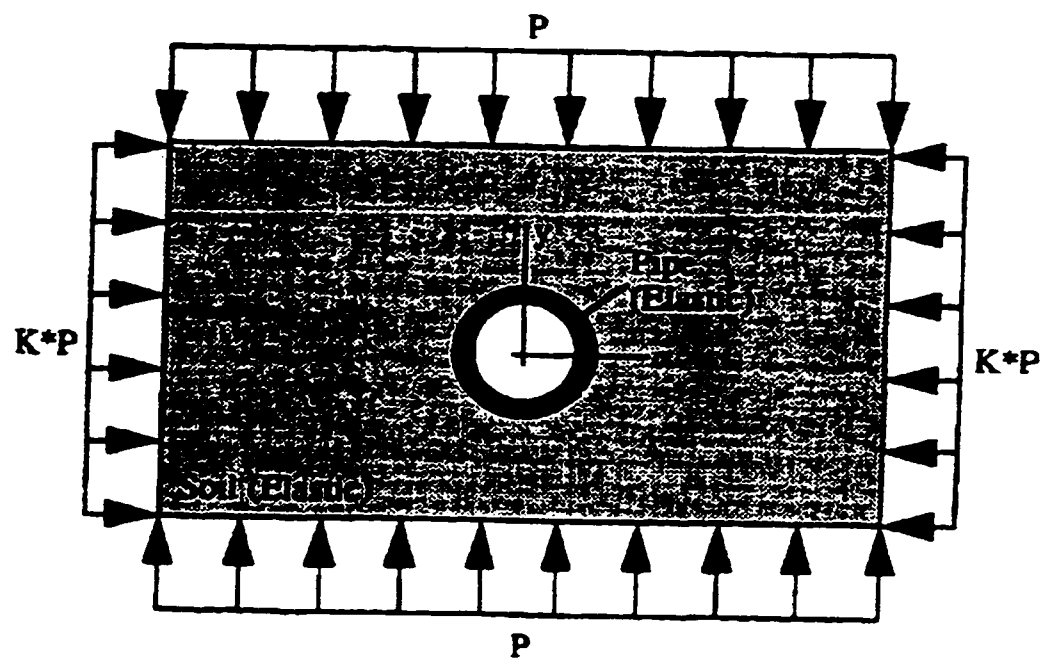


Figure 2.13: Elastic pipe/soil problem considered by Burns and Richard (Masada, 1996)

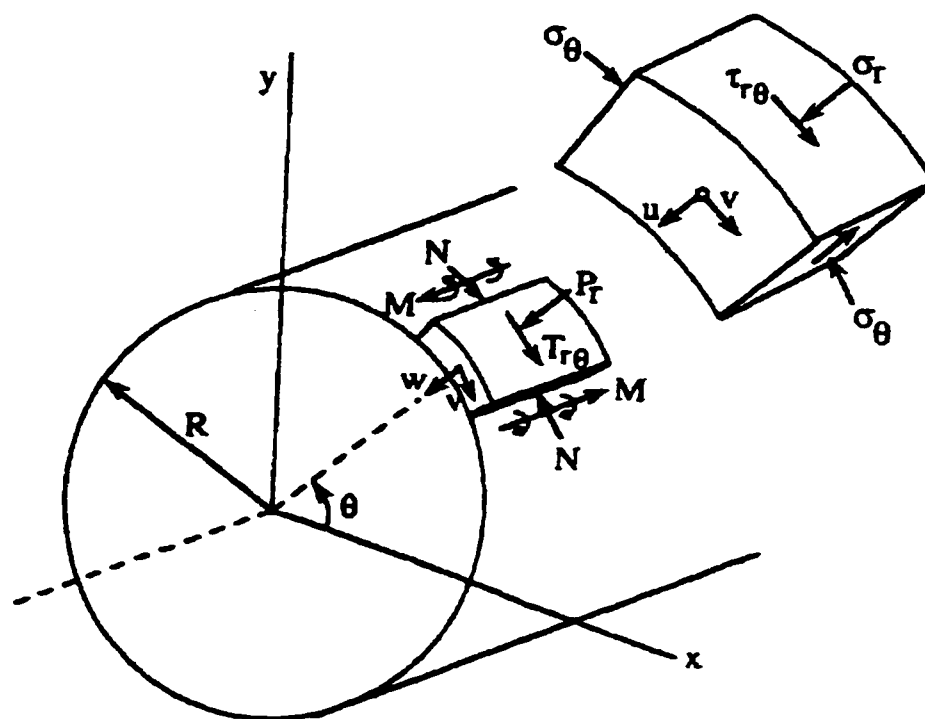


Figure 2.14: Elastic pipe/soil problem considered by Burns and Richard (Masada, 1996)

2.1.10 Elastic solution by Höeg

Höeg (1968) proposed an elastic solution to analyse the magnitude and distribution of static stresses around horizontal cylinders. The study was limited to plane strain condition. The soil was assumed to behave like a linearly elastic, isotropic and homogeneous material. The pipe is also assumed to be elastic material. In addition, his solution considers two extreme interface conditions: full bonding and free slip. A summary of his solution is presented below.

The mathematical solution is expressed in terms of two stiffness ratios. The compressibility ratio (C), which is the compressibility of the structural cylinder relative to compressibility the solid soil cylinder, and the flexibility ratio (F), which relates the flexibility of the structural cylinder to the compressibility of the solid soil cylinder.

$$C = \frac{1}{2} \frac{1}{1-\nu} \frac{M}{\frac{E_c}{1-\nu_c^2}} \left(\frac{D}{t} \right) \quad (2.35)$$

$$F = \frac{1}{4} \frac{1-2\nu}{1-\nu} \frac{M}{\frac{E_c}{1-\nu_c^2}} \left(\frac{D}{t} \right)^3 \quad (2.36)$$

$$\sigma_r = \frac{1}{2} p \left\{ (1-k) \left[1 - a_1 \left(\frac{R}{r} \right)^2 \right] - (1-k) \left[1 - 3a_2 \left(\frac{R}{r} \right)^4 - 4a_3 \left(\frac{R}{r} \right)^2 \right] \cos 2\theta \right\} \quad (2.37)$$

$$\sigma_\theta = \frac{1}{2} p \left\{ (1-k) \left[1 - a_1 \left(\frac{R}{r} \right)^2 \right] - (1-k) \left[1 - 3a_2 \left(\frac{R}{r} \right)^4 \right] \cos 2\theta \right\} \quad (2.38)$$

$$\tau_{r\theta} = \frac{1}{2} p (1-k) \left[1 + 3a_2 \left(\frac{R}{r} \right)^4 + 2a_3 \left(\frac{R}{r} \right)^2 \right] \sin 2\theta \quad (2.39)$$

$$u = \frac{1}{2} p \frac{r}{M} \left\{ \begin{aligned} & (1+k)(1-\nu) \left[1 + \frac{a_1}{1-2\nu} \left(\frac{R}{r} \right)^2 \right] \\ & - (1-k) \frac{1-\nu}{1-2\nu} \left[1 + a_2 \left(\frac{R}{r} \right)^4 + 4(1-\nu)a_3 \left(\frac{R}{r} \right)^2 \right] \cos 2\theta \end{aligned} \right\} \quad (2.40)$$

$$\nu = \frac{1}{2} p \frac{r}{M} \frac{1-\nu}{1-2\nu} (1-k) \left[1 - a_2 \left(\frac{R}{r} \right)^4 + 2(1-2\nu) a_3 \left(\frac{R}{r} \right)^2 \right] \sin 2\theta \quad (2.41)$$

For the case with no slippage at the interface

$$a_1 = \frac{(1-2\nu)(C-1)}{(1-2\nu)(C+1)}$$

$$a_2 = \frac{(1-2\nu)(1-C)F - \frac{1}{2}(1-2\nu)^2 C + 2}{[(3-2\nu) + (1-2\nu)C]F + \left(\frac{5}{2} - 8\nu + 6\nu^2 \right) C + 6 - 8\nu}$$

$$a_3 = \frac{[1 + (1-2\nu)C]F - \frac{1}{2}(1-2\nu)^2 C - 2}{[(3-2\nu) + (1-2\nu)C]F + \left(\frac{5}{2} - 8\nu + 6\nu^2 \right) C + 6 - 8\nu}$$

For the case with free slippage on the interface

$$a_1 = \frac{(1-2\nu)(C-1)}{(1-2\nu)(C+1)}$$

$$a_2 = \frac{2F + 1 - 2\nu}{2F + 5 - 6\nu}$$

$$a_3 = \frac{2F - 1}{2F + 5 - 6\nu}$$

D : Diameter of the cylinder

t : Cylinder thickness

R : Cylinder radius

$\tau_{r\theta}$: Shear stress

ν : Poisson's ratio for medium

ν_c : Poisson's ratio for material in cylinder wall

E_c : Young's modulus for material in cylinder wall

M : One-dimensional modulus of medium

σ_r : Radial stress

σ_θ : Tangential stress

- p : Applied vertical boundary pressure
 k : Coefficient relating vertical and horizontal stress
 r, θ, z : Cylindrical coordinates
 u : Radial displacement in medium
 v : Tangential displacement

Höeg also realised that the density of the soil may be reduced in the vicinity of the pipe. To take into account such a case, he defined a modified compressibility ratio (C_m), which replaces the compressibility ratio (C) in the previous equations.

$$C_m = C + \left(\frac{h}{R+h} \right) \left(\frac{M}{M'} \right) \frac{1}{1-\nu} \quad (2.42)$$

- M' : The one-dimensional modulus of the loose material
 h : The thickness of the loose material

2.1.11 Viscoelastic solution by Chua and Lytton

The deformation properties of the soils and pipe materials, such as thermoplastic pipes, change with time and stress. PVC pipes are known to creep under constant stress and relax under constant strain (Masada, 1996). Chua and Lytton (1989) proposed a viscoelastic solution including the time dependence of the deflections, stresses and strains in the pipe/soil system. Their study is based on the linear elastic solution of Höeg (1968) and modified by Galili and Shmulevish (1988) to include the effect of bedding. The transformation of the elastic solution of Höeg to viscoelastic is obtained by replacing the compressibility and flexibility factors with the following terms:

$$C = \frac{1}{2} \cdot \frac{1}{1-\nu} \cdot \frac{\left(\frac{1}{2t} \right)^m M_1 \Gamma(1-m) \left(\frac{D}{T} \right)}{\left[\left(\frac{1}{2t} \right)^{m_e} E_{cl} \Gamma(1-m) \right] (1-\nu_{c_2})} \quad (2.43)$$

$$F = \frac{1}{2} \cdot \frac{1-2\nu}{1-\nu} \cdot \frac{\left(\frac{1}{2t}\right)^m M_1 \Gamma(1-m) \left(\frac{D}{T}\right)^3}{\left[\left(\frac{1}{2t}\right)^{m_c} E_{c1} \Gamma(1-m) \right] (1-\nu_{c2})} \quad (2.44)$$

t : Time elapsed in minutes

m : Exponent of power law for soil

m_c : Exponent of power law for pipe material

M_1 : Relaxation modulus of soil

T : Pipe wall thickness

Γ : Gamma function $\Gamma(n) = \int_0^{\infty} e^{-x} \cdot x^{n-1} \cdot dx$ $\Gamma(n+1) = n \cdot \Gamma(n)$

In the solution of Chua and Lytton (1989), it is assumed that there is full adhesion of the soil to the outer wall of the pipe. The solution indicates that the additional deflection caused by the soil sliding around the pipe is minimal.

2.1.12 Finite element analysis

The finite element method is a mathematical solution originally developed to solve complex structural systems. The technique is also very useful in the geotechnical field. Moreover, it is successfully used in other areas such as fluid mechanics, thermodynamics, ground water flow, aerodynamics, etc.

As the name implies, the finite element method involves the discretization of a continuum in finite elements. The elements may be one, two or three dimensions and are only connected at their nodes. Shape functions relates the displacements in the elements and along the element boundaries to the nodal displacements. External loads or the self-weight of the element may cause displacements. In addition, prescribed displacements must be specified at the boundary of the system.

Once the continuum is idealised and the boundary conditions are specified, an analysis is performed using the stiffness method. The stiffness method of analysis consists of equilibrium equations, which are solved for unknown nodal displacements. The equilibrium equations, in matrix form, consists of a stiffness matrix $[K]$ that relates the nodal displacements $\{d\}$ to the nodal forces $\{f\}$. The stiffness matrix is a function of the geometry and the properties of materials. The displacements found using the equilibrium equations can then be used to evaluate the element stresses and strains.

In the special case of soil-structure interaction mechanics, the stress-strain behaviour of the elements is non-linear. In order to accommodate the non-linear stress-strain properties, the solution procedure must follow the stress condition incrementally.

Finite element analysis has been proven to be very useful in the analysis of buried structures. Therefore, finite element programs are now available on the market. Each of them has its advantages depending on the problem. Some of these programs are PLAXIS, which is used in this thesis, PIPE5, which is a version of SAP (Wilson, 1971) modified by the Utah State University researchers for the analysis of flexible pipe, CANDE (Culvert Analysis and Design) (Allgood, 1976) (Katona, 1980), which is the most widely used in the U.S., SPIDA (Heger, Liepins, Selig, 1985), which is used for the analysis of rigid concrete pipes and some others such as ABAQUS (1998), ADINA and SIGMA/W.

2.2 Failure modes for flexible pipes

Buried pipe failures occur in several modes, which are related to the performance limit of a specific product. It is said that a performance limit has been reached when a capability of a product is exceeded. For plastic pipes, performance limits are directly related to stress, strain, deflection or buckling. There are eight failure modes applicable to plastic pipes:

2.2.1 Wall crushing

This type of failure occurs when a very stiff pipe is embedded in a very stiff material at a great depth. It is characterised by a localised yielding at the pipe's springline as shown in Figure 2.15. The ring compression stress has a major contribution to this failure mode (Moser, 1990):

$$\text{Ring compression} = \frac{P_v D}{2A} \quad (2.45)$$

P_v : Vertical soil pressure

D : Pipe's diameter

A : Pipe thickness per unit length

The bending stress at the springline can also influence this performance limit (Moser, 1990):

$$\text{Bending stress} = \frac{Mt/2}{I} \quad (2.46)$$

M : Bending moment per unit length

t : Wall thickness

I : Moment of inertia of wall cross section per unit length

Note that the wall crushing is usually a performance limit for rigid or brittle pipe product.

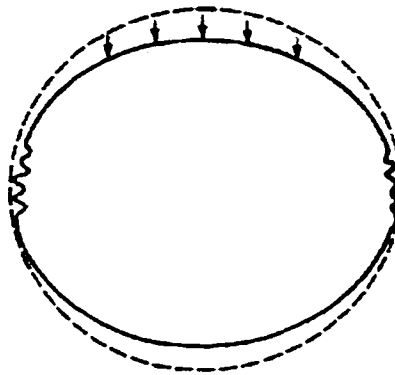


Figure 2.15: Wall crushing at the pipe's springline. The dashed line represents the undeformed pipe. (Moser, 1990)

2.2.2 Wall buckling

This failure mode occurs when the pipe has a low stiffness. The pressure around the pipe or the vacuum inside the pipe causes the pipe to buckle, as shown in Figure 2.16. Kienow and Prevost (1989) calculated the minimum pipe stiffness required to prevent buckling. For a long tube in plane strain, subjected to a hydrostatic pressure, the critical buckling pressure is (Moser, 1990)

$$P_{cr} = \frac{Et^3}{4(1-\nu^2)R^3} \quad (2.47)$$

- P_{cr} : Critical buckling pressure
- E : Young's modulus of the pipe material
- R : Pipe's radius
- ν : Poisson's ratio of the pipe material
- t : Wall thickness

Meyerhof and Baike (1963) developed the following formula for computing the critical buckling pressure:

$$P_{cr} = 2\sqrt{\frac{E'}{1-\nu^2} \left(\frac{EI}{R^3} \right)} \quad (2.48)$$

- E' : Soil modulus

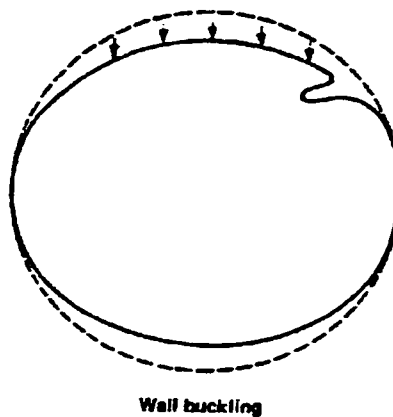


Figure 2.16: Localised wall buckling. The dashed line represents the undeformed pipe. (Moser, 1990)

2.2.3 Over-deflection

The deflection is a design parameter for flexible pipe. A safety factor is added to the maximum deflection of the pipe, and that controls the design deflection limits. For instance, PVC pipe will not undergo reversal curvature until about thirty percent deflections (Spangler, 1941). Thus, engineers generally consider a 7.5 percent deflection limit, as recommended in ASTM D 3034. Some products have deflection limits to limit bending stresses or strains. Figure 2.17 illustrates the ring deflection and the reversal of curvature due to over-deflection.

2.2.4 Normal and shear strain

Strain is related to deflection, then a highly filled plastic pipe, which is more brittle, or a pressurised plastic pipe, can have installation design controlled by strain. Moser (1990) proposed the following equations to evaluate the strains:

Bending strain
$$\varepsilon_b = 6 \left(\frac{t}{D} \right) \left(\frac{\Delta y}{D} \right) \quad (2.49)$$

Δy : Measured change in vertical diameter

Ring compression Strain
$$\varepsilon_c = \frac{P_v D}{2tE} \quad (2.50)$$

Hoop strain (caused by tensile stress in the pipe wall)
$$\varepsilon_p = \frac{PD}{2tE} \quad (2.51)$$

Poisson's circumferencial strain
$$\varepsilon = -\nu * (\text{Longitudinal strain}) \quad (2.52)$$

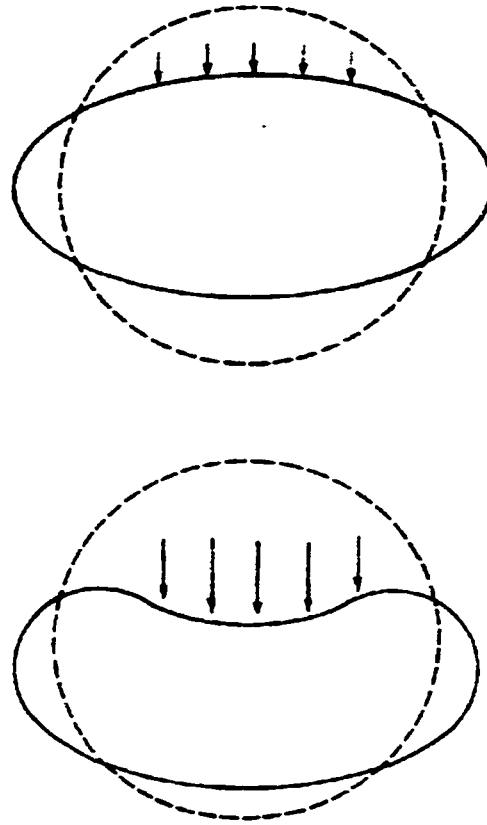


Figure 2.17: Ring deflection and reversal of curvature due to over-deflection (Moser, 1990)

2.2.5 Longitudinal stresses

Thermal expansion and longitudinal bending can produce longitudinal stresses. First, plastic pipes are usually susceptible to temperature change. The frictional resistance, caused by the ground around a buried pipe, restrains the contraction or expansion movements of the pipe and may result in a circumferential break. Figure 2.18 illustrates this phenomenon. Second, as illustrated in Figure 2.19, poor bedding, swelling clay or frost load may produce high longitudinal bending stress and result in circumferential breaks (Rajani et al., 1995).

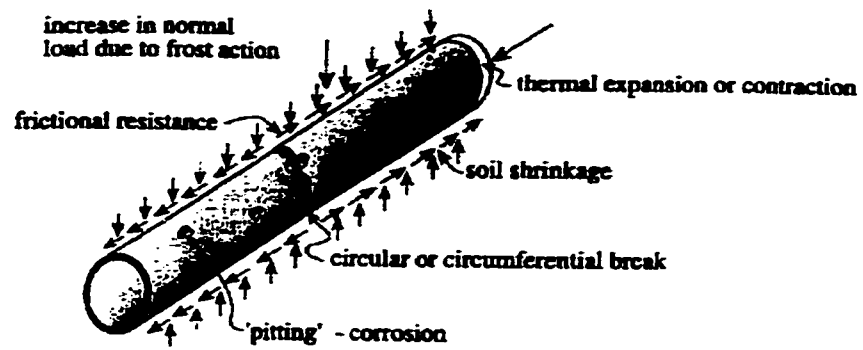


Figure 2.18: Tension failure caused by thermal expansion (Rajani, Zhan, Kuraoka, 1995).

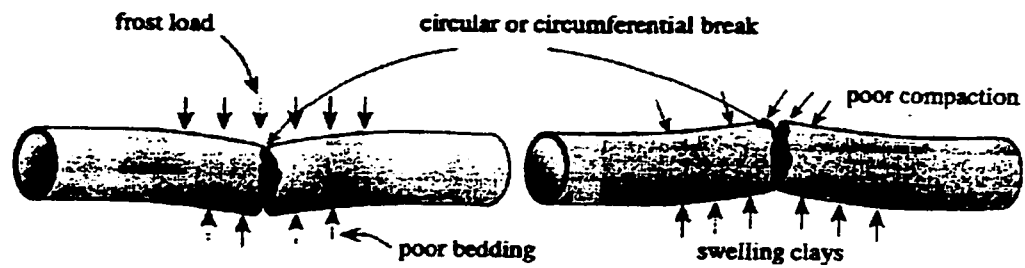


Figure 2.19: Bending or flexural failure (Rajani, Zhan, Kuraoka, 1995).

2.2.6 Shear loading

Forces, which produce shear loading, can be highly variable and difficult to quantify (Moser, 1990). As a result, these forces must be minimised by a proper installation. Some major causes of shear loading are differential settlements of structures to which the pipe is connected, erosion of the soil below the pipe and three-root growth pressure.

2.2.7 Fatigue

Pipe materials will fail at a lower stress if a buried pipe undergoes cyclic stresses (Moser, 1990). Such cyclic stresses may be produced by water hammers or by the traffic loads.

2.3 Investigation of soil-structure interface behaviour

Many problems in geotechnical engineering involve interactions between two materials. Soil-structure interaction takes place in a thin layer of soil, which is called the interface. Examples of systems that involve interfaces are sheet pile walls, piles, foundations and underground pipes. When such systems are subjected to loads, the interface may deform following different modes such as slippage and debonding.

It is important to model the soil-structure interface accurately in order to obtain a realistic solution to any kind of soil-structure interaction problem. The research conducted in recent years has shown the influence of interfaces on overall structural response. Constitutive models for the characterisation of the behaviour of interfaces include factors such as adhesion, friction, roughness, irreversible deformations, hardening and softening.

For the determination of the parameters involved in a constitutive model, it is necessary to develop appropriate laboratory apparatuses and field test devices. Devices used for interface testing are the direct shear type, the annular shear type, the ring torsion type and the simple shear type. In the present research, a three-dimensional computer controlled interface apparatus using the direct shear type was used, which is briefly described in a subsequent chapter.

2.3.1 Interface behaviour

Shear strength

The interface shear strength parameters are important in stability analyses of practical engineering problems. The Coulomb's law of friction defines the friction between two materials by the following equation:

$$F = \mu \cdot N \quad (2.53)$$

Where F is the frictional force required to produce a relative displacement at the contact surface, μ is the coefficient of friction and N is the normal load between the two materials. The initial force required to initiate motion is higher than the force required to maintain sliding. This implies that there are two coefficients of friction. The static coefficient of friction is related to the initial displacement and the kinetic coefficient of friction is related to the subsequent sliding. The static coefficient of friction is greater and it is the major concern in this report. After adding the adhesion, c_i , and rearranging the equation in terms of shear stress, normal stress and angle of friction, the Coulomb's failure criterion is obtained.

$$\tau_i = c_i + (\sigma_n)_i \tan \phi_i \quad (2.54)$$

In this equation, τ_i and σ_i are the shear stress and the normal stress at failure along the contact surface and ϕ_i is the interface angle of friction. Figure 2.20 illustrates the failure envelope defined by the Coulomb's failure criterion, where δ is the slope of the failure envelope and c_i is the intercept with the shear stress axis.

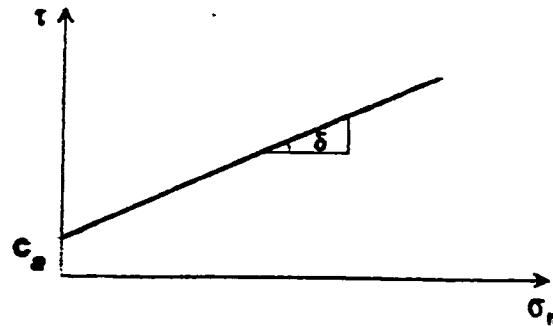


Figure 2.20: Failure envelope determined by the Coulomb's failure criterion

The parameters that influence the shear strength of an interface in a condition of constant normal stress and monotonic shearing are

- Type of surface material
- Roughness of the surface
- Composition of soil
- Relative density of the soil
- Grain size distribution of the soil
- Magnitude of normal stress
- Rate of shearing

Potyondy (1961) performed interface tests with a direct shear type device and recognised the importance of these parameters. Uesugi and Kishida (1986), using a simple shear apparatus, pointed out that the coefficient of friction between steel and air-dried sand increases with the surface roughness and the angularity of the sand grains. They also found that the effect of normal stress and mean grain size on the interface angle of friction is not significant.

Stress-displacement relations

For realistic analysis of soil-structure systems, the stress-displacement relations are necessary. Stress-displacement relations are obtained from four parameters measured with the interface test apparatus: normal stress, $(\sigma_n)_i$, shear stress, τ_i , volume change, v , and shear displacement, u .

Using the three-dimensional interface test device, C3DSSI, Fakharian (1996) reported that the roughness of the surface significantly influences the peak and residual shear strengths. He observed that the peak stress is more pronounced for rough surfaces. In addition, the volume change indicated some initial compression for smooth surface and small dilation after initial compression for rough surface.

Observations of interface test results indicate that the relation between the shear stress and the tangential displacement at the contact surface is not linear. Shear deformation in sand is

the major factor of tangential displacements when the shear stress is below the maximum value. After yielding of the interface, sliding displacements occur at the contact surface while the shear deformation in the soil mass is almost unaffected (Fu, 1998).

Other observation by Fakharian (1996) is that the rate of shearing had no effect on the stress-displacement relations and volume change behaviour of a sand/steel interface. However, Fakharian (1996) did not test interfaces for a wide range of rates.

Cyclic loading

In addition to the parameters mentioned previously, other parameters also influence the behaviour of an interface subjected to cyclic loading:

- Amplitude of displacement
- Frequency
- Number of cycles
- Boundary conditions such as soil container type in laboratory testing of interfaces

Desai et al. (1985) reported cyclic test results using a direct shear type device. They observed that for cohesionless soils the interface response stiffens or hardens with the number of cycles. Moreover, they observed that the maximum shear stress caused by cyclic loading are influenced by the displacement amplitudes and rates. Fakharian (1996) undertook two-way displacements controlled cyclic tests and observed that "...the interface may fail and soften even at tangential displacement amplitude less than that required to mobilise the maximum shear stress in monotonic tests."

2.3.2 Modelling of soil-structure interaction

The accuracy of the results of numerical modelling of soil-structure interaction depends on the constitutive law used to simulate the behaviour of the interface. In the early years of numerical modelling, the rigid-perfectly plastic Coulomb law was considered adequate. However, the comparisons between experience and modelling were not successful.

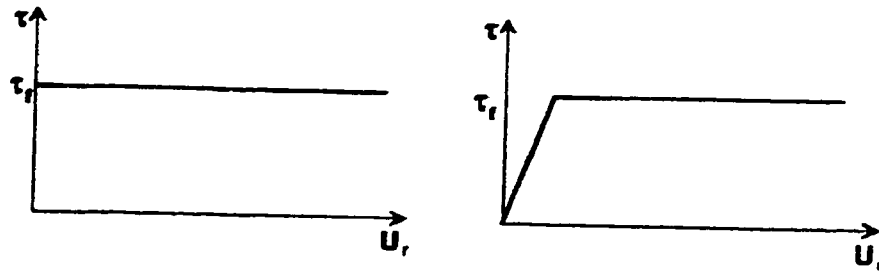


Figure 2.21: Perfectly plastic model: a) Rigid-perfectly plastic b) Elastic-perfectly plastic

Consequently, researchers found out that the interface undergoes a complete change in structure during a shearing load. This major change is due to the localised compression or dilation of granular material and also due to degradation of the friction at large displacements. The rigid-perfectly plastic and the elastic-perfectly plastic models using Coulomb's law are poor in the simulation of the normal displacements, the non-recoverable deformations before failure and the post peak behaviour of interfaces (Figure 2.21).

Non-linear elastic model simulates the behaviour of interfaces more realistically. Ramberg and Osgood proposed a curve fitting procedure for description of stress-strain curves. The model considers the unloading/reloading process, but does not include the theory of plasticity for the calculation of displacements (Desai et al., 1985). Also, the model does not account for the normal displacement in the interface.

Ghaboussi and Wilson (1973) include the theory of plasticity in the modelling of interfaces. They applied the cap plastic model to joints in rock shown in Figure 2.22. This particular model uses a perfectly plastic yield surface to limit shear stresses and a strain hardening cap to control dilatancy. The component of normal strain and shear strain can then be divided into elastic and plastic parts and represent a wide range of interface properties such as the contraction and dilatancy.

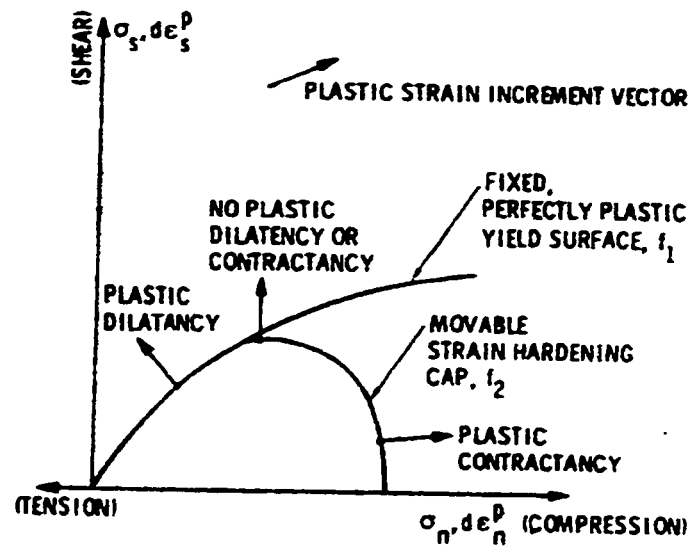


Figure 2.22: Application of capped plasticity model (Ghaboussi and Wilson, 1973)

Other elasto-plastic constitutive models were developed by Boulon and Nova, and Desai and Fishman as mentioned by Fakharian (1996)

2.4 Experimental and numerical studies by Mohri and Kawabata

Mohri and Kawabata (1995) conducted both experimental and numerical studies related to the behaviour of a low stiffness pipe buried in the ground. A large diameter pipe is used in the field test. The numerical analysis includes the simulation of construction process such as compaction.

They showed that the surrounding ground, the construction method and various properties of the backfill material have a major influence on the stresses, the deformation and the settlement of the pipe in the ground. The present study makes use of the Mohri and Kawabata work as described in Chapters 6 and 7.

2.4.1 Field test

Figure 2.23 shows a longitudinal cross section of the field test. The natural ground consists of Kanto loam and the ground water table is relatively stable at 4-m depth. The pipeline tested is made up of three pipes connected for a total length of 18-m (4.5-m, 9.0-m, 4.5-m). The distance from the crown to the ground surface, or the overburden, is 1.8-m.

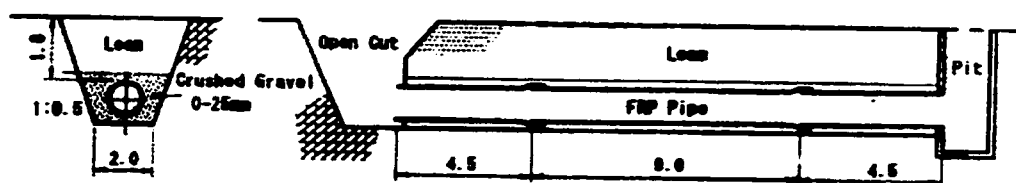


Figure 2.23: Longitudinal cross section

As shown on Figure 2.24, the bed is 0.3-m thick of compacted crushed gravel (0-25-mm) placed on natural ground. The embedment material for the pipe is also made up of crushed stone compacted by an 80 kg tamper, which was moved back and forth twice, in layers of 0.3-m thick up to 0.3-m over the crown. The backfill material above the embedment material was Kanto loam. The physical properties of the Kanto loam and the crushed gravel are presented in Table 2.6 and characteristics of the pipe are presented in Table 2.7.

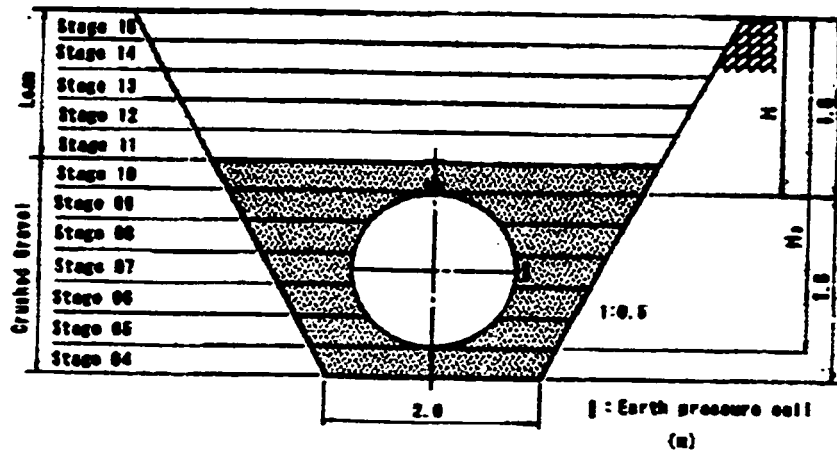


Figure 2.24: Cross section

Table 2.6: Physical properties of Kanto loam

	Kanto loam	Crushed gravel
ρ_s (g/cm ³)	2.957	2.853
ω (%)	103.1	3.4
ω_L (%)	133.2	-
ω_p (%)	51.7	-

Table 2.7: Pipe characteristics

Type of pipe	FRP
Inside diameter (D)	1500-mm
Wall thickness (t)	15.5-mm
Elastic modulus (E_p)	28.1 GN/m ²
Ring stiffness ($E_p I_p / D_2^3$)	2.56 kN/m per unit length
Pipe diameter from wall centres, D_2	1515.5-mm
Moment of inertia I_p	310.32-mm ⁴ per unit length

To measure the pipe behaviour, displacement gauges were installed across the diameter of the central pipe section. The deformations after each construction step were measured.

Twelve strain gauges were attached at intervals of thirty degrees to the inner pipe circumference. Finally, earth pressure cells were installed at the pipe crown and springline.

2.4.2 Compaction analysis

Flexible pipe uses the passive side fill pressure to sustain loads. Therefore, the compaction of the side fills is important to restrain the deformation of the pipe. In the case of a low stiffness pipe, the effect of the compaction is so large that the pattern of pipe deformation can be decided in the early stages of backfilling.

In order to undertake a static numerical analysis, it was necessary to evaluate the compacting energy in terms of loads applied to the backfill. Mohri and Kawabata determined the load applied to the compaction surface by the 80-kg tamper on the basis of the measured tamper acceleration.

Mohri and Kawabata observed that there is a hyperbolic relationship between compaction energy and compacted soil density. They conducted an experiment on a 15-m² surface of crushed gravel. The results are illustrated in Figure 2.25.

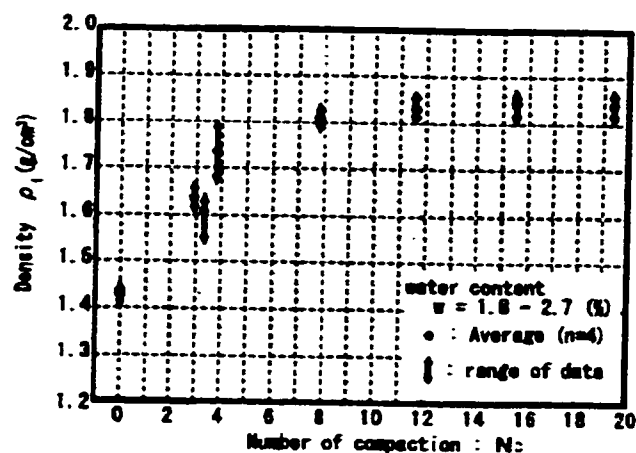


Figure 2.25: Relation between number of impacts and density

The compaction is affected by the impact that the tamper produces as it falls. Mohri and Kawabata installed an accelerometer to the bottom plate of the tamper and measured the acceleration of vibration. The results are shown in Figure 2.26 and 2.27. It is observed that the acceleration increase as the compaction of the backfill progresses. The static load applied to the surface of compaction is calculated by the following equation using the measured average acceleration. Table 2.8 presents the calculated values of the equivalent static load and stress.

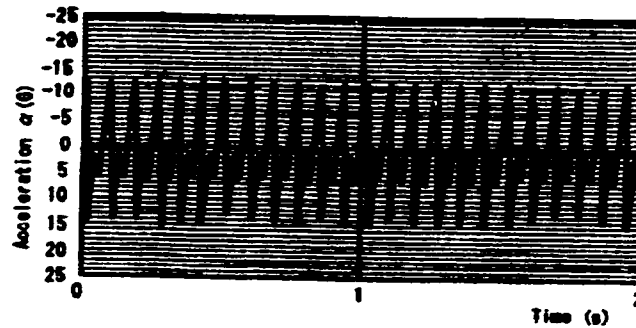


Figure 2.26: Initial compaction stage (Loose soil)

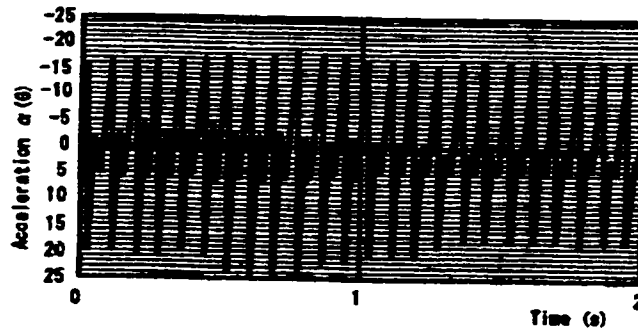


Figure 2.27: Final compaction stage (Dense soil)

$$\sigma_v = \frac{P_v}{A_T} = \frac{m \cdot \alpha}{A_T} \quad (2.55)$$

σ_v : Pressure of tamper

P_v : Load of tamper

m : Mass of tamper

α : Average acceleration

A_T : Tamper soil contact area

Table 2.8: Values for the calculation of the static load

Characteristics	Measured values
Mass of tamper m (kg)	80
Frequency (blow/s)	12
Acceleration (G)	17
Load for one blow P_v (kN)	13.6
Pressure for one blow σ_v (kPa)	174

2.4.3 Numerical analysis of Mohri and Kawabata

The numerical analysis consists of a nonlinear finite element analysis of stresses and deformations taking into account the construction stages. The analysis is based on a plane strain model with the initial stress condition calculated before excavation in the natural ground. The compaction process is considered in terms of residual strain in the backfill caused by loading and unloading by the number of effective blows. During the compaction of each layer, a prescribed displacement for restraining vertical movement was set at the bottom of the layer being compacted. Thus, the compaction load was not transmitted deeper than 0.3-m (Fukuoka et al., 1987).

Three analyses were undertaken. Analysis 1 considers the load evaluated from the measured tamper acceleration. Analysis 2 considers only the construction process. Finally, Analysis 3 is a general finite element analysis at the completion of the backfill.

Mohri and Kawabata used a finite element program that allows using rectangular quadratic-order (8-nodes) elements. Joint elements (6 nodes) were used for the interface between the pipe and the backfill. The Variable Moduli model of Nelson (Nelson and Baron, 1971) was used to represent the nonlinear soil characteristics of the backfill and the natural ground. The input parameters for the model are presented in Table 2.8.

Table 2.9: Input parameters of the Variable Moduli model.

Parameters	Kanto Loam	Crushed Gr.	Parameters	Kanto Loam	Crushed Gr.
G_o (MN/m ²)	2.028	1.262	K_o (MN/m ²)	3.436	2.2
G_{ou} (MN/m ²)	5.631	1.696	K_{ou} (MN/m ²)	1.533	5.449
G_{or} (MN/m ²)	6.785	4.088	K_{or} (MN/m ²)	3.628	6.977
$\bar{\gamma}_1$	-126.2	-25.73	K_1 (MN/m ²)	-33.669	-31.967
$\bar{\gamma}_{1u}$	122.3	87.34	K_{1u}	132.47	288.58
$\bar{\gamma}_{1r}$	-422.27	-50.69	K_{1r}	19.78	74.56
γ_1	49.79	16.66	K_2 (MN/m ²)	26005	19.396
γ_{1u}	27.15	5.85			
γ_{1r}	166.61	43.70			

2.4.5 Results

The deflections of the pipe obtained from the field test and the finite element analysis are illustrated in Figure 2.28. From the field test results, it is observed that the process of compaction and backfilling to the pipe crown significantly influences the behaviour of a buried low stiffness pipe, but after that it remains almost unaffected by the backfill from the crown to the ground surface. In addition, the deformation shows stability on a long-term basis.

The FE analysis taking into account the compaction process simulated fairly well the behaviour of the pipe. However, Analysis 1 gave a vertical deflection 70% higher than the measured value in the process of backfilling up to the pipe crown. Analysis 2 did not produce the actual pipe deformation. In both analyses, the flattening mode by backfilling from the crown to the ground surface was greater than the measured value. In the case of the third analysis, it was impossible to predict the behaviour of the pipe.

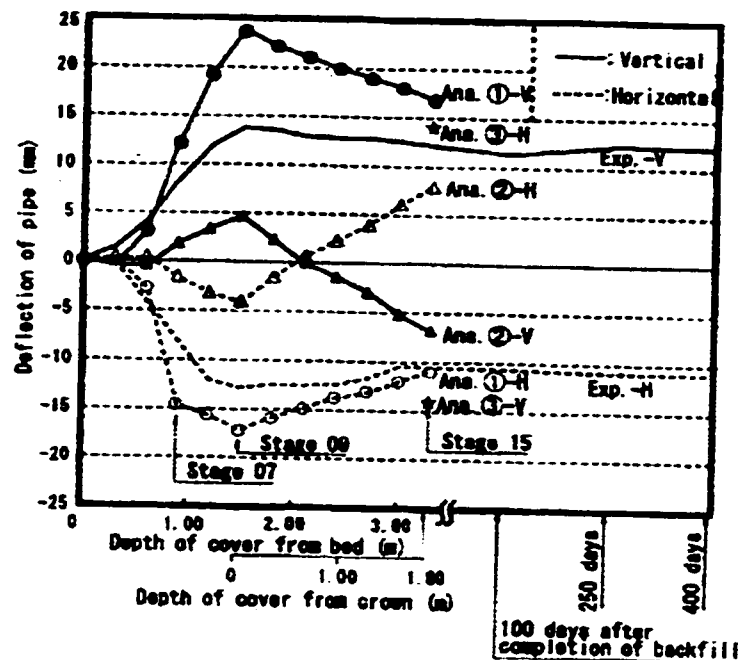


Figure 2.28: Change of deflection of pipe

Figure 2.29 shows the variation of vertical and horizontal pressure with time in the field test. It is observed that the vertical pressure on the crown is greater than the horizontal earth pressure at the springline. Moreover, the vertical pressure on the crown is 1.25 times that of overburden pressure or prism load (γH). These observations are unusual for flexible pipes. Mohri and Kawabata explained high value of the vertical pressure by the increase of the apparent relative stiffness of the pipe, because the pipe deflection due to the backfill above the pipe crown did not increase so much.

The earth pressures from the FE analysis are shown in Figure 2.30 at two stages: Backfilling up to the pipe crown (Stage 9) and backfilling up to the ground surface (Stage 15). It is observed that the stresses are high and discontinuous due to the residual strain in the backfill. As measured in the field test, the vertical pressure upon the crown is higher than the prism load. The Analysis 2 presents a smoother earth pressure distribution and the vertical pressure is again higher than the prism load.

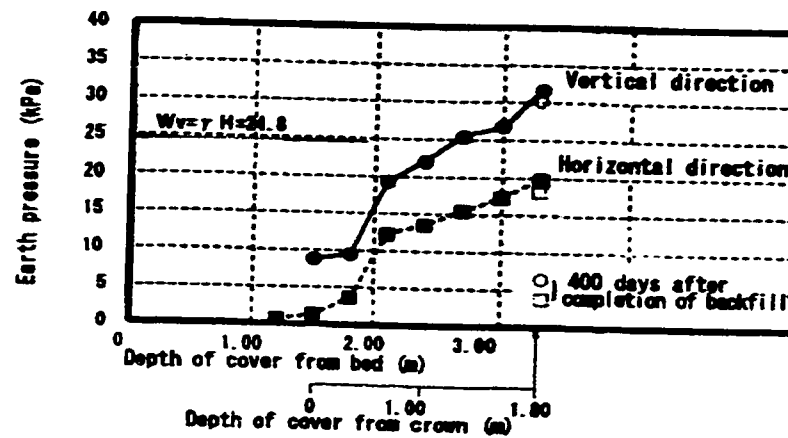


Figure 2.29: Change of vertical and horizontal earth pressure on pipe (Field test)

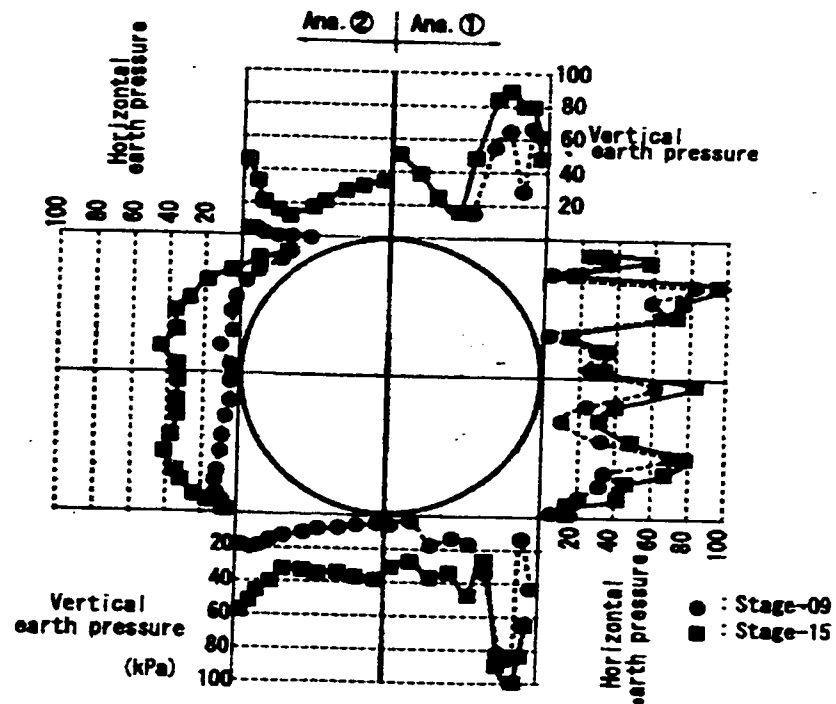


Figure 2.30: Distribution of earth pressure around the pipe

2.5 Discussion

The Marston's load theory gives realistic results and it is still used in the current practice. The theory accounts for the shearing forces between the native soil and the backfill in a trench installation and between the soil directly above the pipe and the side fill in an embankment installation. It is assumed that the shear planes are vertical and that the ratio of active lateral pressure to vertical pressure is expressed by the Rankine's ratio, K_a . These assumptions are subjected to discussion. For instance, there is no proof that the horizontal stress acting on the vertical planes are the active Rankine pressure, and that the shear planes are always vertical.

Spangler derived its Iowa formula in order to predict the deflection of flexible pipes. He assumed that a flexible pipe deflects elliptically and that the Marston's load theory applies. Therefore, he made additional assumptions. If the ratio of soil stiffness to pipe stiffness is high, then the deformation of the pipe may be rectangular instead of elliptical. Moreover, Watkins determined that the constant of proportionality of Spangler could not be a true property of the soil. As a result, Watkins defined the modulus of soil reaction, E' . The modulus of soil reaction of Watkins should not be a constant. It varies with the depth of soil over the pipe, the size of the pipe, the stiffness of the pipe relative to the soil, the trench width, the soil type, the compaction density, etc.

Höeg's method assumes that the soil behaves like an elastic material, which is isotropic and homogeneous. This is an unrealistic assumption because the soil behaviour is always non-linear. In addition, the Höeg's method does not take into account the installation practice.

The finite element method seems to be the most realistic method in predicting the deformations of buried pipes because it avoids several assumptions that have been made in the development of analytical methods. It also allows the consideration of construction stages in complex trench configurations and to simulate complex loading patterns. However, the accuracy of the results depends on the soil models available.

CHAPTER 3

PLAXIS: FINITE ELEMENT CODE FOR THE ANALYSIS OF SOIL MECHANICS AND ROCK MECHANICS PROBLEMS

PLAXIS is a finite element code specifically developed for the analysis of geotechnical problems such as the settlement of foundations, the stability of slopes, or the deformation of buried structures. PLAXIS stands for PLANE strain and AXISymmetric analysis, two idealizations commonly used in geotechnical engineering. The plane strain idealization is used for structures having a constant cross section over a significant length. This characteristic simplifies the problem because it is assumed that the displacements perpendicular to the cross-section are zero. The axisymmetric model is used for problems that are symmetrical relative to a central axis. This idealization simplifies the problem by assuming the deformation and the stress state identical in any radial direction. Geotechnical problems involve non-linear and time-dependent behaviour of soils that can only be handled by advanced constitutive models. PLAXIS integrates some well known constitutive models that can deal with such complex analysis, as well as pore pressure in multi-phase materials. In addition, PLAXIS can model the interaction between a structure and soil. A brief summary of all the features is given below. A special attention is devoted to the interface feature.

3.1 Review of the features

3.1.1 Graphical input of geometry

The version 7 of PLAXIS includes a computer assisted drawing program. This drawing program is convenient and allows the possibility to model a wide variety of geometry. Applied loads, prescribed displacements and structural members can be directly added to the geometry in the drawing area.

3.1.2 Automatic mesh generation

Precious time is saved by the automatic mesh generation of finite elements. This feature allows the automatic generation of a random mesh of triangular elements. There is an option available to refine the mesh where the displacement and stress are concentrated.

3.1.3 High order continuum elements

Two types of triangular elements are available in PLAXIS. The standard quadratique 6-node element gives good results. However, the cubic 15-node element is even more accurate and gives a smooth distribution of stresses in the soil. It might be advantageous to use the 6-node element in problems with very large number of elements, since this element type is less computer-time consuming.

3.1.4 Soil models

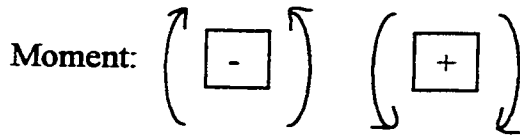
As it is mentioned before, PLAXIS includes some advanced constitutive models, as well as some simple models. The simplest model is the well-known Mohr-Coulomb model. This model gives a good approximation of the ultimate load for simple problems. For more complex problems, involving time-dependent behaviour or unloading-reloading residual strain, more advanced models are necessary. The Soft-Soil model, which is based on the Cam-Clay model (Schofield and Wroth, 1968), is efficient to analyse the behaviour of normally consolidated soft soils. Secondary compression can also be modelled. For stiffer soils, the Hardening Soil model of Schanz (PLAXIS, 1998), which is based on the Hyperbolic model (Duncan and Chang, 1970), gives good results. A detailed presentation of the Hardening Soil Model is presented in Chapter 5.

3.1.5 Beam elements

A beam in PLAXIS is "a structural object used to model slender structures in the ground with a significant flexural rigidity and normal stiffness." The beam element can include three or five nodes depending on the type of elements previously chosen. Each node has three degrees of freedom. There are two degrees of freedom related to the displacement, and one related to the rotation. It is also possible to simulate the development of a plastic hinge when the maximum bending moment or maximum axial force is reached. The weight of the beam can be taken into account. However, care must be exercised in order to obtain accurate values. In the formulation of PLAXIS, beams are superimposed as a continuum and therefore "overlap" the soil. It is then necessary to subtract the unit weight of the soil from the unit weight of the beam.

The sign convention in PLAXIS is shown on the diagrams below:

Shear:



3.1.6 Automatic load stepping

This feature allows to optimise the step size to get an efficient calculation process. Load increments that are too small would require many steps, and the computer-time could be excessive. Nevertheless, too large increments would require an excessive number of iteration to reach equilibrium, and the solution could even diverge. The automatic load stepping procedure from the user manual of PLAXIS is presented below.

1. If the solution reaches equilibrium within a number of iterations that is less than the desired minimum control parameter, then the calculation step is assumed to be too small. In this case, the size of the load increment is multiplied by two and further iterations are applied to reach equilibrium.
2. If the solution fails to converge within a desired maximum number of iterations, then the calculation step is assumed to be too large. In this case, the size of the increment is reduced by a factor of two and the iteration procedure is continued.
3. If the number of required iterations lies between the desired minimum and the desired maximum, then the size of the load increment is assumed to be satisfactory. After the iterations are complete, the next calculation step begins. The initial size of this calculation step is made equal to the size of the previous successful step.

3.1.7 Staged construction

This procedure is important to get the most realistic results. The staged construction feature simulates the construction, excavation, and backfill processes by changing the properties of soil clusters and structural elements during the calculation. For example, soil clusters, which are part of the geometry, are switched off to simulate the excavation. Structural elements can also be switched off.

3.1.8 Updated Lagrangian analysis

PLAXIS is equipped with a special option for large displacement problems. The updated Lagrangian analysis continuously updates the mesh during the calculation. This option was not used in the finite element analysis of the pipes in this research work.

3.1.9 Presentation of the results

The results in PLAXIS are presented graphically following different format. First, a view of the deformed mesh is presented. Second, it is possible to visualise the total, incremental, horizontal and vertical deformations by vectors, contour lines, or shaded areas. Third, the effective and total stresses are shown in the form of principal stresses, mean contour lines, or mean shaded areas. It is possible to visualise deformations, bending moments, shear stresses and normal stresses for structural and interface element. Underground water flow and pore pressure outputs are also available.

In addition, PLAXIS includes a special curve program to visualise load or time versus displacement, and stress-strain diagrams. This information is particularly useful to analyse local behaviour of soils.

3.2 Interface

The formulation of interface element in PLAXIS uses an elasto-plastic model. The model is governed by the Coulomb failure criterion, which differentiate between plastic and elastic behaviour. Elastic displacement occur when the shear stress is lower than $(\sigma_n)_i \tan \phi_i + c_i$, where σ_n is the normal stress acting on the interface, ϕ_i is the friction angle of the interface

and c_i is the adhesion of the interface. Plastic displacement occurs when the shear stress is equal to $\sigma_n \tan \varphi_i + c_i$.

The parameters φ_i and c_i are determined from the properties of the soil. Usually, the strength of the interface is less than the strength of the soil. Therefore, a reduction factor is applied to the friction angle and the cohesion of the soil resulting in the properties of the interface. However, the reduction factor does not apply to the dilation angle ψ_i of the interface. The dilation angle of the interface is set to zero if the reduction factor is smaller than 1, otherwise, it is equal to the dilation angle of the soil.

The magnitude of the elastic relative displacement perpendicular and parallel to the interface is determined by the following equations:

$$\text{Elastic gap} = \frac{\sigma_i}{E_{eod,i}} \quad \text{Elastic slip} = \frac{\tau_i}{G_i}$$

Where $E_{eod,i}$ is the one-dimensional compression modulus of the interface, G_i is the shear modulus of the interface, and t_i is the virtual thickness of the interface. In the user manual of PLAXIS, the virtual thickness is described as: "...an imaginary dimension used to obtain the material properties of the interface...defined as the virtual thickness factor times the average element size." The average element size depends on the degree of refinement applied with the automatic mesh generation. The default value of the virtual thickness factor is 1. The one-dimensional compression modulus is determined by the following equations:

$$E_{eod,i} = 2G_i \frac{1-\nu_i}{1-2\nu_i} \quad G_i = R_{inter}^2 G_{soil} \quad \nu_i = 0.45$$

R_{inter} : Reduction factor

ν_i : Poisson's ratio

CHAPTER 4

EXPERIMENTAL OBSERVATIONS ON STONE DUST / POLYVINYLE CHLORIDE (PVC) INTERFACE

Direct shear type interface tests were performed on soil-Polyvinyle Chloride (PVC) pipe surfaces to study pipe-to-soil friction. The C3DI (Cyclic three-dimensional testing of interfaces) apparatus (Fakharian and Evgin, 1996) was the test device. Soil was stone dust, which is a type of soil widely encountered in the current practice of buried plastic pipe installation. The major goal of these tests was to study the shear strength of an interface between PVC pipe and stone dust and formulate reasonable pipe-to-soil friction parameters. These parameters were then used in a finite element analysis.

4.1 Description of the apparatus

The C3DI computer controlled apparatus was developed by Fakharian and Evgin (1996) to study the behaviour of interfaces between sand and steel. A schematic view of the apparatus is illustrated in Figure 4.1. The capabilities of the apparatus are the following:

- Direct and simple shear types of test
- Displacement or load controlled in three directions
- Monotonic or cyclic loading at various loading rates
- Circular or elliptical stress path on the interface plane
- Constant normal stress or constant normal stiffness boundary conditions
- Separating the sliding displacement at the interface from the shear deformation in the soil mass

The apparatus consists of a reaction frame that can withstand a vertical or horizontal load up to 25 kN. The actuators used for applying the normal and tangential loads have each a capacity of 10 kN. A pneumatic actuator is mounted on the top of the frame to apply the normal load. Two stepper motors are fixed to the bottom of the frame to apply the tangential load through an X-Y loading table. Load cells with an accuracy of 5-N monitor the loads and

linear variable displacement transducers (LVDTs) with an accuracy of 0.002-mm monitor the displacements in the X, Y and Z directions. The direct shear box is a hollow aluminium box, 25-mm thick, with an inside area of 100-mm x 100-mm.

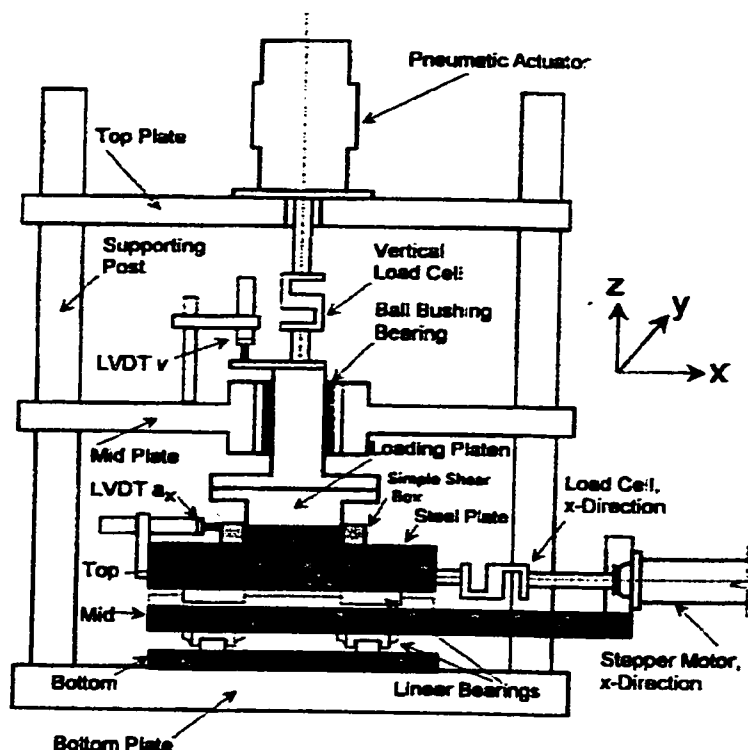


Figure 4.1: Schematic view of the C3DI apparatus

4.2 Materials characteristics

4.2.1 Soils

Two types of stone dust were used as soil samples throughout this study. Both types of soil are classified as “ Well-Graded Gravel with Silt” using the Unified Soil Classification System, ASTM D2487-93, “Classification of Soil for Engineering Purposes”. These crushed soils are composed of mechanically crushed limestone. In order to differentiate the two types of soil, they will be designated as stone dust and crushed stone throughout the thesis.

The physical characteristics of the stone dust and the crushed stone are presented in Table 4.1. The minimum and maximum unit weights were determined following the procedure of ASTM D 4254. The grain size distributions obtained from a sieve analysis are illustrated in

Figures 4.2 and 4.3. The stress-strain curves of the soils as determined by triaxial tests are illustrated in Figures 4.4 through 4.9. The results of the triaxial tests are also used to define the failure envelopes presented in Figures 4.10 and 4.11. The stress-strain curves and the strength parameters correspond to a relative density between 85% and 98%.

Table 4.1: Properties of stone dust and crushed stone

Soil type	Mean grain size D ₅₀ (mm)	Maximum unit weight $\gamma_{\min}$ (kN/m ³)	Minimum unit weight $\gamma_{\max}$ (kN/m ³)	Water content ω %	Friction angle (°)
Stone dust	1.4	20.2	16.1	0.4	45.1
Crushed Stone	2.3	20.4	15.3	0.4	41.1

4.2.2 Structural material

The present analysis deal with the interaction between Polyvinyle Chloride (PVC) pipes and backfill material, that is the crushed stone. It was not possible to obtain PVC plates made of exactly the same material of PVC pipe. However, a different grade of PVC plates were commercially available. Plates used in the tests had a surface area of 900 cm² and a thickness of 3 mm. The properties of those plates are presented in Table 4.2.

Table 4.2: Properties of Polyvinyle chloride (PVC) under normal impact

Property	Test method: ASTM	Type 1: Normal impact
Tensile strength (kPa)	D638	53 917
Modulus of elasticity (kPa)	D638	2 826 850
Elongation (ultimate) (%)	D638	60
Hardness-Rockwell (R scale)	D785	1.5
Flexural strength (kPa)	D790	88 252
Compressive strength (kPa)	D695	76 532
Shear strength (kPa)	D732	55 089
Vicat softening temperature (°C)	D1525	83
Coefficient of linear expansion (cm/cm/°C)	D696	5.8×10^{-5}

4.3 Procedure

4.3.1 Sample preparation

The soil was carefully placed in the direct shear box, which was fixed at the centre of a PVC plate, avoiding segregation. Then, the sample was vibrated about ten seconds with a mass of 28 kg on top of the soil sample. The tests were performed with dense soil samples in order to simulate a compacted embedment around a buried PVC pipe. A suction device is usually used to level the top of soil samples before it was placed in the interface apparatus. However, this method was not successful with a type of soil with large grain sizes such as 5 mm. As an alternative, a thin coarse sand layer was added on top of the sample. This layer was then leveled off at the final height by the suction device. This avoided concentration of stresses at the contact surface. The relative density of the crushed stone and the stone dust was about ninety percent. The unit weight of the coarse sand was 12.92 kN/m^3 in its loosest state.

4.3.2 Test procedure

The PVC plate with the direct shear box was fixed to the loading table in the interface apparatus. Then, the loading piston was moved downward to touch the sand surface. At this point, the LVDTs were installed in order to measure the initial compression of the soil sample when the load is applied. The direct shear box was unlocked after configuring the LVDTs, the load cells and the sequence of the test. Finally, the normal load was applied and the initial normal displacement was measured. Before starting the test, all LVDTs were set to zero. The output parameters are automatically recorded and displayed by the computer.

Monotonic and cyclic two-way tests were performed in order to evaluate the effect of the normal stresses and the number of cycles on the roughness of a PVC surface. The constant normal stresses ranged from 100 kPa to 500 kPa. In addition to the coefficient of friction and the friction angle of interfaces, the relation of the shear strength to the number of cycles, the normal stress and the roughness of the PVC surface were considered. All tests were displacement controlled at 5-mm. The maximum displacement in each direction was 5 mm to ensure that the tests were conducted well beyond the peak strength.

The roughness of the PVC surfaces were determined at five points, in the X and Y directions, as shown in Figure 4.12. An average of measurements at these five points is the value reported in this study. The device used to measure the roughness is the T500 tester made by the German Company Hommel. The T500 surface measuring instrument is a device of accuracy class 1 (Hommel user guide, 1998). It permits mobile recording of roughness values and transfers these to a computer for further processing. Figure 4.13 presents an example of the output of the software. The roughness parameter (R_a) from the tester is defined as “the arithmetic mean of the profile deviation of the filtered roughness profile from the centre line within the measuring length, l_m ”, and it is illustrated in Figure 4.14. Table 4.3 presents the configuration of the device.

The upper portion of the Figure 4.13 displays the surface profile. It is possible to display three different types of profiles: the total profile, the waviness profile and the roughness profile (Figure 4.15). These profiles are differentiated on the basis of wavelength. Undulations with a relatively long wavelength are represented by the waviness profile, and undulations with a much shorter wavelength are represented by the roughness profile. The total profile combines the effect of both the waviness and the roughness profile.

Table 4.3: Configuration of the Hommel T500 device

Parameter	Value
Stroke length (L_t)	15-mm
Cut off length (L_c)	2.5-mm
Measurement range (Filt)	MB1 (+20- μm to -60- μm)
Type of profile (Prof)	Total profile
Unit of measurement (Unit)	Metric
Vertical magnification (V_v)	13
Measurement length (l_m)	15-mm

4.4 Test results

4.4.1 Surface Roughness versus Constant Normal Stress

Figure 4.16 shows that the PVC surfaces were damaged an increasing amount during shearing as the normal stress increased. The roughness of a PVC plate along the direction of the shear test and sheared with a normal stress of 300 kPa is 0.65, while the roughness with a normal stress of 100 kPa is 0.33. It is also observed that the roughness of a PVC plate is significantly higher in the direction perpendicular to the direction of shearing than the roughness along the direction of shearing.

4.4.2 Surface Roughness versus Number of Cycle

PVC plates were further damaged as the number of cycles increased. Figure 4.17 illustrates the results of eight cyclic tests where the roughness was measured after each test. Each test consisted of one cycle, which is defined as an interface sheared back and forth. The roughness seems to reach a constant value after six cycles. However, more tests should be done to clearly establish this threshold.

4.4.3 Shear Strength versus Surface Roughness

Figure 4.18 shows that the maximum shear stress stayed constant although the surface roughness increased. It is most likely that the roughness values are so small that small changes in the roughness values do not affect the shear strength.

4.4.4 Shear Strength versus Normal Stress

Four interface tests at constant normal stresses of 100, 200, 300 and 500 were performed. As shown in Figure 4.19, the shear strength increased linearly with increasing normal stress. The slope of the curve was about 29° passing through the origin. Thus, there is no apparent adhesion at the interface between soil and PVC.

4.4.5 Shear Strength versus Number of Cycle

Figure 4.20 presents the results of a shear test involving 18 cycles with a constant normal stress of 200 kPa. The shear strength of the interface decreased as the number of loading cycles increased. As it was shown in Figure 4.18, variations in surface roughness for

relatively smooth PVC surfaces do not significantly influence the shear strength. Thus, the reduction of the shear strength during cyclic loading is due to changes in the soil structure. A simple shear type interface apparatus would be a better device than the direct shear apparatus to evaluate the deformation in the soil mass.

4.4.6 Shear Strength versus Relative Density

The graph of shear strength as a function of relative densities is given in Figure 4.21. In general, the shear strength seems to increase as the relative density increases. The scatter in the values may be related to the method of levelling the samples with a layer of sand. It was difficult to have a constant thickness of sand and that may have influenced the determination of relative densities. In addition, there was some variations in the density of soil samples densified using a shaking table.

4.4.7 Friction Angle of the Interface

The friction angle of the interface between a PVC plate and the crushed stone varied from 27.9° to 32.1° . These values are about 70-75% of the internal friction angle of the crushed stone. In the case of the stone dust, the friction angle of the interface varied from 28.6° to 29.4° . These values are about 60-65% of the internal friction angle of the stone dust. The coefficient of friction varied from 0.53 to 0.63 for the crushed stone and from 0.55 to 0.57 for the stone dust. The normal stress did not influence the friction parameters. The same behaviour was observed by Uesugi and Kishida (1986) and by Fakharian (1996).

4.5 Field observation

The city of Gatineau has constructed a water distribution system for a future development. The level of the future street has been modified a few years after the installation. Consequently, the 450-mm PVC pipes have been withdrawn from the soil.

This operation allowed me to look at the roughness and the degradation of the pipe caused by the compaction of the backfill. Crushed stone with a maximum grain size of 20 mm was used as backfill material. Usually, crushed stone is used when there is an appreciable slope in the pipe. Unlike the sand, the crushed stone will not be washed away by the water flow

under the pipe. In order to obtain a good compaction around the pipe without damaging it, construction crew compacted the bedding and the embedment below the springline with 300-mm soil layers. The backfill was placed up to 300 mm above the crown in one step, and then the compaction was completed.

The surface of the pipes seemed to be smooth, although there were some significant damage caused by the machinery, which was used to withdraw the pipes. The roughness of the pipes in the longitudinal direction varied from 0.72 to 3.4. The roughness in the circumferential direction varied from 1.26 to 1.9. . It was difficult to obtain very reliable measurements because of the shape of the pipe. However, it could be observed that the pipe was scratched in the longitudinal direction and in the circumferential direction.



Figure 4.2: PVC 450-mm pipes withdrawn from the ground

4.6 Discussion

Pipe-to-soil interface shear strength is an important characteristic in the design of water distribution systems. Especially for northern countries, where plastic pipes are subjected to considerable movements in the ground due to temperature changes. Interface shear strength is also a major concern in the design of thrust restraint systems. Kennedy et al. (1990) stated

that the value of the interface shear strength depends on the soil type, the moisture content, the soil compaction and the surface roughness. In the light of the present study, the interface shear strength depends on the compaction of the soil, but does not seem to be affected significantly by the surface roughness. The shearing did not increase the roughness values large enough to make a difference in the shear strength. In addition, field observations at one site showed that a PVC pipe surface was not significantly damaged when the pipe was backfilled with sand or a granular material with maximum grain size of 20-mm.

The proportionality constant relating the adhesion of the PVC/crushed stone interface and the cohesion of the crushed stone is found to be zero. In the case of the friction angle, the proportionality constant was found to be 67%. These values are used as reference for input parameters in an analysis of buried plastic pipes using the PLAXIS finite element code.

Kennedy et al. (1990) demonstrated that the moisture content has a major effect on the shear strength of an interface. Therefore, further studies should be undertaken to investigate the behaviour of interfaces between crushed stone and PVC plates under various moisture contents using simple shear type interface apparatus.

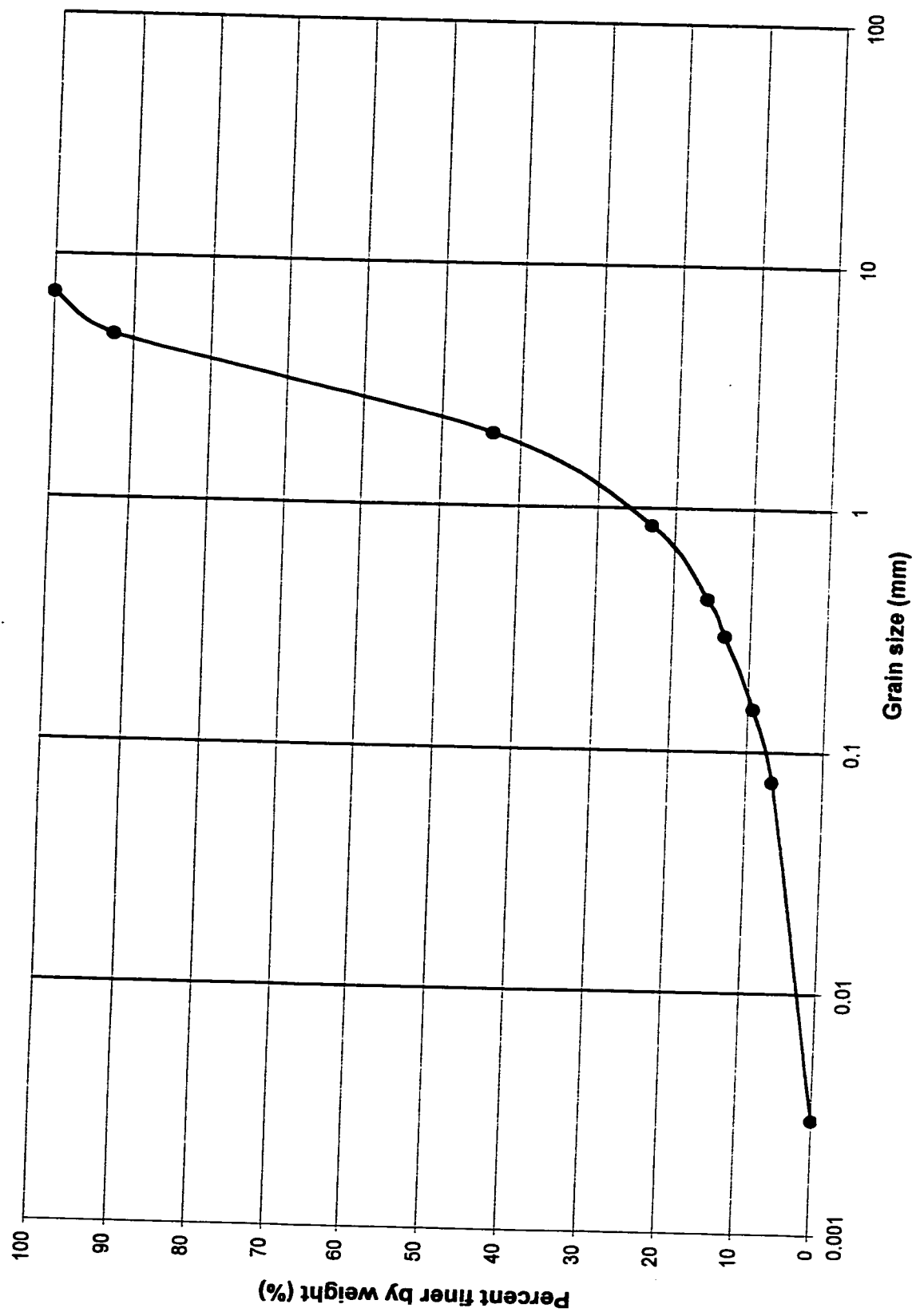


Figure 4.3: Grain size distribution of the crushed stone

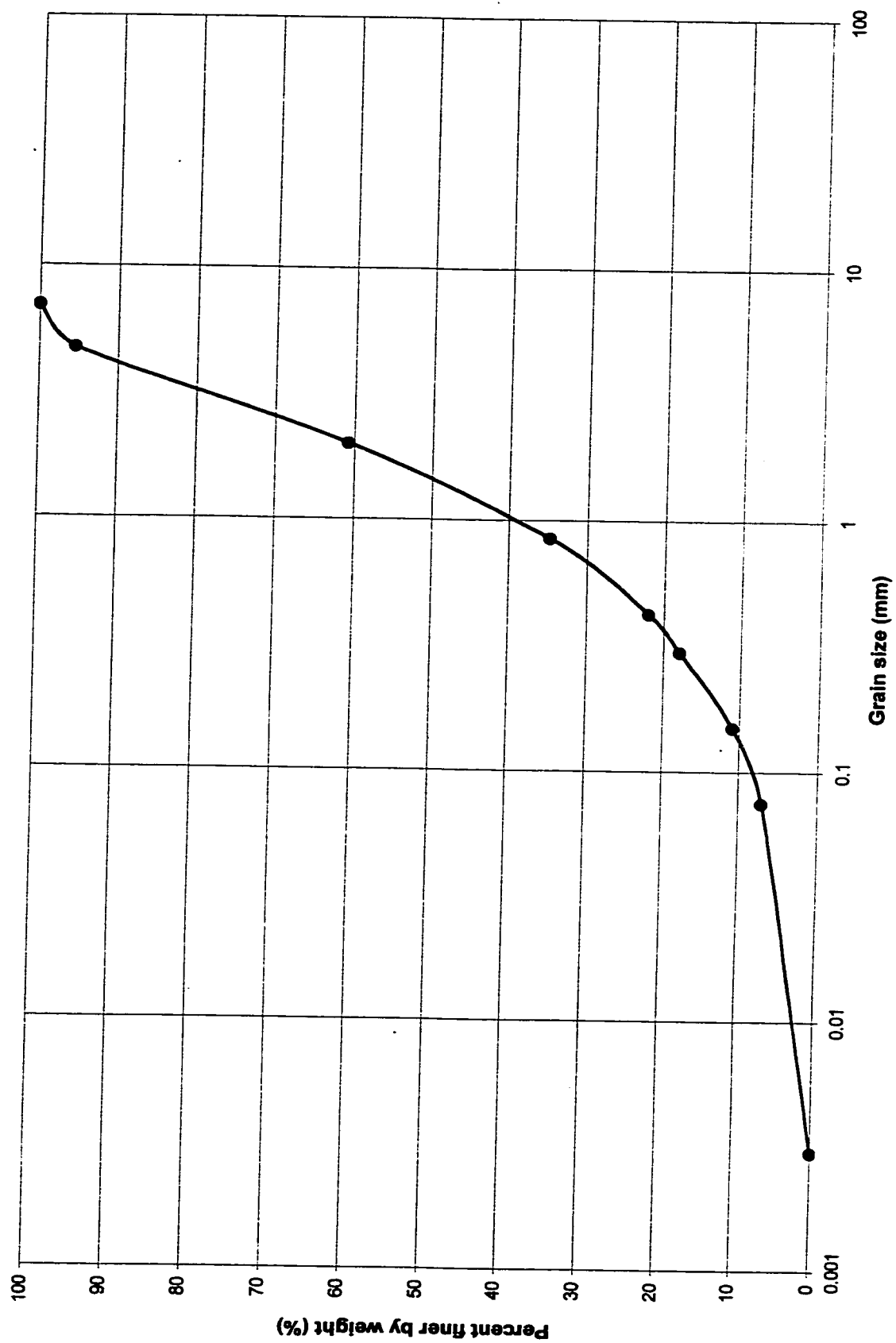


Figure 4.4: Grain size distribution of the stone dust

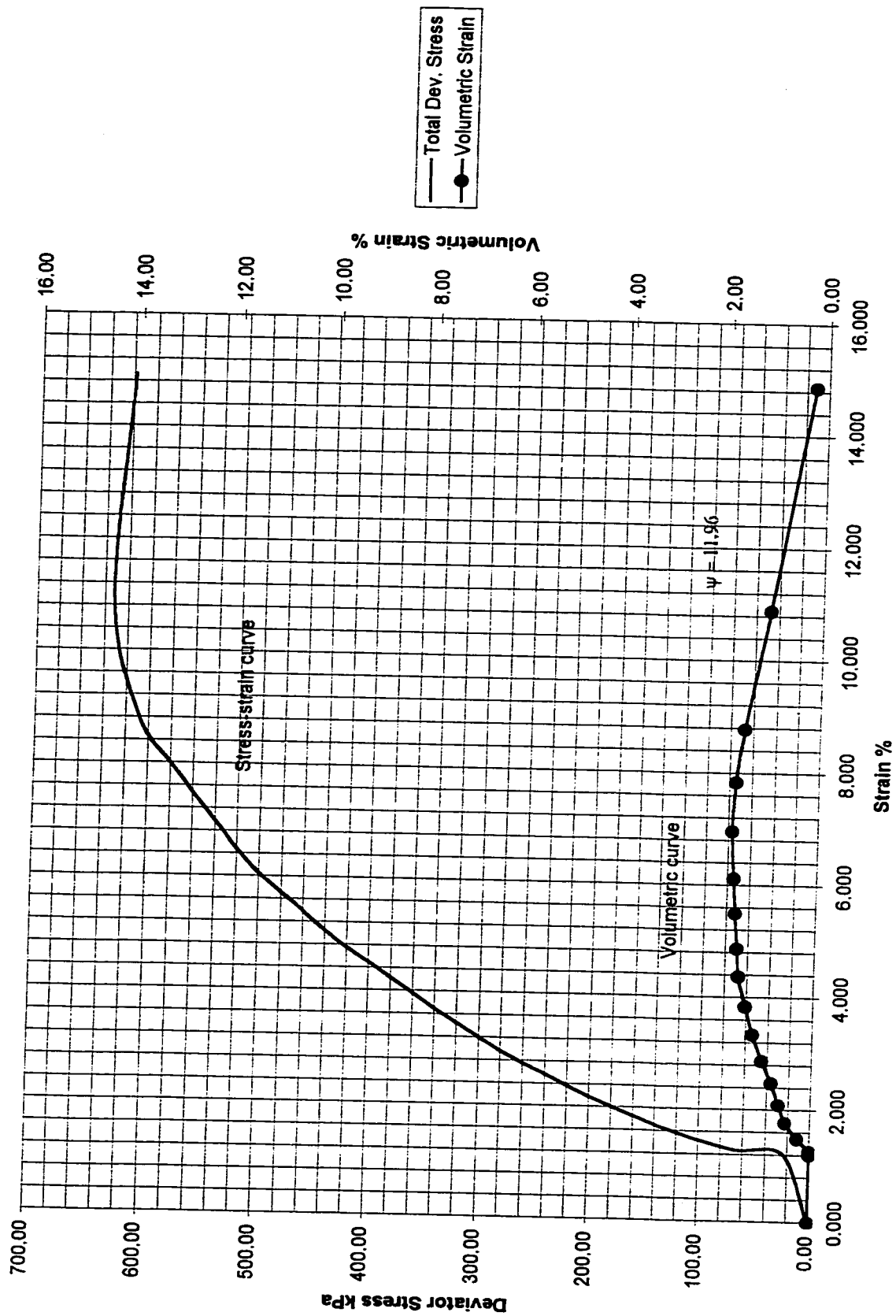


Figure 4.5: Drained triaxial test on crushed stone at a confining pressure of 100 kPa and a relative density of 89%

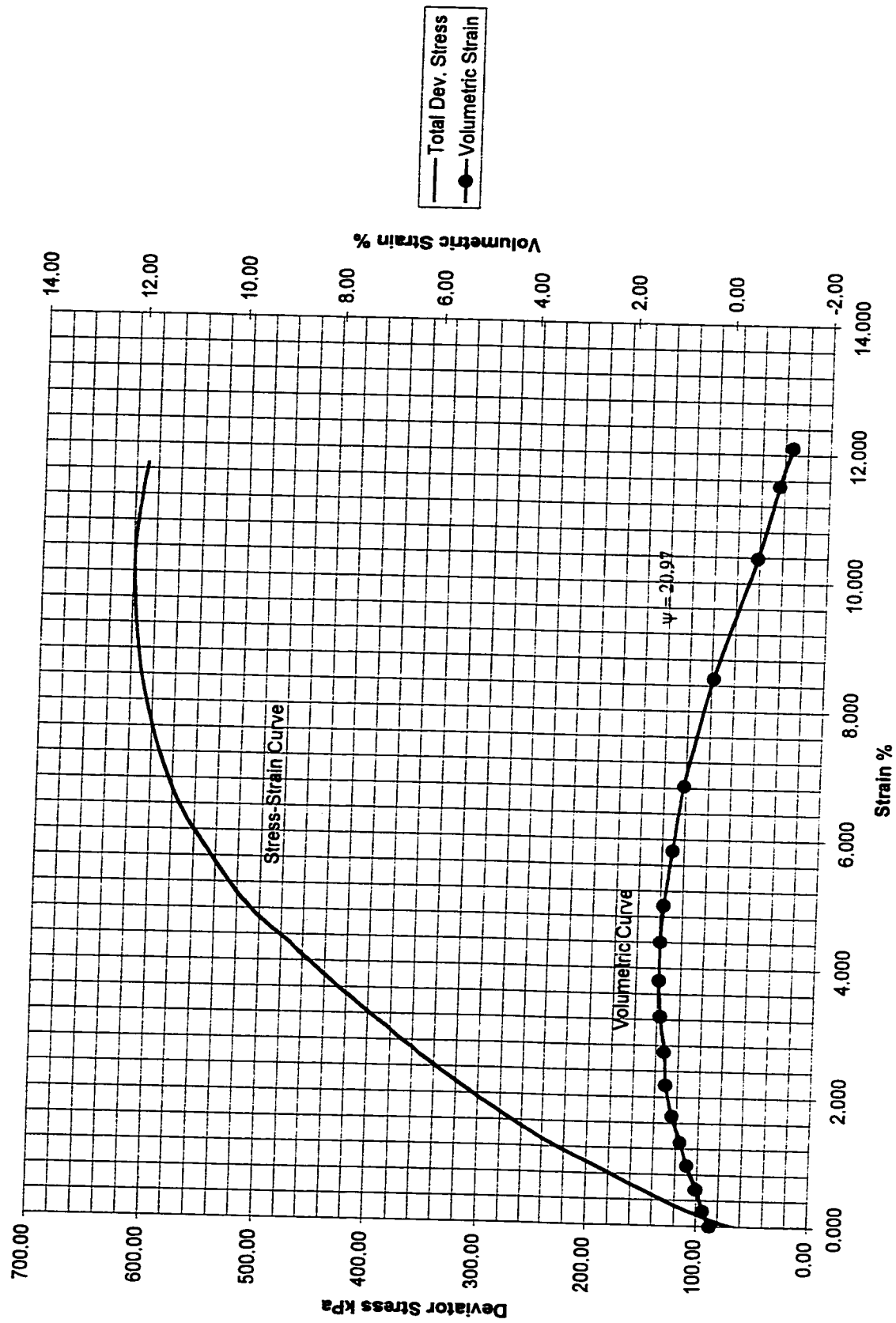


Figure 4.6: Drained triaxial test on stone dust at a confining pressure of 100 kPa and a relative density of 89%

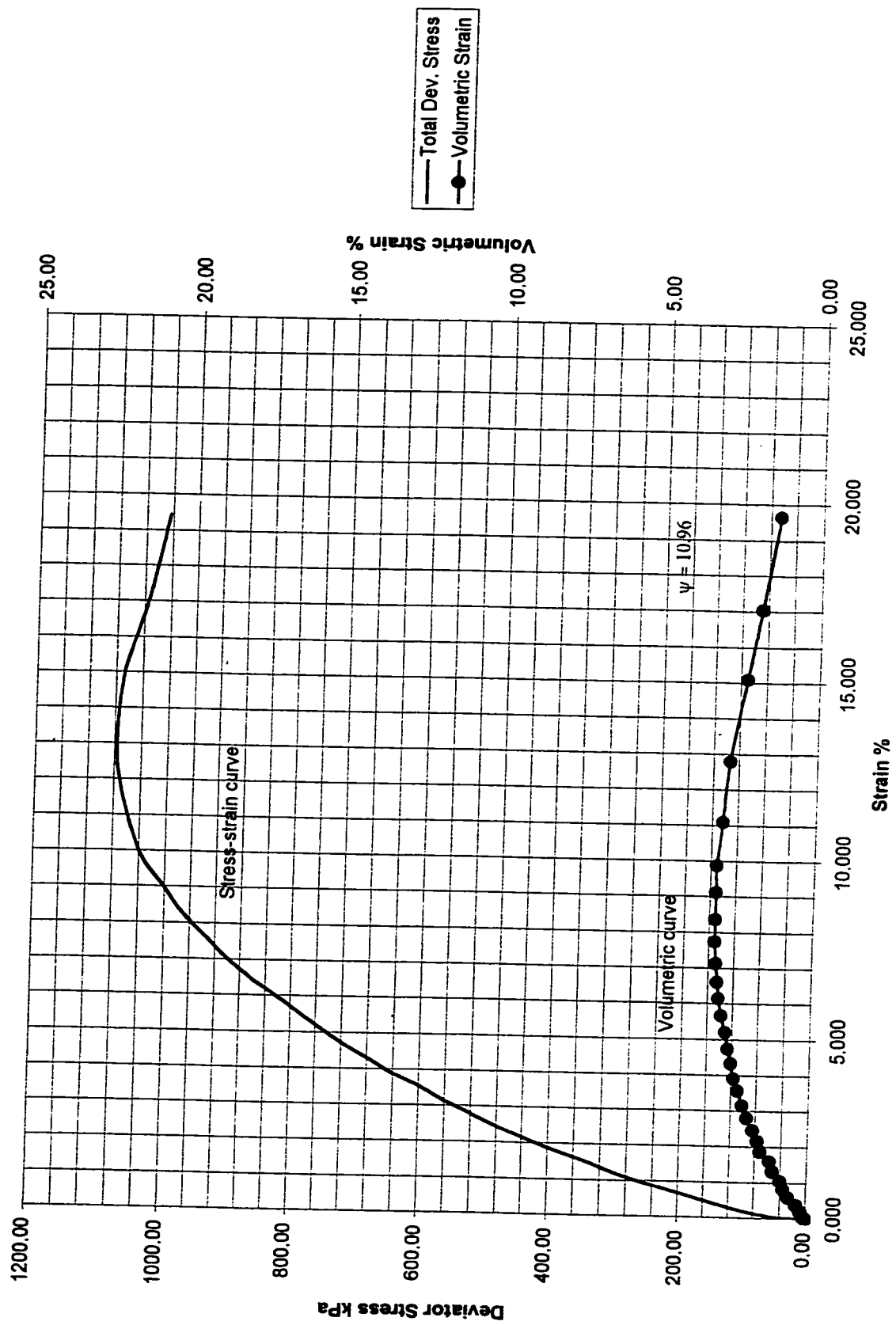


Figure 4.7: Drained triaxial test on crushed stone at a confining pressure of 200 kPa and a relative density of 85%

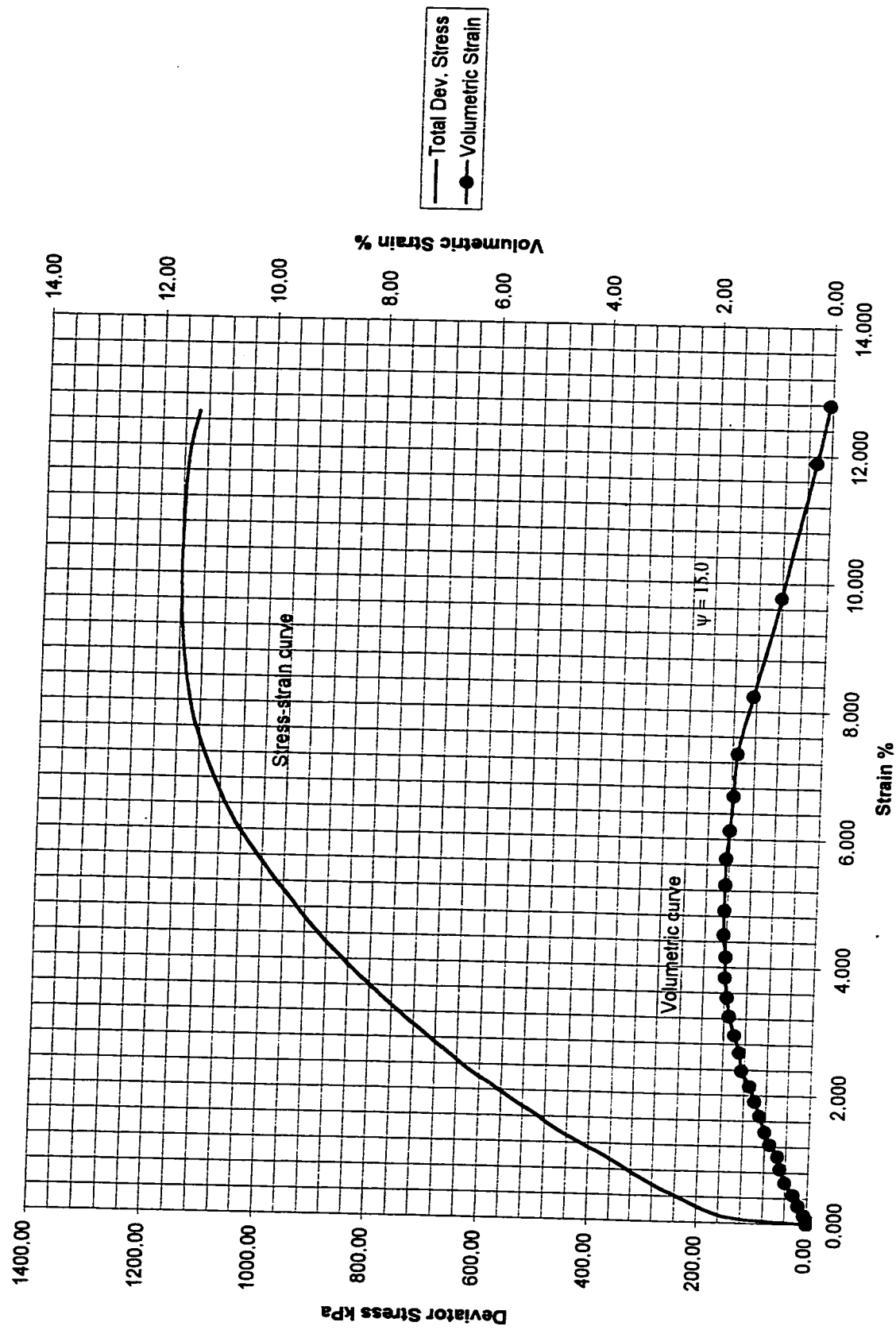


Figure 4.8: Drained triaxial test on stone dust at a confining pressure of 200 kPa and a relative density of 97%

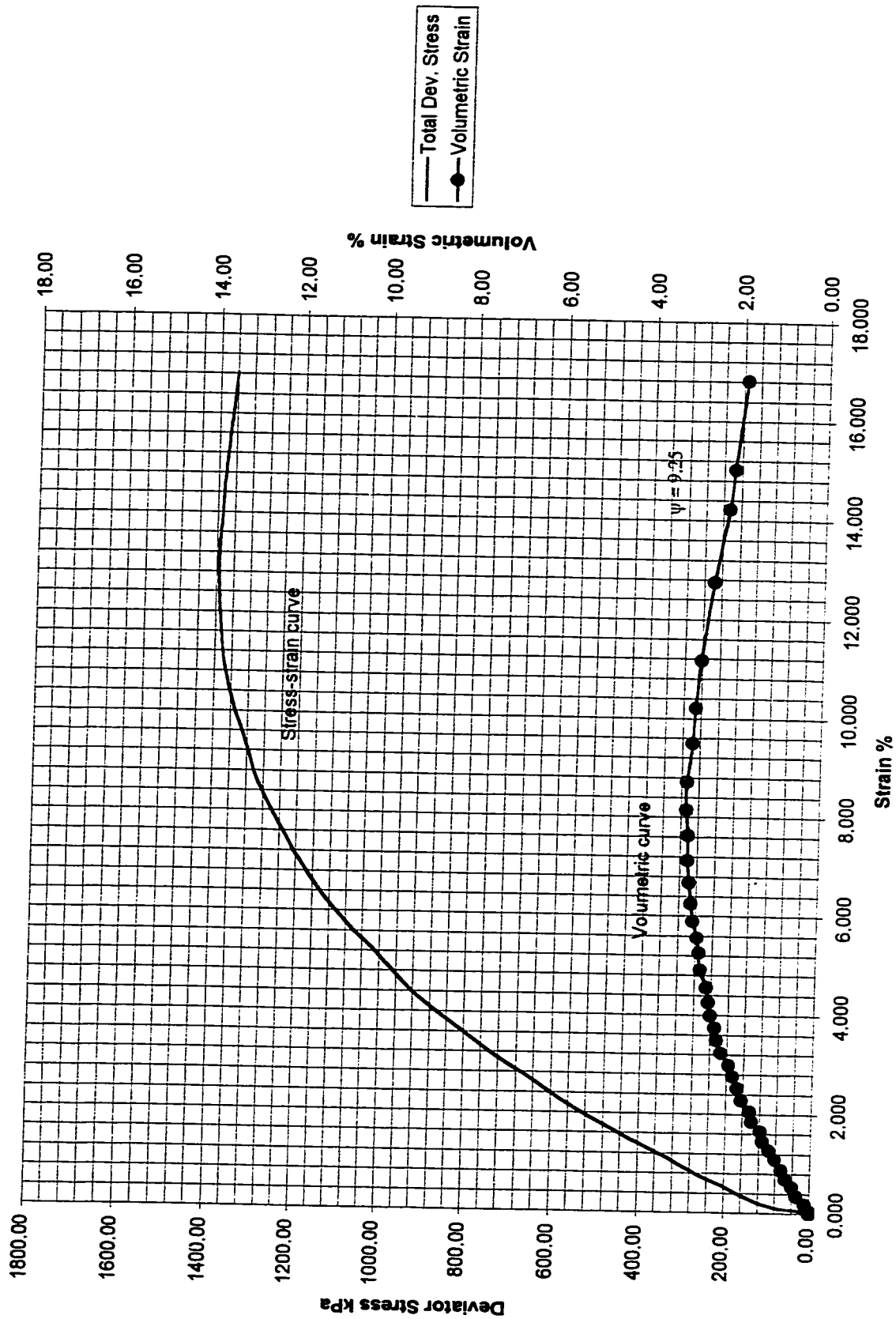


Figure 4.9: Drained triaxial test on crushed stone at a confining pressure of 300 kPa and a relative density of 98%

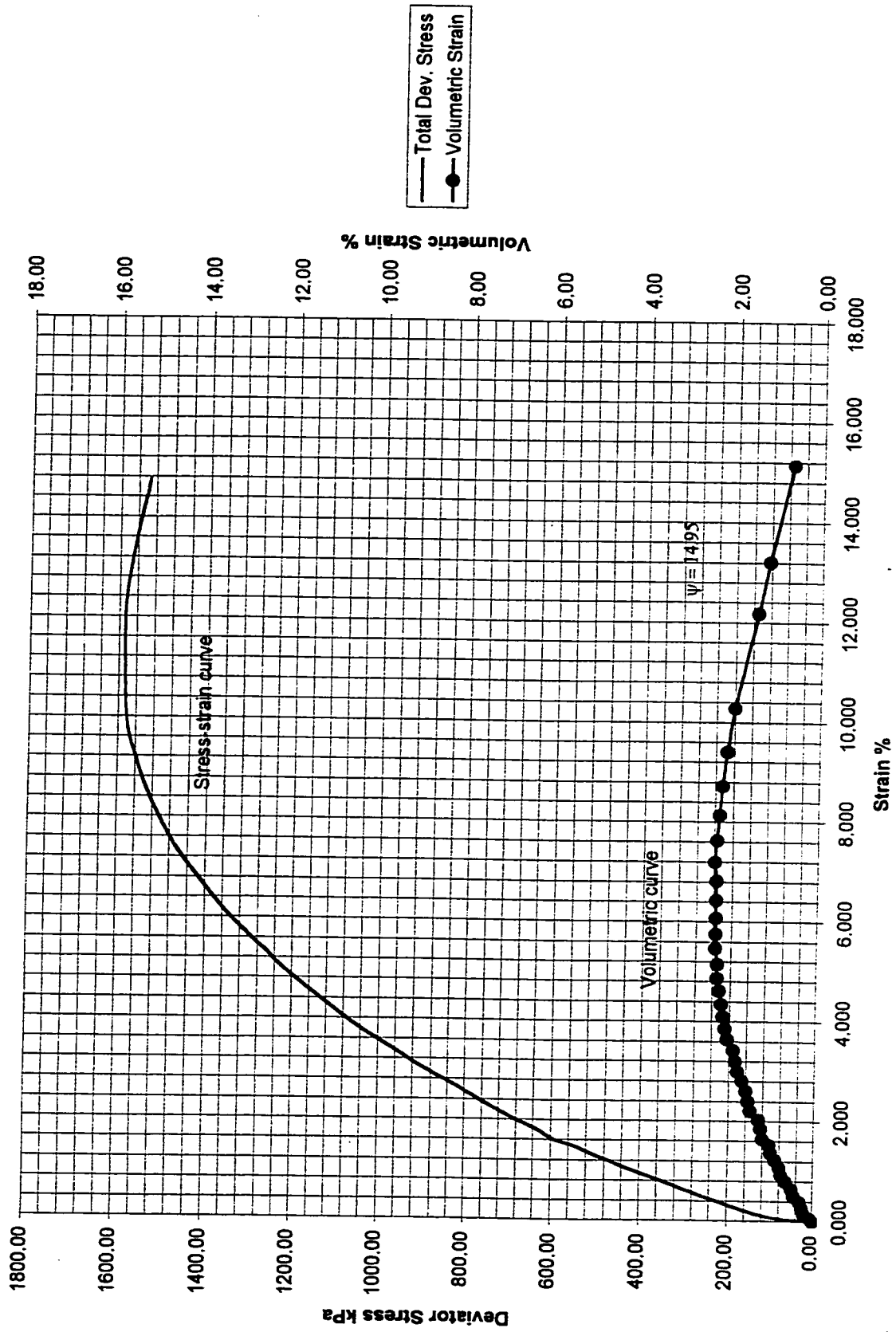


Figure 4.10: Drained triaxial test on stone dust at a confining pressure of 300 kPa and a relative density of 94%

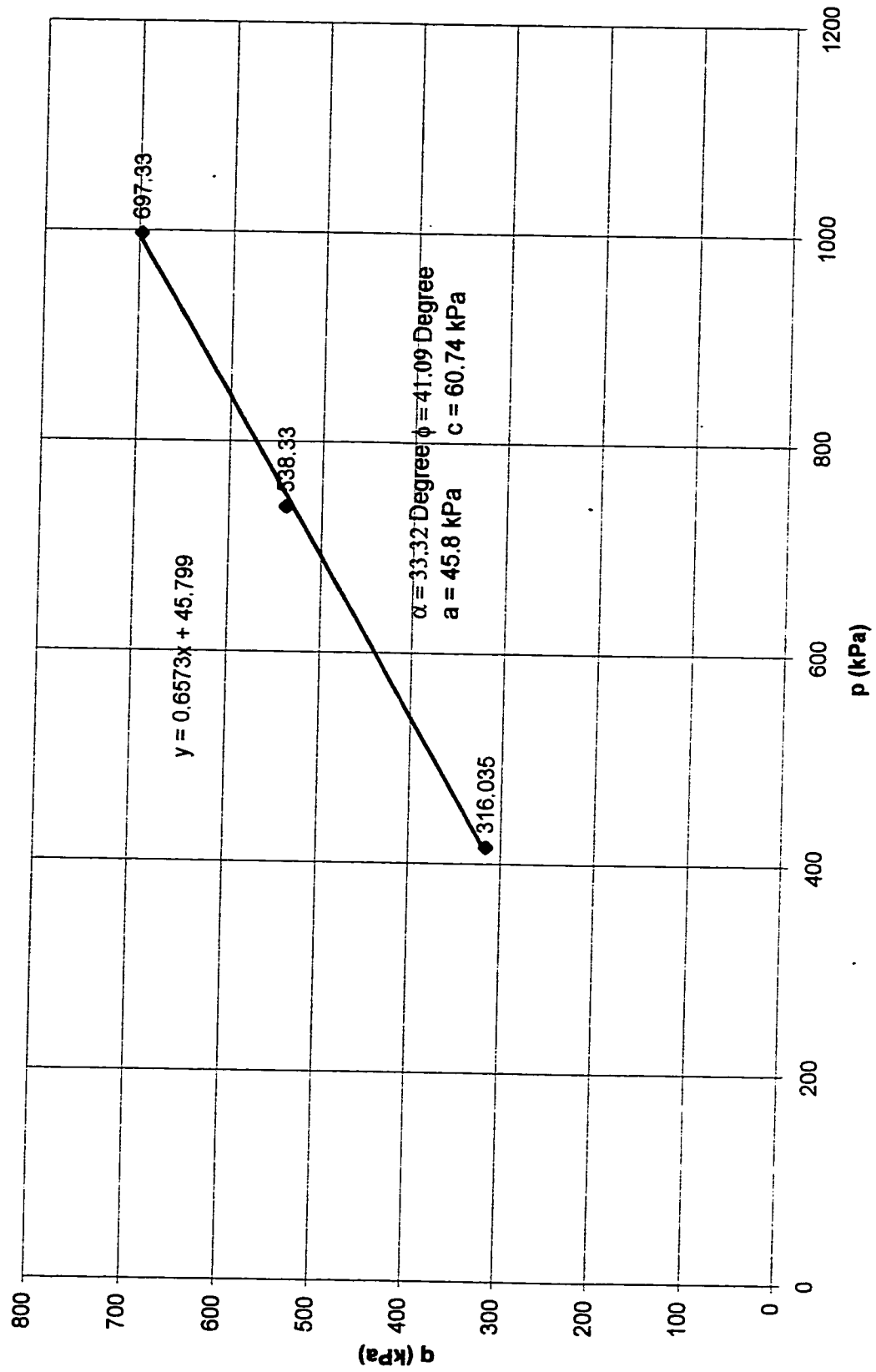


Figure 4.11: Failure envelope for the crushed stone

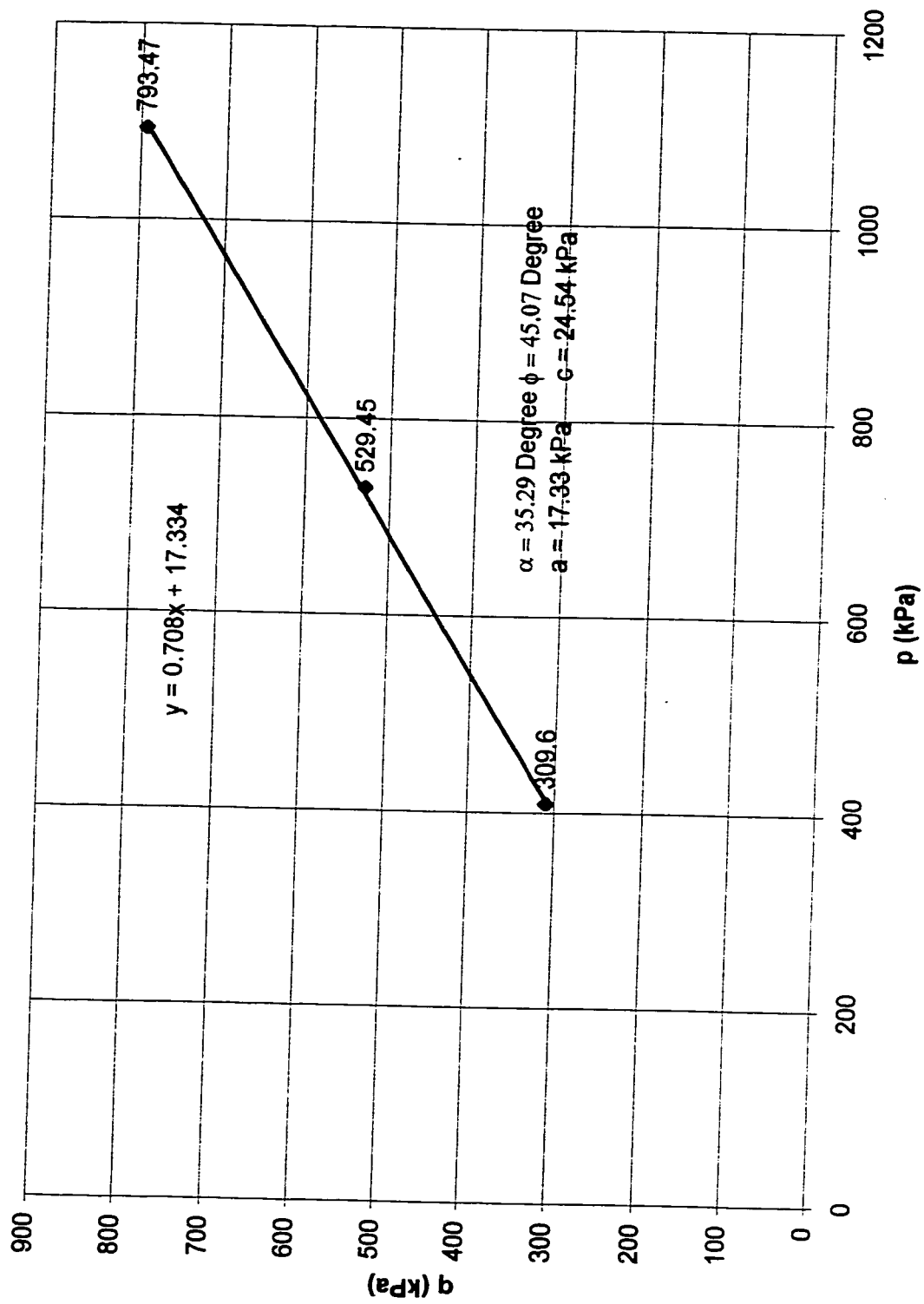


Figure 4.12: Failure envelope for the stone dust

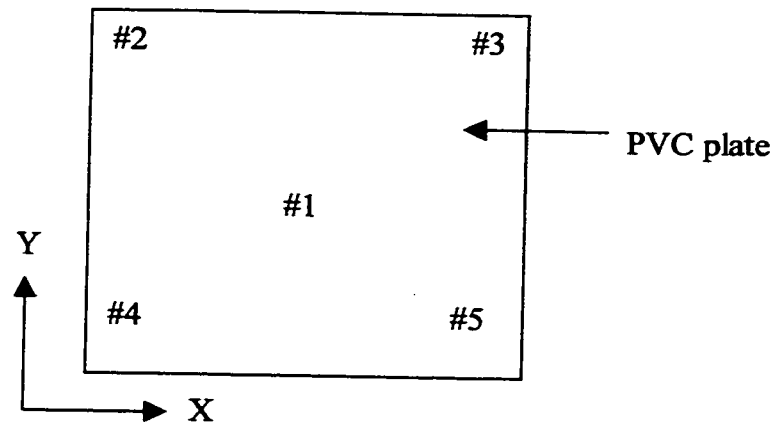


Figure 4.13: Location of roughness measurements on a PVC surface

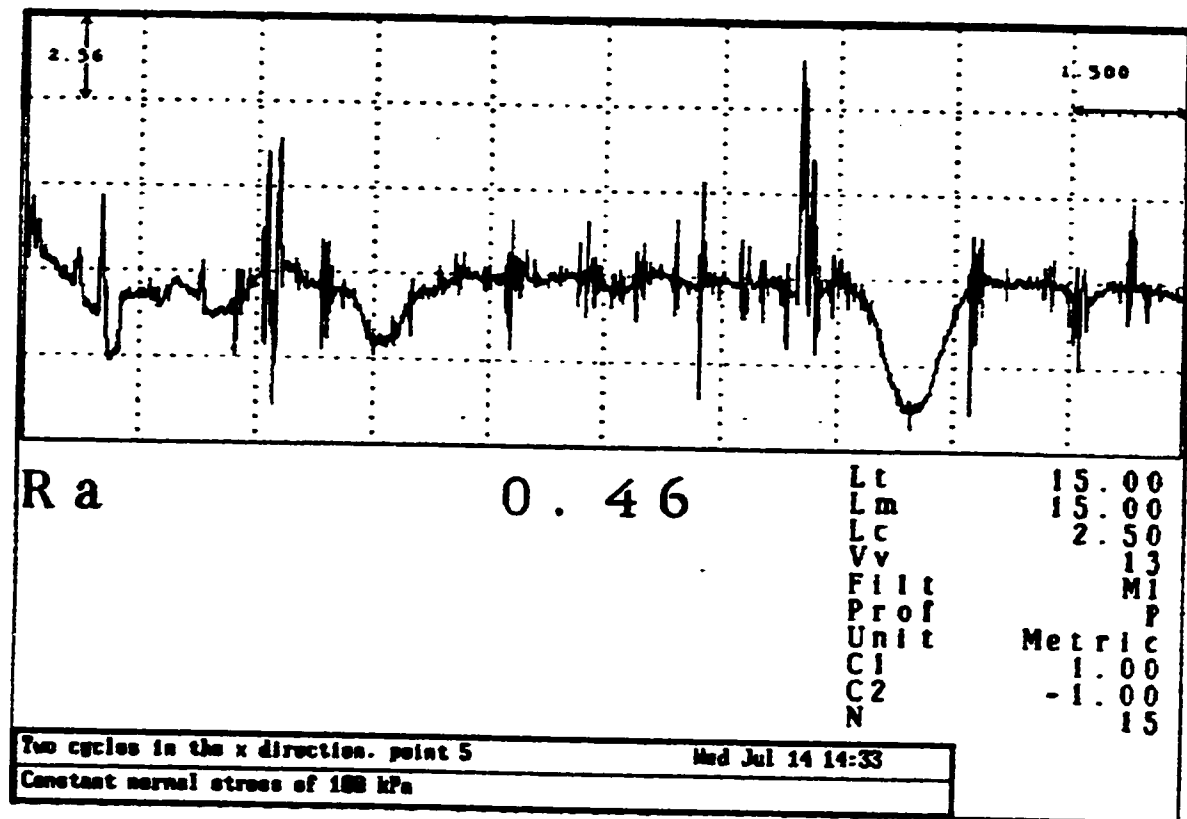
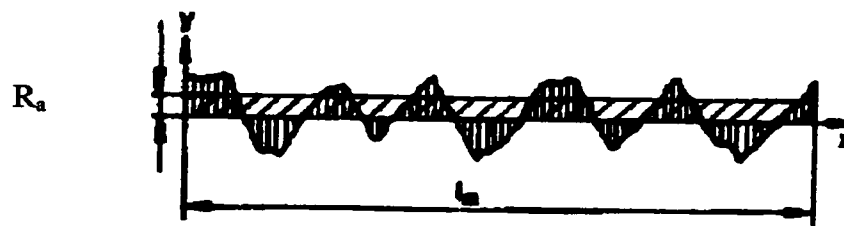


Figure 4.14: Display of the output from the Hommel software



$$R_a = \frac{1}{l_m} \int_{x=0}^{x=l_m} |y| dx$$

- l_m : The measurement length
- x : Coordinate in the surface
- y : Height of the surface measured above the mean level

Figure 4.15: Definition of the roughness parameter R_a (Williams, J.A., 1994)

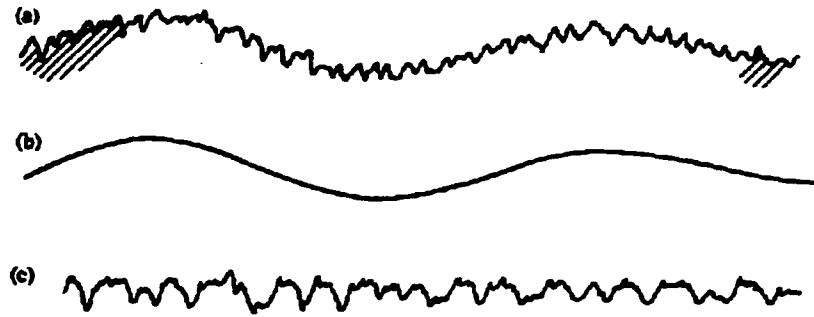


Figure 4.16: a) Total profile b) waviness profile c) roughness profile

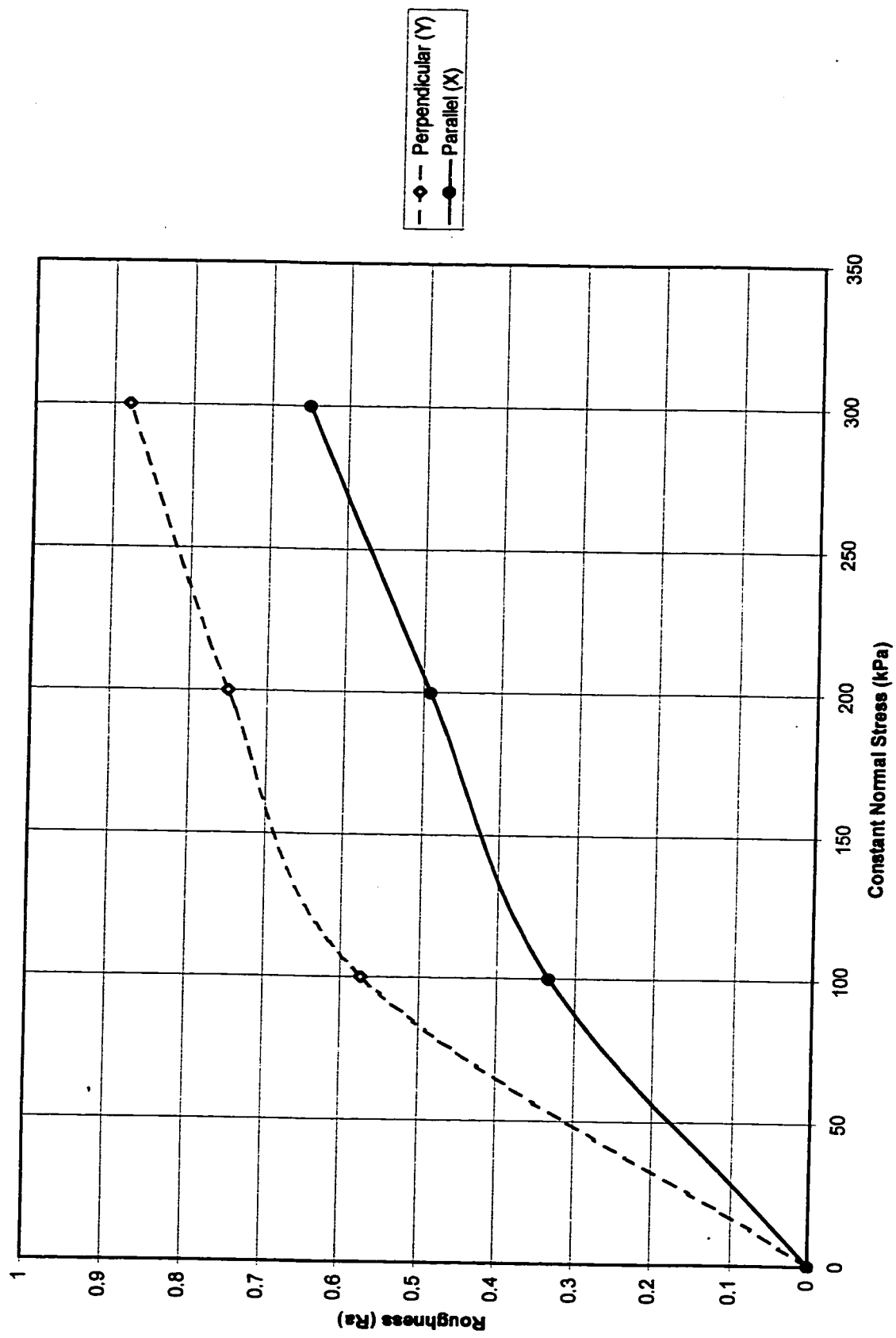


Figure 4.17: Surface roughness relative to the normal stress on PVC surfaces

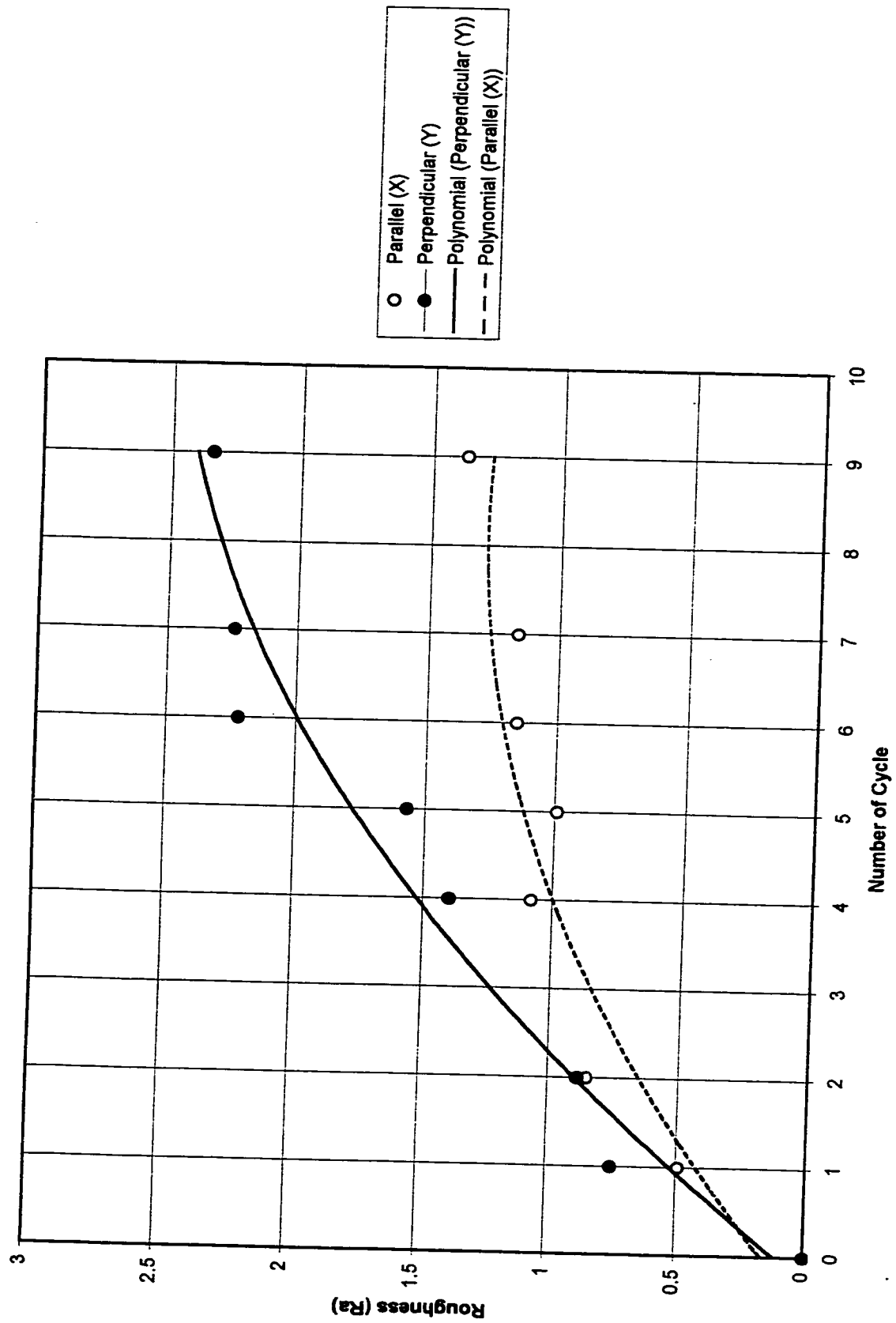


Figure 4.18: Surface roughness relative to the number of cycle on a PVC surface sheared with a constant normal stress of 200 kPa.

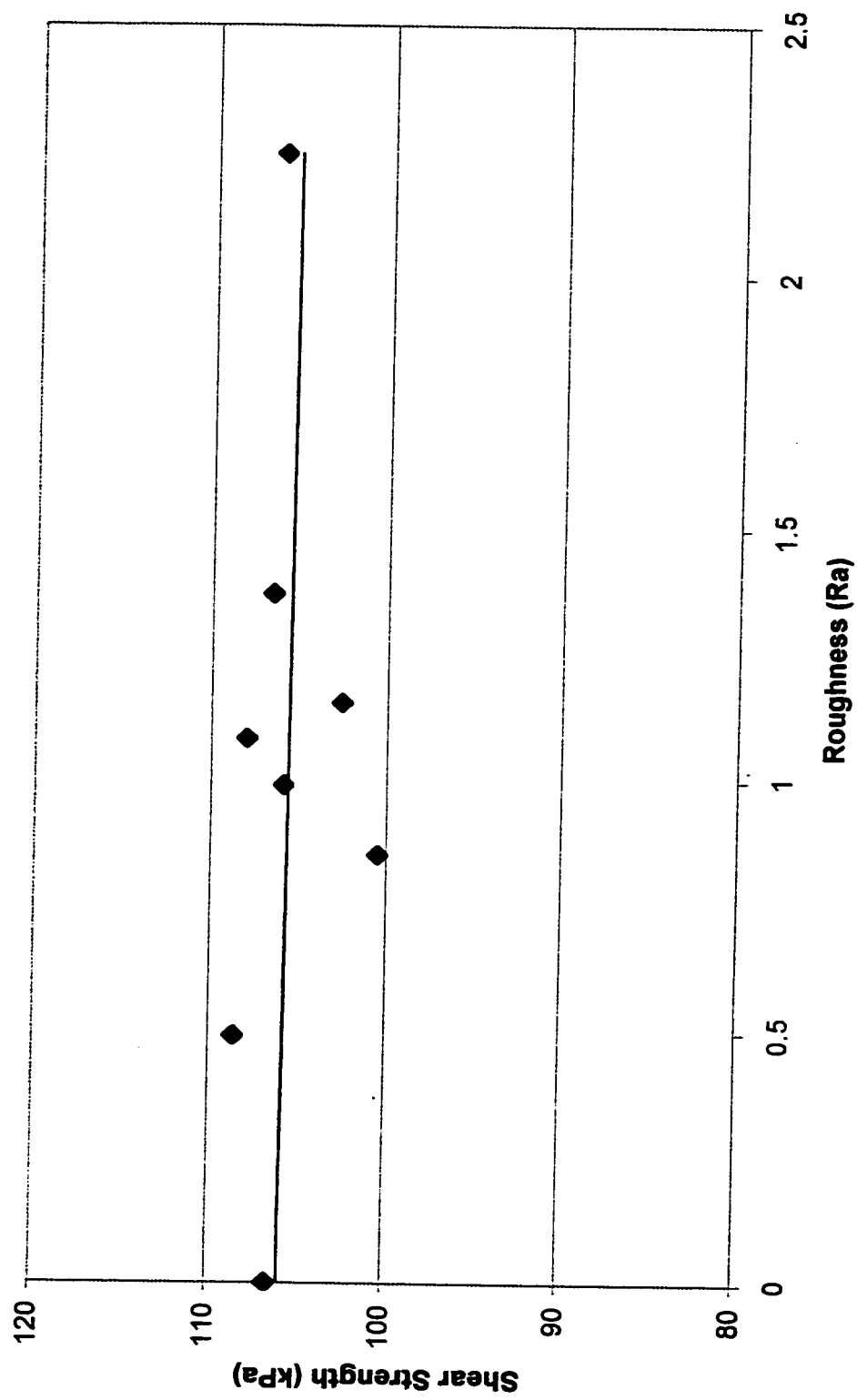


Figure 4.19: Maximum shear stress relative to the roughness of the PVC surface sheared with a constant normal stress of 200 kPa.

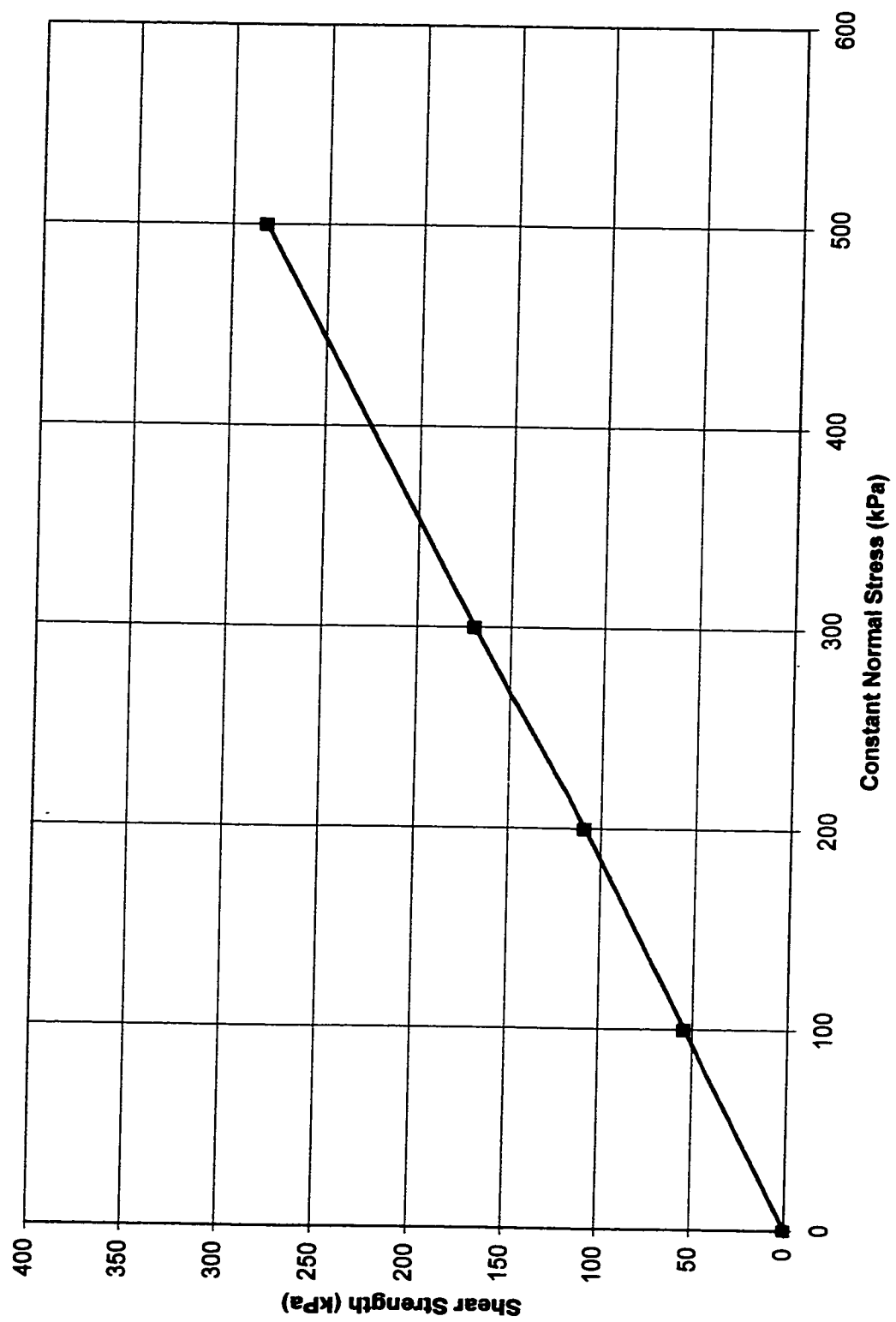


Figure 4.20: Shear stress vs normal stress.

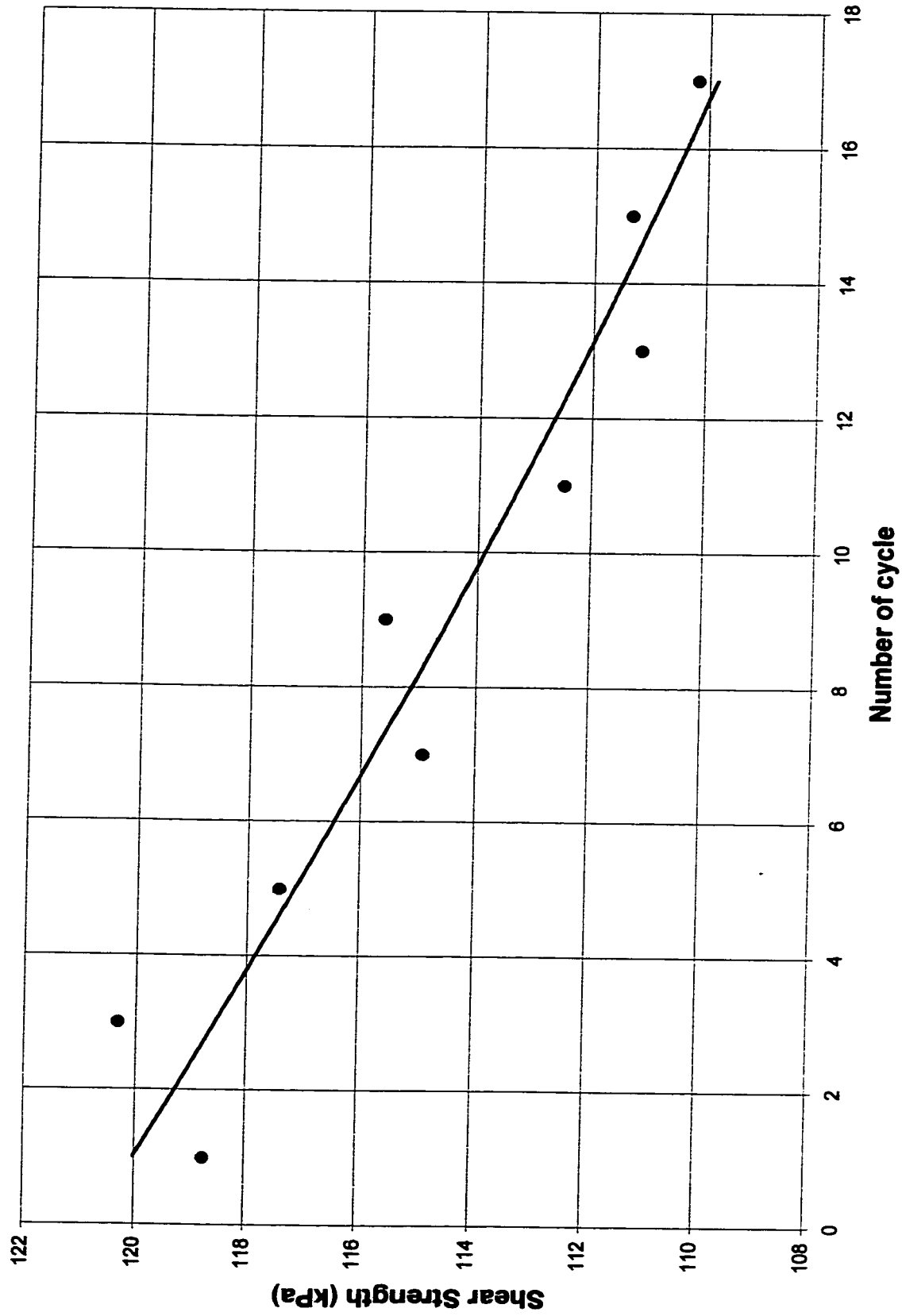


Figure 4.21: Average of shear strength relative to the number of cycle with a constant normal stress of 200 kPa.

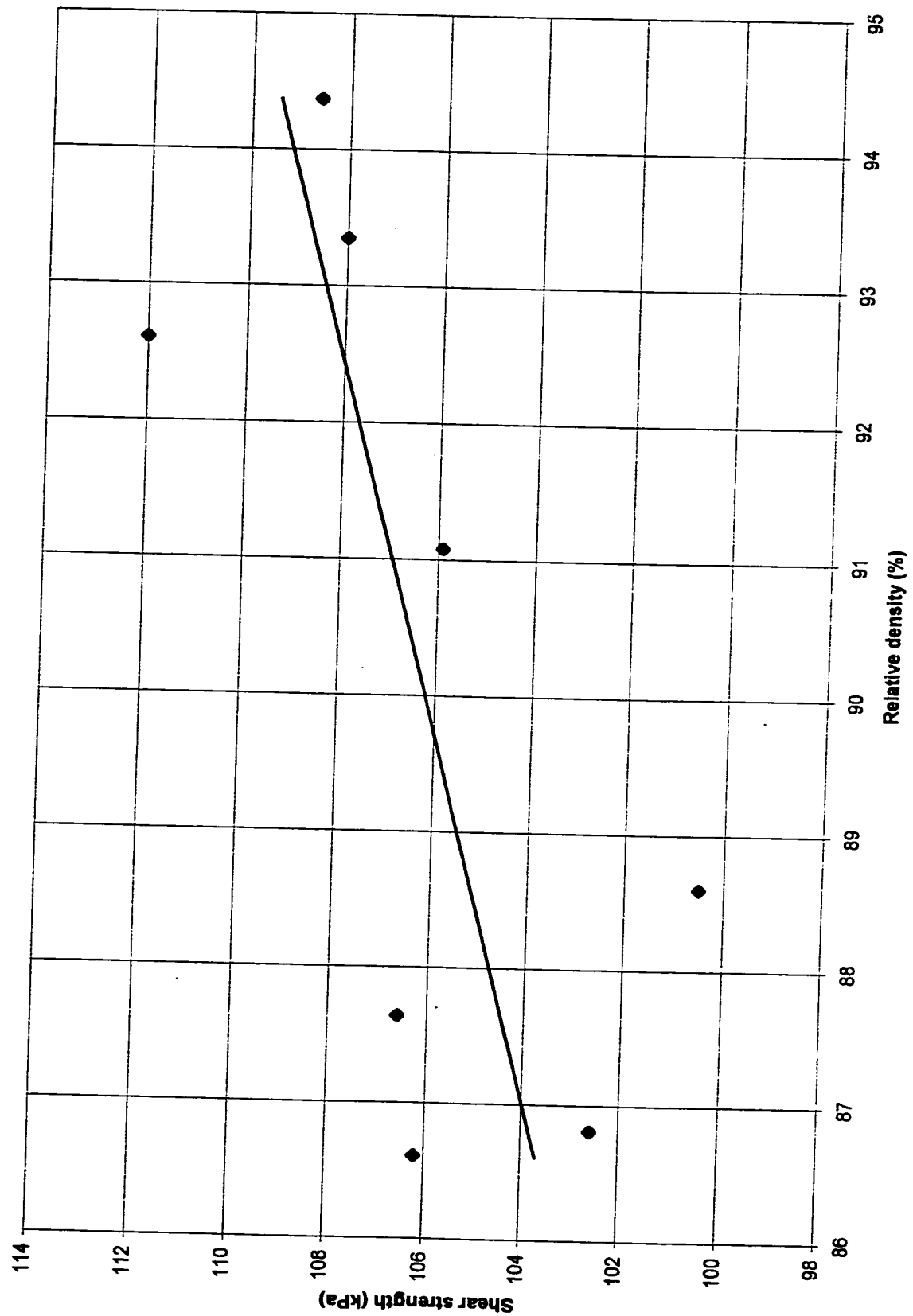


Figure 4.22: Shear strength in fonction of relative density

CHAPTER 5

MODELING OF SOIL BEHAVIOUR

The finite element method is a useful method to analyse the behaviour of a buried flexible pipe. A soil model, which simulates the behaviour of a certain type of soil, has to be specified in order to proceed with the analysis. The soil model available in PLAXIS, which represents most adequately the behaviour of the Kanto loam and the crushed stone, is the Hardening-Soil model. Hence, the Hardening-Soil model is explained in detail in the present chapter. The Variable Moduli model is also explained in detail because the input parameters, used in the Hardening-Soil model, are taken from the analysis of Mohri and Kawabata (1995) using the Variable Moduli model.

5.1 The Hardening-Soil Model (PLAXIS, 1998)

The Hardening-Soil model is an advanced soil model, included in PLAXIS, taking into account the hardening behaviour of soil and using a non-associated flow rule. In order to understand the hardening behaviour and the non-associated flow rule, it is important to review some principles of the plasticity theory. First, soils undergo elastic and plastic deformations when sheared. If a state of stress were below a yield surface, then the deformations would be only elastic or recoverable. However, if the states of stress were equal or higher than the yield surface, then the deformations would be elastic and plastic. A certain amount of the deformation would be unrecoverable. Second, a flow rule relates the direction of the vector of plastic strain increment to the yield surface. According to an associated flow rule, a vector of plastic strain increment is normal to the yield surface. On the other hand, a non-associated flow rule involves a vector of plastic strain increment that is not normal to the yield surface. Hardening behaviour happens when the yield surface increase in size as the soil undergoes plastic straining. In other words, after unloading/reloading, the soil would yield at a higher stress level than the previous stress level, which caused yielding. A hardening law relates the magnitude of the plastic strain to the magnitude of the increment of stress.

The Mohr-Coulomb model is useful to solve simple problems, or problems with very limited information about the characteristics of a soil. However, it is not realistic and does not represent the behaviour of the soil at different states of stress very well. The Mohr-Coulomb model is also called an elastic-perfectly-plastic model, because the strain of the soil can only be elastic before the yield surface or perfectly plastic when the states of stress traverse the yield surface. It is then impossible to model the hardening behaviour of soils. Figure 5.1 shows test results put into an idealised form using the Mohr-Coulomb model.

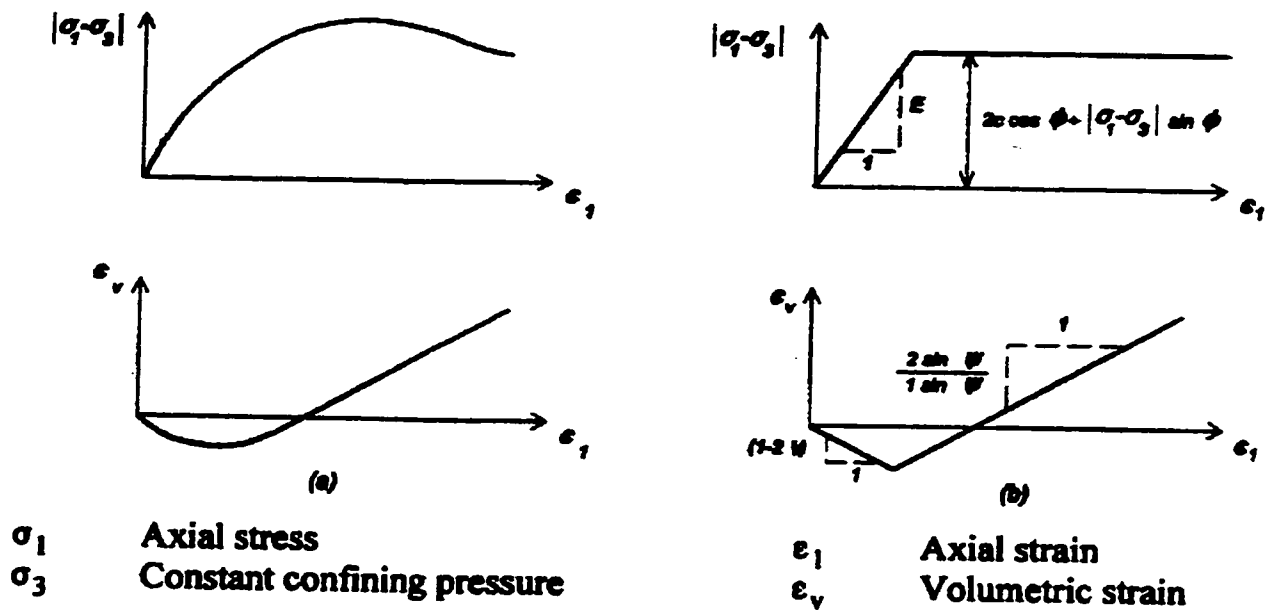


Figure 5.1: The Mohr-Coulomb's model (PLAXIS User Guide, 1998).

The Cam-clay model, also include in PLAXIS, was originally developed for soft clays. The model was subsequently modified by Roscoe and Burland (1968). A clearly written description of the Cam-clay model can be found in Atkinson and Bransby (1978).

The Hardening-Soil model can simulate the behaviour of both soft soils and stiff soils (PLAXIS, 1998). The model uses the theory of plasticity, includes soil dilatancy, has a non-associated flow rule, and introduces a yield cap for isotropic compression. The basic characteristics of the model are:

- Stress dependent stiffness according to a power law.
- Plastic straining due to primary deviatoric loading.
- Plastic straining due to primary compression.
- Elastic unloading/reloading condition.
- Failure according to Mohr-Coulomb theory.

5.1.1 Hyperbolic relationship for standard drained triaxial test

Generally speaking, the Hardening-Soil model is based on the well-known Hyperbolic model by Duncan and Chang (1970). It is then useful to establish first the hyperbolic relationship for standard drained triaxial test before getting into the details of the Hardening-Soil model.

The soil behaviour over a wide range of stresses is non-linear, inelastic, and dependent upon the magnitude of the confining pressure. Kondner (1963) found that the non-linear stress-strain curve of both clay and sand might be approximated by a hyperbolic curve with a high degree of accuracy. The stress-strain relationship, in a standard drained triaxial test, tends to yield a curve described by the following equation and shown in Figure 6.2:

$$-\varepsilon_1 = \frac{1}{2E_{50}} \frac{q}{\left(1 - \frac{q}{q_a}\right)} \quad q < q_f \quad (5.1)$$

q_a : Asymptotic value of the shear strength = q_f / R_f .

q : Deviatoric stress

q_f : Ultimate deviatoric stress

$$q_f = (c \cot \varphi - \sigma_3) \frac{2 \sin \varphi}{1 - \sin \varphi} \quad (5.2)$$

E_{50} : The confining stress dependent stiffness modulus for primary loading.

$$E_{50} = E_{50}^{ref} \left(\frac{c \cot \varphi - \sigma_3'}{c \cot \varphi + p^{ref}} \right)^m \quad (5.3)$$

E_{50}^{ref} : Reference stiffness modulus corresponding to the reference confining pressure p^{ref} . In PLAXIS, the default value of p^{ref} is 100 kPa.

σ_3' : Minor principal stress

m : A parameter for stress dependency of soils. Stiffer soils are simulated with a high value of m . Janbu reports values of m around 0.5 for Norwegian sands and silts (PLAXIS, 1998).

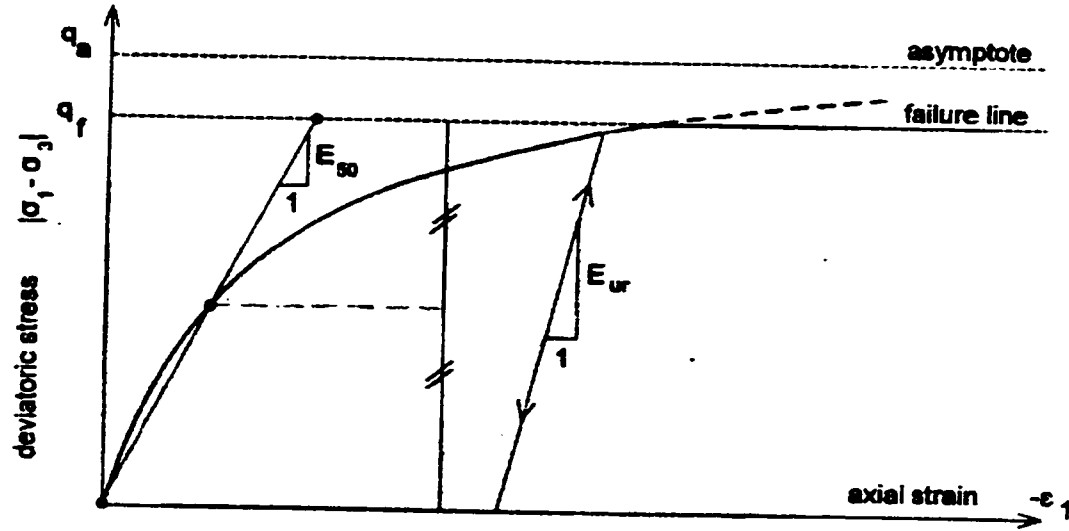


Figure 5.2: Hyperbolic stress-strain curve (PLAXIS, 1998).

For unloading/reloading stress paths, another stress-dependent stiffness modulus is used:

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cot \phi - \sigma'_3}{c \cot \phi + p^{ref}} \right)^m \quad (5.4)$$

E_{ur} : The reference Young's modulus for unloading and reloading, corresponding to the reference pressure p^{ref} .

5.1.2 Approximation of Hyperbola by the Hardening-Soil Model

The Hardening-Soil model is described in this section. The stress-strain behaviour of soils in standard drained triaxial tests is considered. A yield function of the following form is used:

$$f = \bar{f} - \gamma_p \quad (5.5)$$

$\bar{f}$ is a function of stresses as given below.

$$\bar{f} = \frac{1}{E_{s0}} \frac{q}{\left(1 - \frac{q}{q_a}\right)} - \frac{2q}{E_{ur}} \quad (5.6)$$

γ_p is a function of plastic strains

$$\gamma_p = -(2\varepsilon_1^p - \varepsilon_v^p) \quad (5.7)$$

where ε_v^p is the plastic volumetric strain.

In the case of stiff soils, the value of the volumetric plastic strain is usually very small. Therefore, γ_p can be approximated as $\gamma_p \approx -2\varepsilon_1^p$ (PLAXIS, 1998).

Primary loading of a standard drained triaxial test implies that the yield function $f = 0$, or $\gamma^p = \bar{f}$. Substituting the equation 5.6 and 5.7 in this relation gives:

$$-\varepsilon_1^p \approx \frac{1}{2} \bar{f} = \frac{1}{2E_{s0}} \frac{q}{\left(1 - \frac{q}{q_a}\right)} - \frac{q}{E_{ur}} \quad (5.8)$$

In contrast to the plastic strains, the elastic strains develop in both primary loading and unloading/reloading. For stress path with $\sigma_3 = \sigma_2 = \text{constant}$, the Young's modulus for unloading/reloading is constant and the elastic strains are given by the following equations:

$$-\varepsilon_1 = \frac{q}{E_{ur}} \quad -\varepsilon_2^e = -\varepsilon_3^e = -\nu_{ur} \frac{q}{E_{ur}} \quad (5.9)$$

ν_{ur} : Poisson's ratio

Note that the strain in the first stage of isotropic compression is considered elastic according to Hooke's law, and it is not included in Equation 5.9. The axial strain is given by the addition of the elastic component and the plastic component as shown in the next equation:

$$-\varepsilon_1 = -\varepsilon_1^e - \varepsilon_1^p \approx \frac{1}{2E_{s0}} \frac{q}{\left(1 - \frac{q}{q_a}\right)} \quad (5.10)$$

Equation 5.10 is the hyperbolic equation described before. The Hardening-Soil model produces hyperbolic stress-strain curves for standard drained triaxial tests.

5.1.3 Plastic Volumetric Strain for Triaxial State of Stress

The Hardening-Soil model involves a non-associated flow rule relating an increment of volumetric plastic strain to an increment of axial plastic strain:

$$\dot{\varepsilon}_v^p = \sin \psi_m \dot{\gamma}^p \quad (5.11)$$

ψ_m : Mobilised dilation angle

$$\sin \psi_m = \frac{\sin \varphi_m - \sin \varphi_{cv}}{1 - \sin \varphi_m \sin \varphi_{cv}} \quad (5.12)$$

φ_{cv} : Material constant representing the critical state friction angle, and being independent of the density.

φ_m : Mobilised friction angle

$$\sin \varphi_m = \frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3 - 2c \cot \varphi} \quad (5.13)$$

These equations correspond to the stress-dilatancy theory by Rowe (PLAXIS, 1998). This theory implies that the material contracts for small stress ratios ($\varphi_m < \varphi_{cv}$) and dilates for high stress ratios ($\varphi_m > \varphi_{cv}$). The value of φ_{cv} can be computed from the previous equations if the user provides input data on the ultimate friction angle and the ultimate dilatancy angle:

$$\sin \varphi_{cv} = \frac{\sin \varphi - \sin \psi}{1 - \sin \varphi \sin \psi} \quad (5.14)$$

5.1.4 Parameters for the Hardening-Soil Model

Failure parameters:

c : Cohesion [kN/m²]

ϕ : Angle of internal friction [°]

ψ : Angle of dilatancy [°]

Soil deformation parameters:

E_{50}^{ref} : Secant modulus in standard drained triaxial tests [kN/m²]

E_{oed}^{ref} : Tangent modulus for primary oedometer loading [kN/m²]

m : Power for stress-level dependency of tangent modulus [dimensionless]

Advanced parameters:

E_{ur}^{ref} : Unloading/Reloading modulus (default $E_{ur}^{ref} = 3E_{50}^{ref}$) [kN/m²]

ν_{ur} : Poisson's ratio for unloading/reloading (default $\nu_{ur} = 0.2$) [-]

p^{ref} : Reference stress for modulus (default $p^{ref} = 100$) [kN/m²]

K_o^{nc} : K_o -value for normal consolidation (default $K_o^{nc} = 1 - \sin \phi$) [-]

R_f : Failure ratio q_f/q_a (default $R_f = 0.9$) [-]

$\sigma_{tension}$: Tensile strength (default $\sigma_{tension} = 0$ stress units) [kN/m²]

$c_{increment}$: Increase of cohesion with depth (default $c_{increment} = 0$) [kN/m²]

The modulus in standard drained triaxial tests is not related to the modulus for primary oedometer loading. The user has to input these parameters independently. The tangent modulus for one-dimensional compression is given by the following equation and represented in Figure 5.3:

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cot \phi - \sigma'_1}{c \cot \phi + p^{ref}} \right)^m \quad (5.15)$$

E_{oed}^{ref} : Tangent modulus at a vertical stress of $-\sigma'_1$.

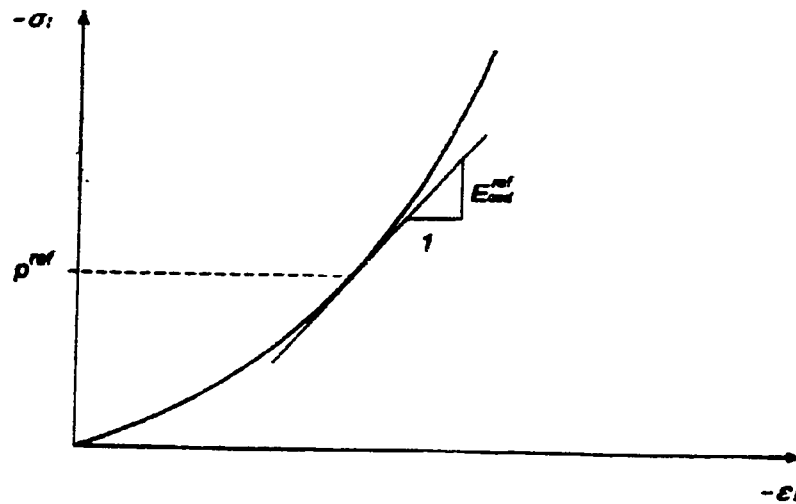


Figure 5.3: One-dimensional compression stress-strain curve (PLAXIS User Guide, 1998).

PLAXIS includes a dilatancy cut-off option controlled by an initial void ratio and a maximum void ratio. Extensively sheared soil cannot dilate indefinitely; therefore, a dilatancy cut-off set the dilation angle to zero when the maximum void ratio is reached. Figure 5.4 shows a volumetric strain-axial strain curve for standard triaxial test including a dilatancy cut-off. The void ratio is related to the volumetric strain by the following relationship:

$$-(\varepsilon_v - \varepsilon_v^{init}) = \left(\frac{1+e}{1+e_{init}} \right) \quad (5.16)$$

ε_v is positive for dilatancy.

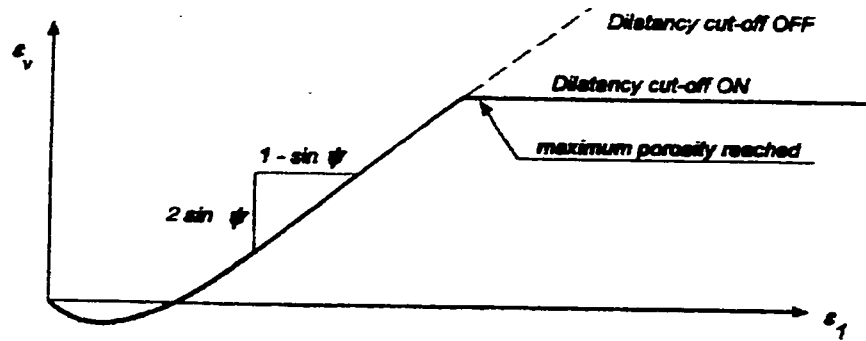


Figure 5.4: Volumetric strain-axial strain curve for standard triaxial test including a dilatancy cut-off (PLAXIS, 1998).

5.1.5 Yield Surface in Hardening-Soil Model.

The yield condition $f = 0$, at a constant hardening parameter γ_p , defines a yield surface. Successive yield surfaces at different constant hardening parameter are shown in Figure 5.5. These yield surfaces explain the plastic strain occurring in deviatoric loading. However, they do not explain the plastic volumetric strain measured in isotropic compression. Another yield surface is then defined to close the elastic region in the direction of isotopic compression. This second yield surface is called the cap yield surface, and it makes possible the formulation of a model with independent input of the secant modulus in standard drained triaxial tests (E_{50}^{ref}) and the tangent modulus for primary oedometer loading (E_{oed}^{ref}). E_{50}^{ref} controls the plastic strains associated to the shear yield surface, and E_{oed}^{ref} controls the plastic strains associated with the cap yield surface.

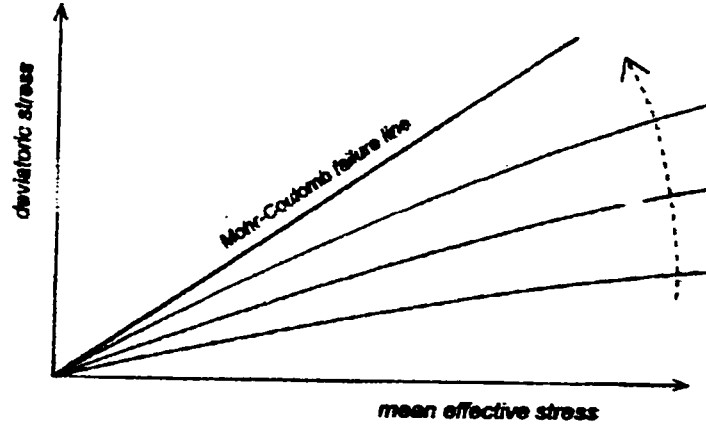


Figure 5.5: Successive yield surfaces for various constant values of the hardening parameter γ_p (PLAXIS, 1998).

The definition of the cap yield surface is similar to the equation of an ellipse:

$$f_c = \frac{\tilde{q}^2}{\alpha^2} + p^2 - p_p^2 \quad (5.17)$$

α : Cap parameter related to K_o^{nc} and defining the aspect ratio of the ellipse.

p : $-(\sigma_1 + \sigma_2 + \sigma_3)/3$

p_p : Isotropic pre-consolidation stress determining the magnitude of the yield cap or the ellipse. This parameter is provided by the PLAXIS initial stresses procedure.

$\tilde{q}$: Special stress measure for deviatoric stress $= \sigma_1 + (\delta - 1)\sigma_2 - \delta\sigma_3$

δ : $(3 + \sin\phi)/(3 - \sin\phi)$

The special stress measure for deviatoric stress yields $\tilde{q} = -(\sigma_1 - \sigma_3)$ for triaxial compression, and yields $\tilde{q} = -\delta(\sigma_1 - \sigma_3)$ for triaxial extension. In addition, a hardening law relating p_p to volumetric cap strain ε_v^{pc} defines the yield cap:

$$\varepsilon_v^{pc} = \frac{\beta}{m+1} \left(\frac{p_p}{p^{ref}} \right)^{m+1} \quad (5.18)$$

β : Cap parameter related to the tangent modulus for primary oedometer loading.

The ellipse is used both as a yield surface and as a plastic potential surface:

$$\dot{\varepsilon}^{pc} = \lambda \frac{\partial f_c}{\partial \sigma} \quad \lambda = \frac{\beta}{2p} \left(\frac{p_p}{p^{ref}} \right)^m \frac{\dot{p}_p}{p^{ref}} \quad (5.19)$$

Figure 5.6 shows the two yield surfaces in a $\tilde{q} - p$ plane, and figure 6.7 shows the surfaces in principal stress space.

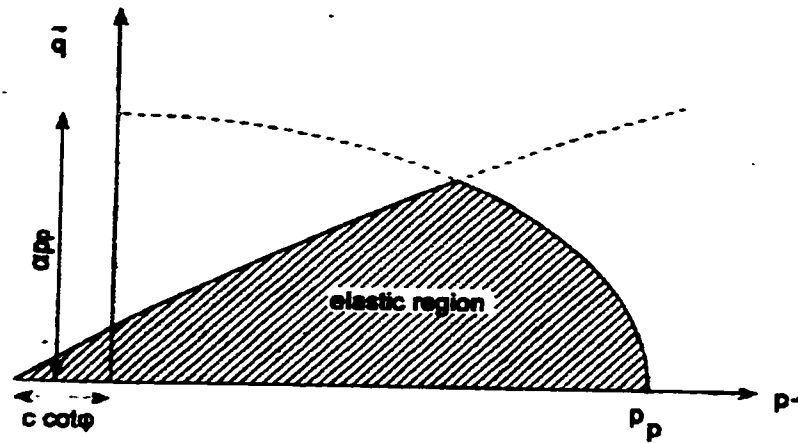


Figure 5.6: Yield surface and cap yield surface in the Hardening-Soil model (PLAXIS, 1998).

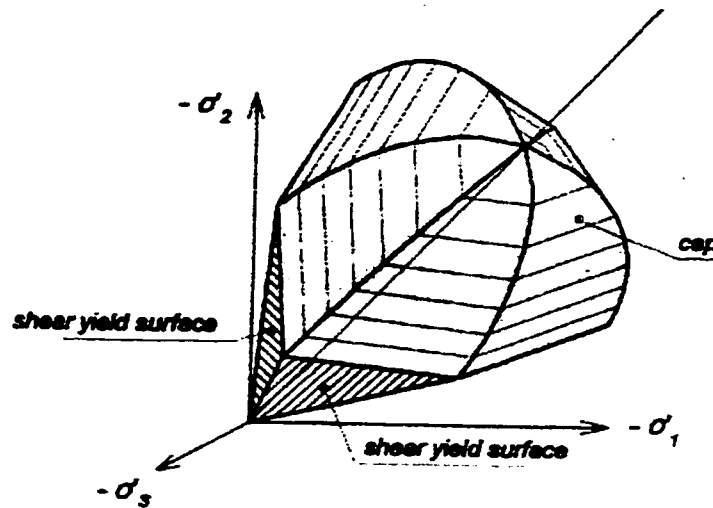


Figure 5.7: Yield surface and cap yield surface in principal stress space in the Hardening-Soil model (PLAXIS, 1998).

5.2 Variable Moduli Model of Nelson and Baron

The variable moduli model of Nelson and Baron (1971) is a mathematical model that does not contain an explicit yield surface. It has a bulk and a shear modulus, which are functions of the invariants of the stress and/or strain tensors. The following equations describe the model in terms of incremental stress-strain relations:

$$\dot{s}_{ij} = 2G\dot{e}_{ij} \quad (5.20)$$

$$\dot{p} = 3K\dot{e} \quad (5.21)$$

$\dot{s}_{ij}$: Deviatoric stress increments

$\dot{e}_{ij}$: Deviatoric strain increments

$\dot{p}$: Mean stress increment

$\dot{e}$: Mean strain increment

G : Shear modulus

K : Bulk modulus

The assumptions made in the Variable Moduli model are that the material is isotropic, both the shear and the bulk modulus depend upon the stress and/or strain invariants, the strains are small, and the loading is quasi-static.

5.2.1 Combined Stress-Strain Variable Moduli Model

The initial loading of the combined stress-strain Variable Moduli model is defined by a bulk modulus, which is a function of the mean strain, and a shear modulus, which is a function of two stress invariants. These relations are:

$$K = K(e) = K_o + K_1 e + K_2 e^2 \quad (5.22)$$

$$G = G(p, \sqrt{J_2'}) = G_o + \gamma_1 \cdot p + \bar{\gamma}_1 \cdot \sqrt{J_2'} \quad (5.23)$$

$p = (\sigma_1 + 2\sigma_3)/3$: Mean stress

$\sqrt{J_2'}$: Square root of the second invariant of deviatoric stress state

$\gamma_1, \bar{\gamma}_1$: Constants
 K_o, K_1, K_2 : Constants
 e : Mean strain

Equation 5.21 shows that the bulk modulus refers to the mean incremental pressure and mean incremental strain. According to equation 5.23, with γ_1 positive and γ_2 negative, the material hardens in shear with increasing pressure and softens with increasing shear stress. The pressure may be obtain by direct integration of equation (5.21):

$$p = \int_0^e 3K(\xi)d\xi = 3K_o e + \frac{3}{2}K_1 e^2 + K_2 e^3 \quad (5.24)$$

5.2.2 Uniaxial Strain

By using stress symmetry and that two of the principal strain rates become zero, the uniaxial strain may be expressed:

$$\frac{d\sigma_1}{de} = 3K + 4G = 3[K_o + K_1 e + K_2 e^2] + 4[G_o + \gamma_1 p + \bar{\gamma}_1 \cdot \sqrt{J_2}] \quad (5.25)$$

$$\sqrt{J_2} = \frac{\sqrt{3}}{2}s_1 = \frac{\sqrt{3}}{2}(\sigma_1 - p)$$

Substituting equation (5.24) into equation (5.25):

$$\begin{aligned} \frac{d\sigma_1}{de} - 2\sqrt{3}\gamma_1\sigma_1 = & (3K_o + 4G_o) + 3\left[4K_o\left(\gamma_1 - \frac{\sqrt{3}}{2}\bar{\gamma}_1\right) + K_1\right]e + 3\left[2K_1\left(\gamma_1 - \frac{\sqrt{3}}{2}\bar{\gamma}_1\right) + K_2\right]e^2 \\ & + 4K_2\left(\gamma_1 - \frac{\sqrt{3}}{2}\bar{\gamma}_1\right)e^3 \end{aligned} \quad (5.26)$$

Using the initial condition that stress and strain become zero simultaneously, the solution for stress as a function of strain is found by integrating equation (5.26):

$$\begin{aligned}
\sigma_1 = & - \left\{ \frac{2G_o}{\sqrt{3}\gamma_1} + \frac{\gamma_1}{\gamma_1^2} \left[K_o + \frac{K_1}{2\sqrt{3}\gamma_1} + \frac{2K_2}{(2\sqrt{3}\gamma_1)^2} \right] \right\} [1 - \exp(2\sqrt{3}\gamma_1 e)] \\
& - \left\{ \frac{2\sqrt{3}}{\gamma_1} K_o \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) + \frac{\gamma_1}{\gamma_1^2} \left(K_1 + \frac{K_2}{\sqrt{3}\gamma_1} \right) \right\} e - \left\{ \frac{\sqrt{3}K_1}{\gamma_1} \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) + \frac{\gamma_1}{\gamma_1^2} K_2 \right\} e^2 \\
& - \frac{2K_2}{\sqrt{3}\gamma_1} \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) e^3
\end{aligned} \tag{5.27}$$

From equation (5.27), the slope of the stress-strain curve at any point, or the tangent modulus is:

$$\begin{aligned}
\frac{d\sigma_1}{d\varepsilon_1} = & - \left\{ \frac{4}{3} G_o + \frac{2\gamma_1}{\sqrt{3}\gamma_1} \left[K_o + \frac{K_1}{2\sqrt{3}\gamma_1} + \frac{2K_2}{(2\sqrt{3}\gamma_1)^2} \right] \right\} [\exp(2\sqrt{3}\gamma_1 \varepsilon_1)] \\
& - \left\{ \frac{2K_o}{\sqrt{3}\gamma_1} \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) - \frac{\gamma_1}{3\gamma_1^2} \left(K_1 + \frac{K_2}{\sqrt{3}\gamma_1} \right) \right\} - \left\{ \frac{2K_1}{\sqrt{3}\gamma_1} \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) + \frac{2\gamma_1}{3\gamma_1^2} K_2 \right\} \varepsilon_1 \\
& - \frac{2K_2}{\sqrt{3}\gamma_1} \left(\gamma_1 - \frac{\sqrt{3}}{2} \gamma_1 \right) \varepsilon_1^2
\end{aligned} \tag{5.28}$$

5.2.4 Triaxial Stress

In the case of triaxial stress, the second invariant of the stress deviator is related to the stress difference by:

$$\sqrt{J_2'} = \frac{\sqrt{3}}{2} s_1 = \frac{1}{\sqrt{3}} (\sigma_1 - \sigma_3) \tag{5.29}$$

The strain deviator e_1 is found by integration:

$$e_1 = \int \frac{ds_1}{2G} = \int_{\sigma_3}^{\sigma_1} \frac{d\xi}{3G_o + \sigma_3 (2\gamma_1 - \sqrt{3}\gamma_1) + \xi(\gamma_1 + \sqrt{3}\gamma_1)} \tag{5.30}$$

The strain deviator e_1 is obtained as a function of the stresses σ_1 and σ_3 , since $e_1 = 0$ when $\sigma_1 = \sigma_3$:

$$e_1 = \frac{1}{\gamma_1 + \sqrt{3}\gamma_1} \ln \left[\frac{3G_o + \sigma_3(2\gamma_1 - \sqrt{3}\gamma_1) + \sigma_1(\gamma_1 + \sqrt{3}\gamma_1)}{3(G_o + \gamma_1\sigma_3)} \right] \quad (5.31)$$

From equation (5.23), the maximum stress deviator may be written as:

$$(\sigma_1 - \sigma_3)_{\max} = -\frac{3(G_o + \gamma_1\sigma_3)}{\gamma_1 + \sqrt{3}\gamma_1} \quad (5.32)$$

The slope of the triaxial stress-strain curve is:

$$\frac{d\sigma_1}{d\varepsilon_1} = \frac{9KG}{3K + G} \equiv E \quad (5.33)$$

The measured strain $\Delta\varepsilon_1$ is related to the strain deviator e_1 , the mean strain e , and the initial (hydrostatic) mean strain e_o by:

$$\Delta\varepsilon_1 = e_1 + e - e_o \quad (5.34)$$

5.3 Application of the Variable Moduli Model

Mohri and Kawabata (1995) used the Variable Moduli Model in their numerical analysis. It would have been easier to use directly the Variable Moduli Model for the present analysis; However, the PLAXIS finite element code does not include this model. As stated before, the Hardening-Soil Model available in PLAXIS is used in this study. The soil parameters needed for the Hardening-Soil Model are determined from the stress-strain curves calculated in the present study by using the Variable Moduli Model and the model parameters provided by Mohri and Kawabata.

Kanto loam is the native soil, and it is defined by the Japanese unified soil classification system as inorganic silt. The backfill material, which replaces the native soil around the buried pipe, is a crushed granular material up to 25 mm in size. Table 5.1 below presents the parameters for the Variable Moduli Model as published by Mohri and Kawabata (1995):

Table 5.1: Input parameters for the Variable Moduli model provided by Mohri and Kawabata (1995).

Parameters	Kanto Loam	Crushed Gr.	Parameters	Kanto Loam	Crushed Gr.
G_o (MN/m ²)	2.028	1.262	K_o (MN/m ²)	3.436	2.2
G_{ou} (MN/m ²)	5.631	1.696	K_{ou} (MN/m ²)	1.533	5.449
G_{or} (MN/m ²)	6.785	4.088	K_{or} (MN/m ²)	3.628	6.977
$\bar{\gamma}_l$	-126.2	-25.73	K_l (MN/m ²)	-33.669	-31.967
$\bar{\gamma}_{lu}$	122.3	87.34	K_{lu}	132.47	288.58
$\bar{\gamma}_{lr}$	-422.27	-50.69	K_{lr}	19.78	74.56
γ_l	49.79	16.66	K_2 (MN/m ²)	26005	19.396
γ_{lu}	27.15	5.85			
γ_{lr}	166.61	43.70			

Three triaxial drained tests for each type of soil are simulated at a confining pressure of 100, 200 and 300 kPa. Stress-strain curves are shown in Figures 5.8, 5.9, 5.10, and give the value of the secant modulus (E_{50}). Unloading/reloading stress-strain curves, which show the unloading/reloading modulus (E_{ur}^{ref}), can be visualised in Figures 5.17 and 5.18. Volumetric strain-axial strain curves are shown in Figures 5.11 and 5.12. The simulations also allow the possibility of finding a failure envelope by drawing Mohr circles at failure (Figures 5.13 and 5.14) for each test, and then calculate a friction angle (ϕ) and a cohesion value (c). Moreover, Figure 5.15 and 5.16 show a simulation of oedometer tests. The tangent modulus for primary oedometer loading (E_{oed}) is calculated from these graphs. All graphs are produced using ten increments of load. Table 5.2 shows the parameters calculated from the Variable Moduli Model and used in the Hardening-Soil model:

Table 5.2: Parameters calculated from the Variable Moduli model and used in the Hardening-Soil model.

Parameters	Kanto Loam	Crushed Stone
Friction angle ϕ (°)	17.9	28.2
Dilation angle ψ (°)	0.0	0.0
Cohesion c (kPa)	13.1	40.8
E_{50}^{ref} (kPa)	11 554.9	5602.8
E_{oed}^{ref} (kPa)	15 192.7	17 554.3
E_{ur}^{ref} (kPa)	23 760.0	20 930.2

The Variable Moduli model gives low values for the friction angles for both Kanto loam and crushed stone. The cohesion is abnormally high for the crushed stone. It is impossible to use the cohesion equal to zero for the crushed stone, which would be more realistic, because the friction angle is too low and the Hardening-Soil model would simulate an excess of plastic deformations. Considering the experimental results given in Chapter 3, a higher friction angle and a lower value for cohesion would be more realistic for the crushed stone. However, the present values, taken from the Variable Moduli model with the input parameters of Mohri and Kawabata, give acceptable results in the finite element analysis.

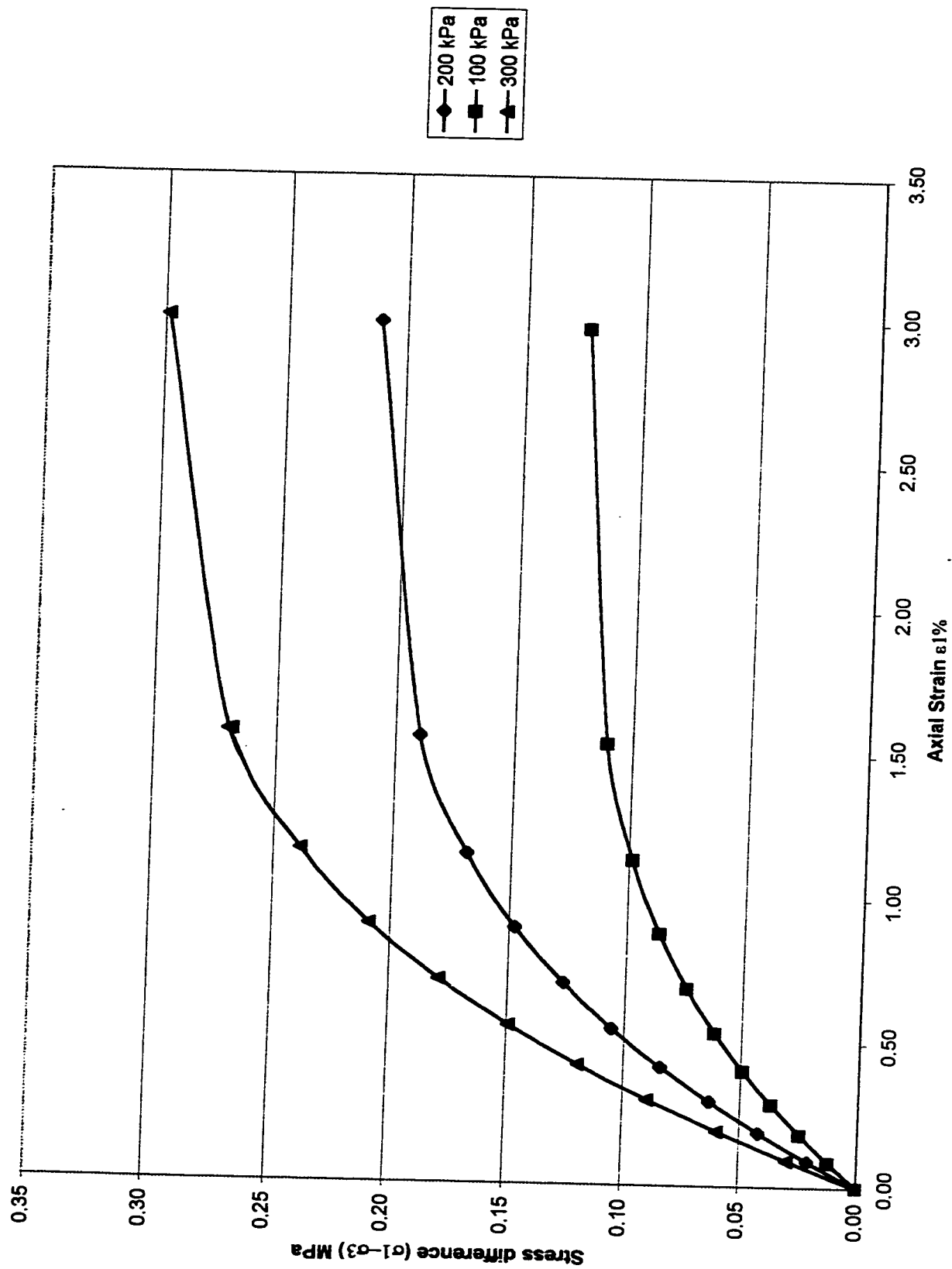


Fig.5.8: Calculated stress-strain curves using the Variable Moduli model for the Kanto Loam at a confining pressure of 100, 200 and 300 kPa.

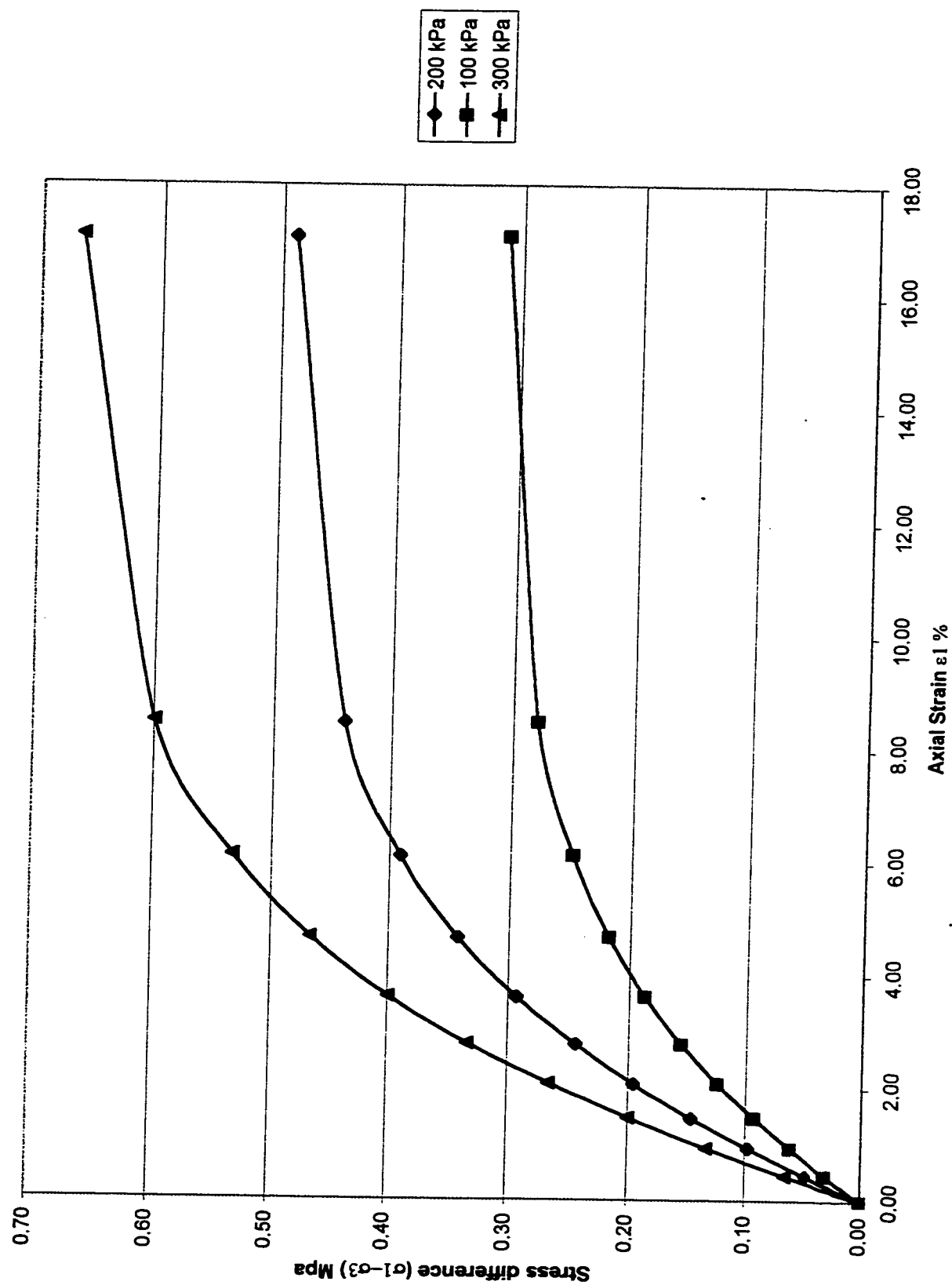


Fig.5.9: Calculated stress-strain curves using the Variable Moduli model for the crushed stone at a confining pressure of 100, 200 and 300 kPa.

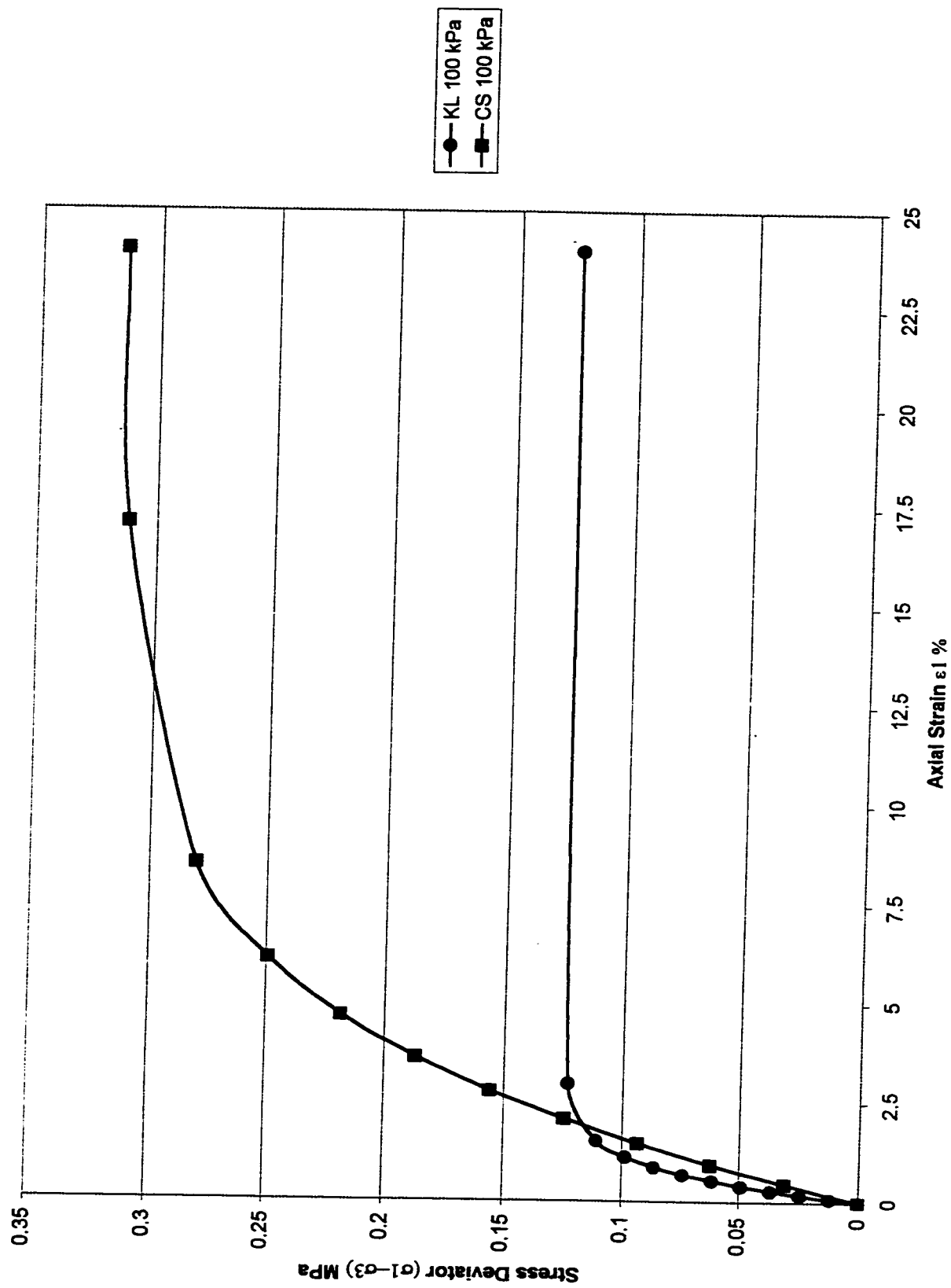


Figure 5.10: Comparison of the stress-strain curves of the Kanto loam and the crushed stone at a confining pressure of 100 kPa.

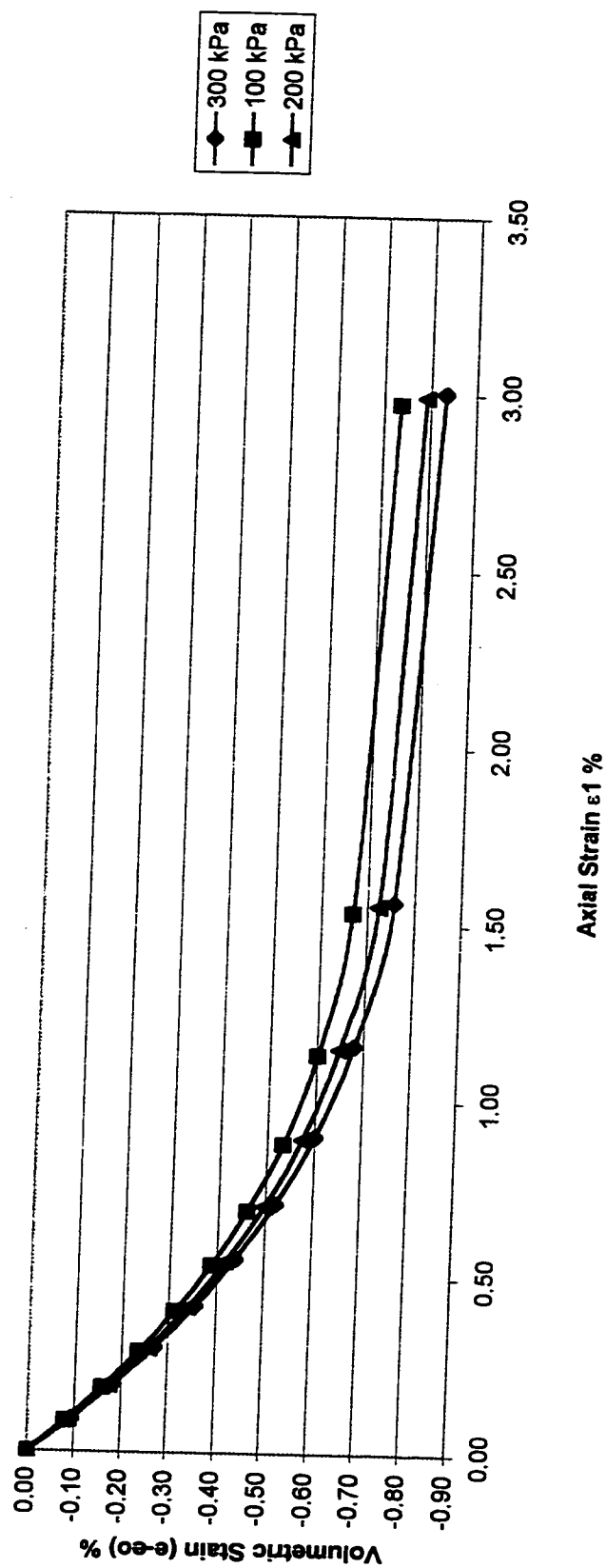


Figure 5.11: Calculated volumetric strain-axial strain curves using the Variable Moduli model for the Kanto loam at a confining pressure of 100, 200 and 300 kPa.

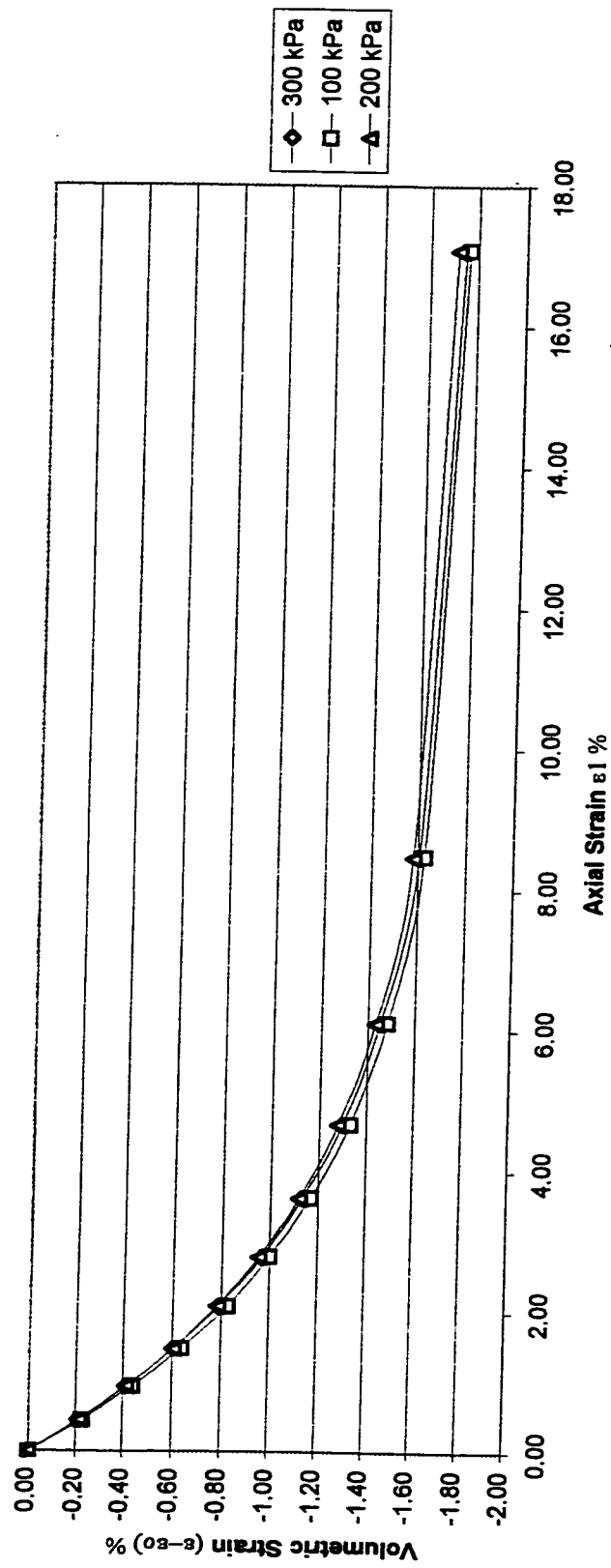


Figure 5.12: Calculated volumetric strain-axial strain curves from the Variable Moduli model for the crushed stone at a confining pressure of 100, 200 and 300 kPa.

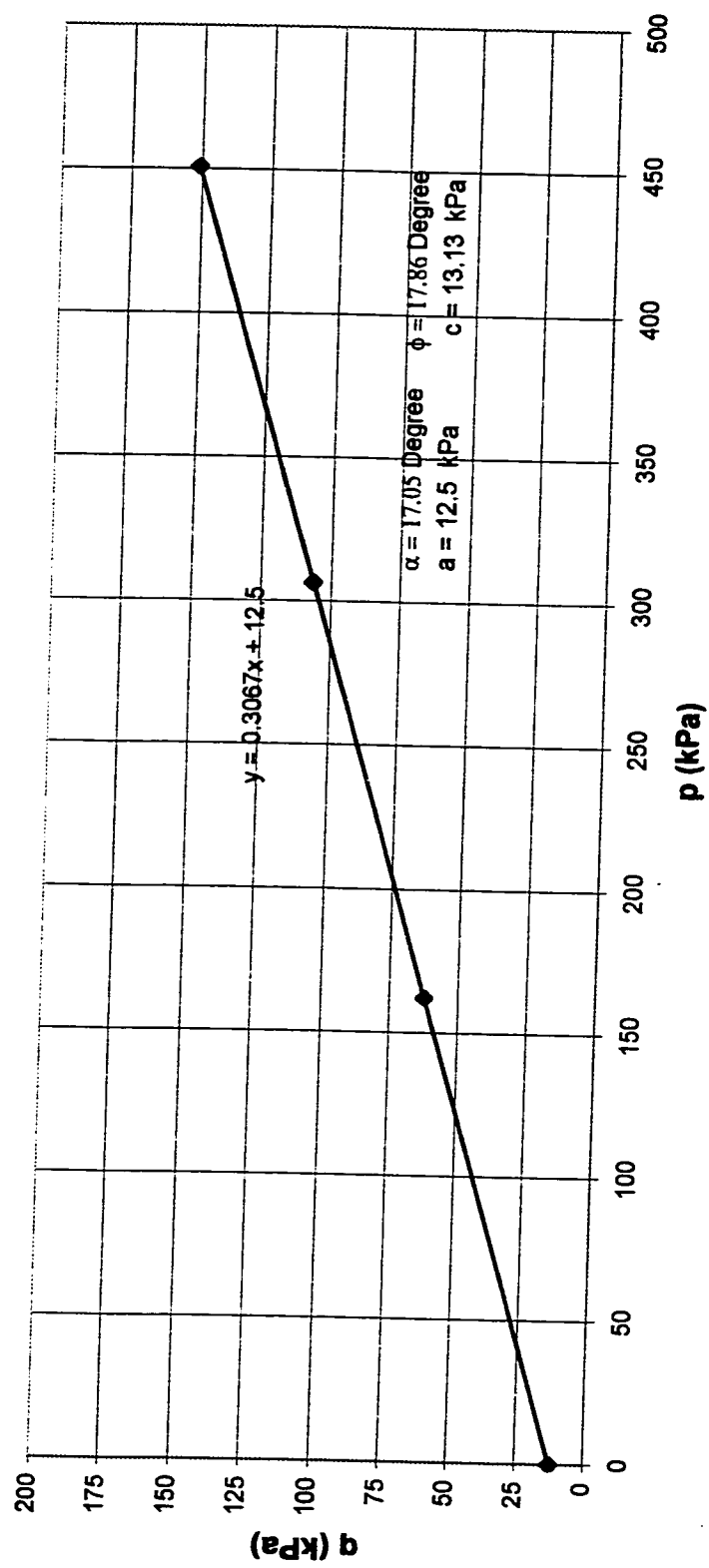


Figure 5.13: Calculated failure envelope using the Variable Moduli model for the Kanto loam.

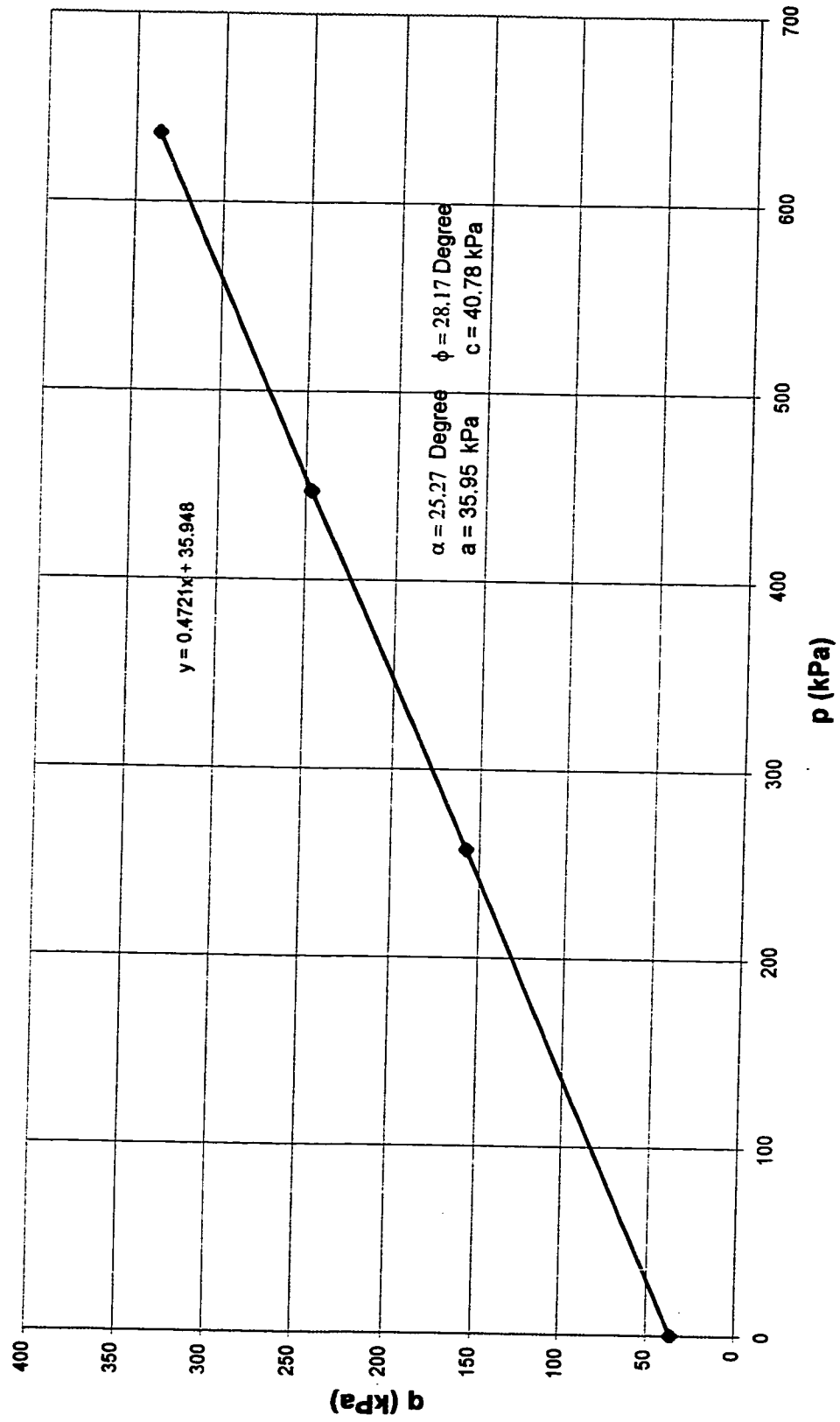


Figure 5.14: Calculated failure envelope using the Variable Moduli model for the crushed stone.

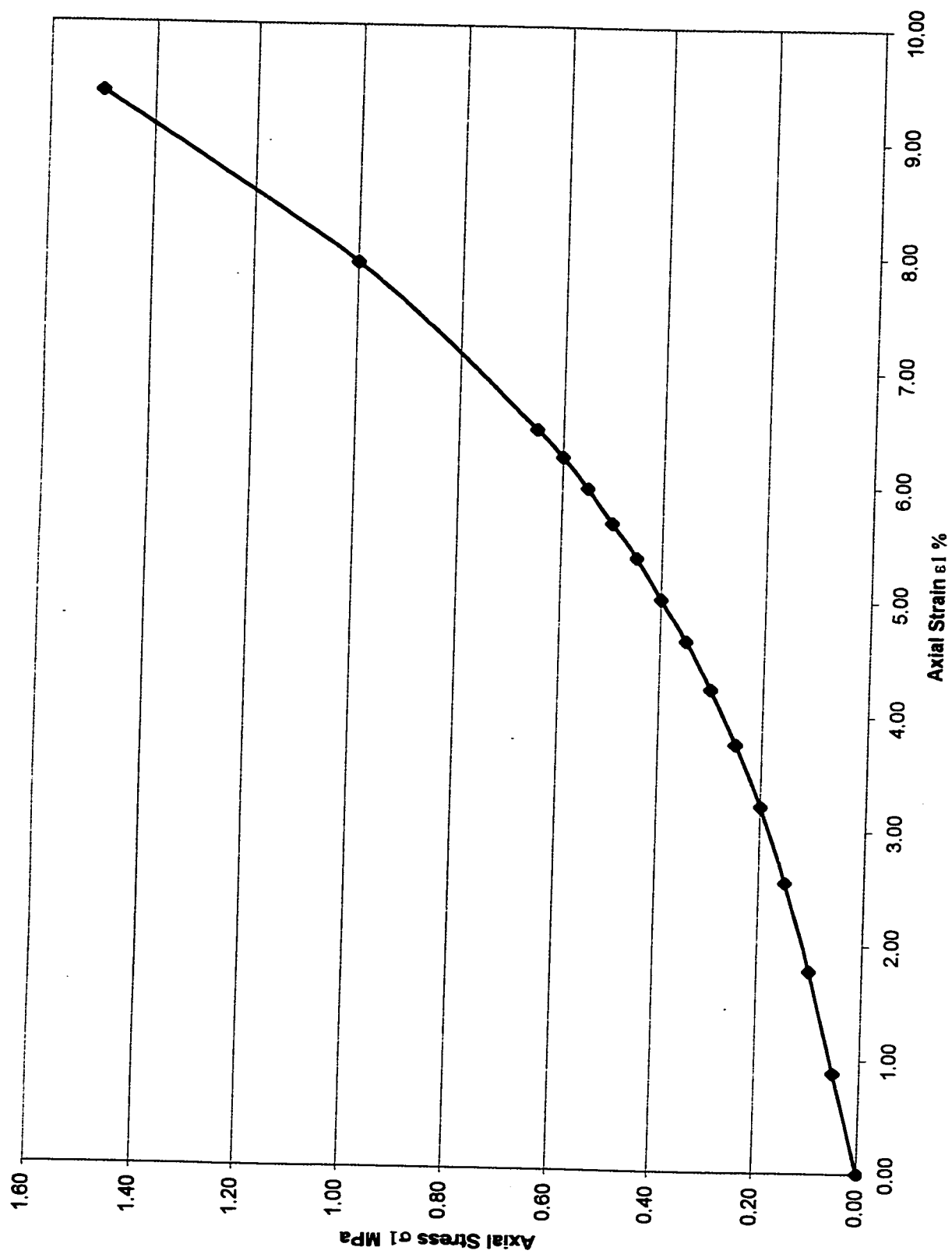


Figure 5.15: Calculated stress-strain curve using the Variable Moduli model for the simulation of an oedometer test on the Kanto loam.

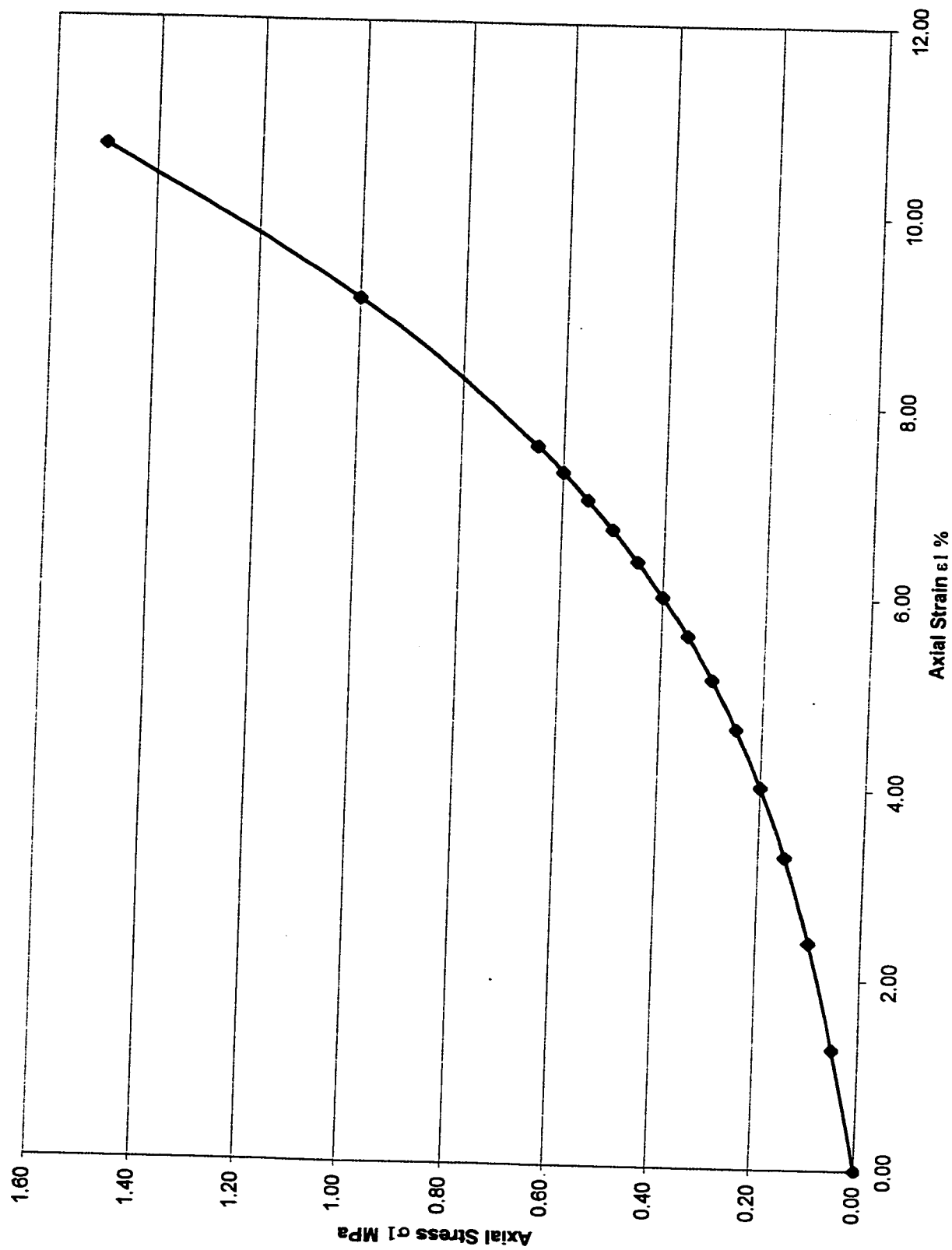


Figure 5.16: Calculated stress-strain curve using the Variable Moduli model for an oedometer test on the crushed stone.

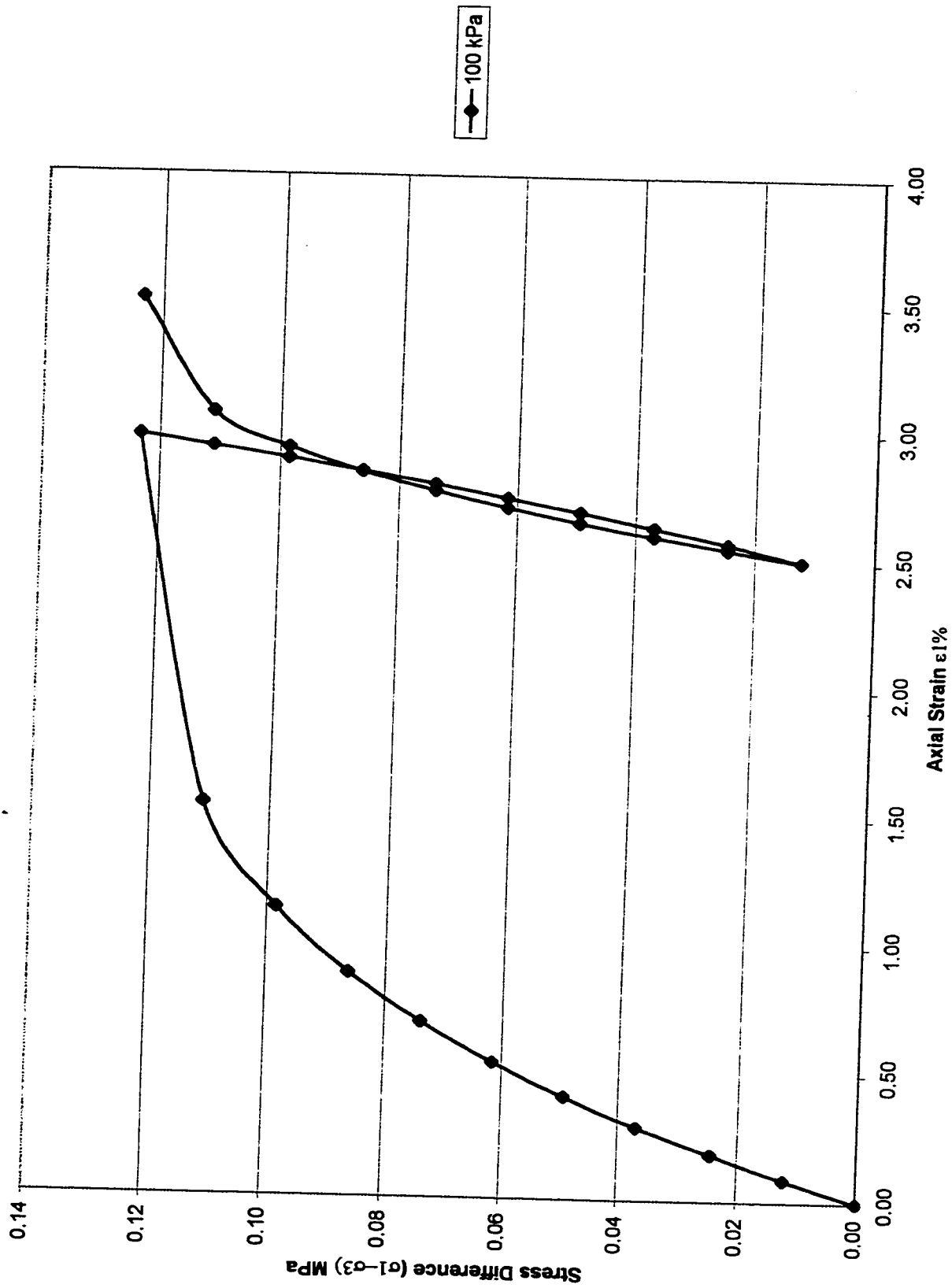


Figure 5.17: Calculated unloading/reloading stress-strain curve using the Variable Moduli model for the Kanto loam.

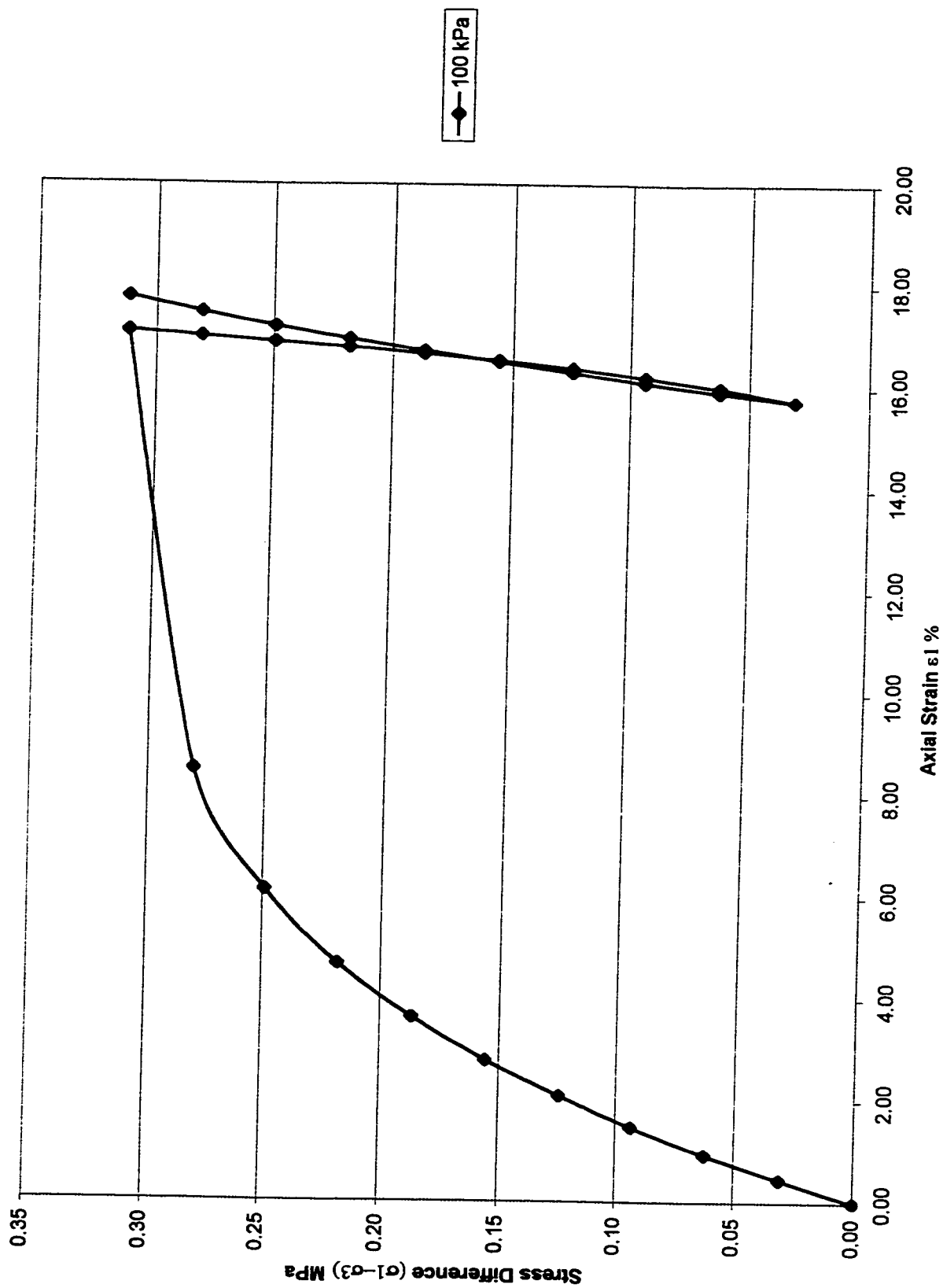


Figure 5.18: Calculated unloading/reloading stress-strain curve using the Variable Moduli model for the crushed stone.

CHAPTER 6

TWO-DIMENSIONAL FINITE ELEMENT ANALYSES

6.1 Finite element analysis of a 1500 mm FRP pipe

Mohri and Kawabata (1995) demonstrated that a numerical analysis of a large buried flexible pipe must consider the stages of construction and the compaction process. Nevertheless, the effect of interface elements in their analysis was not pointed out. Two-dimensional finite element analyses with and without interface elements will then provide some information about the usefulness of the interface elements in the analysis of buried flexible pipes. First, an attempt is made to reproduce the results of Mohri and Kawabata using the PLAXIS finite element code for soil and rock. Basic features of the software are discussed in a previous chapter. General settings, input parameters, and results of the analysis are presented herein in detail. Second, the importance of interface elements, which are located between the pipe and the soil, is demonstrated by a comparison of analysis with and without interface elements.

6.1.1 General Settings and Boundary Conditions

Plane strain model

The goal of the analysis is to predict the deflection and the stress distribution of a buried flexible pipe, considering the stages of construction and the compaction of the backfill material. For this type of analysis the pipe will be subjected to loads that act only in the x and y direction, and that the cross-sectional area including the pipe and the backfill material is constant along an indefinite length in the z direction. These characteristics allow assuming the state of strain normal to the x-y plane ϵ_z and the shear strains γ_{xz} and γ_{yz} to be zero. This state of strain, called plane strain, simplify the calculation and it is the model used for this analysis.

Elements

In the present analysis, the fifteen-node triangular element is selected in order to get the most accurate results. This type of element provides a fourth order interpolation involving twelve Gauss points.

Geometry

The total area of the problem studied has to be wide enough to ensure that the displacements at the extreme boundaries would be nearly zero. A pipeline installed in a ditch has a symmetrical geometry; therefore, it would be more convenient to base the analysis on one half of the problem. An area of thirty meters wide by fifteen meters deep is large enough to avoid any effect of the boundaries on the displacements of the pipe. In the first analysis (PipeAnalysis1), the horizontal displacements at the ground surface are almost zero at about fifteen meters from the centre of the ditch. For the purpose of accelerating the calculation, the area of the third analysis (PipeAnalysis3) is chosen to be twenty meters wide by ten meters deep. The dimensions are shown in Figures 6.2 and 6.3.

In order to simulate a staged construction project and define different layers of soil, the trench has to be divided in several horizontal layers. Based on the results of Fukuoka (1987), the compaction effect using a tamper does not extend more than about 300 mm (Mohri and Kawabata, 1995). Therefore, it is realistic to define a thickness of 300 mm for each layer that has to be backfilled including the bedding. The depth of the ditch is 3.6 meters, providing a cover of 1.8 meters over a pipe with a diameter of 1.5 meters. The stages of construction and a cross section of the geometry of the trench are shown below (Table 6.1 and Figure 6.1).

Table 6.1: Stage of construction

Stage	Stage of construction	H (m)
1	Natural Ground	15.0
2	Excavation, Kanto Loam	15.0-13.5
3	Excavation, Kanto Loam	13.5-12.45
4	Excavation, Kanto Loam	12.45-11.4
5	Bedding, Crushed Stone	11.7
6	Pipe	-
7	Backfill and Compaction, Crushed Stone	12.0
8	Backfill and Compaction, Crushed Stone	12.45
9	Backfill, Crushed Stone	12.9
10	Backfill, Crushed Stone	13.2

11	Backfill, Crushed Stone	13.5
12	Backfill, Kanto Loam	13.8
13	Backfill, Kanto Loam	14.4
14	Backfill, Kanto Loam	15.0

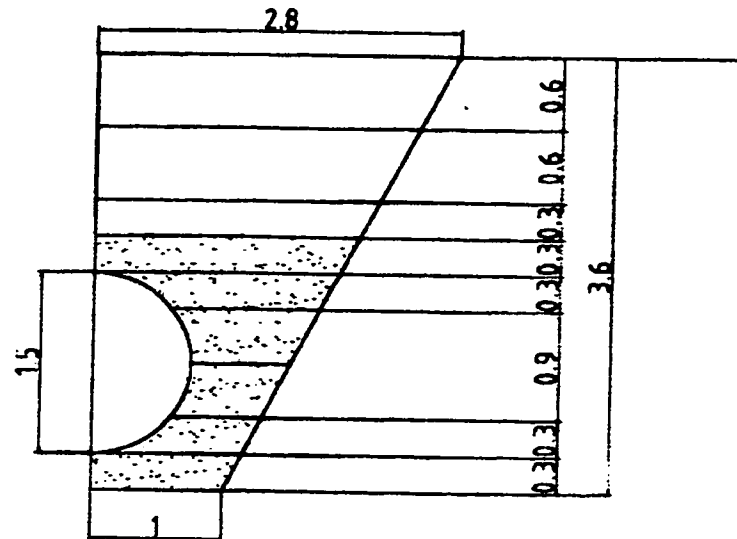


Figure 6.1: Cross section (Dimensions in meter)

Pipe

Curved beams represent the pipe in the ground. These beams are real plate in the out-of-plane direction and can therefore be used to adequately model the pipe. Five-node beam elements, with eight stress points, are used together with the fifteen-node soil elements. PLAXIS allows beam deflection due to shearing as well as bending moment. Care must be taken with the sign convention in PLAXIS. The sign convention is presented in a previous chapter.

Mesh Generation

Mesh generation in finite element software is often cumbersome. Fortunately, PLAXIS automatically creates a random mesh. An option is available to refine the mesh where stress concentration and large deformations occur. In the case of a buried pipe analysis, the mesh is refined in the trench, where most of the stress change and deformation occur.

Standard Fixities

The standard fixities include a horizontal displacement equal to zero for the extreme x-coordinates of the total geometry, a vertical and horizontal displacement equal to zero for the lowest y-coordinate of the total geometry, and a rotation equal to zero for beams that extend to the boundary of the geometry. These rules apply for most geotechnical problems, and they perfectly fit to the present problem.

Traction

Two load systems are applied in order to simulate the compaction of the backfill around the pipe. The first traction is applied at the haunch of the pipe, or 0.6 meters from the bottom of the trench, and the second at the springline of the pipe, or 1.05 meters from the bottom of the trench. The compaction is expressed in the same way as in the Mohri and Kawabata's paper. A static load of 174 kPa is applied and removed 11 times for each load systems, and they do not act simultaneously. The compaction simulation is not a usual operation with PLAXIS, and it creates some instability in the calculations. Thus, there are some limitations in the selection of parameters such as the interface reduction factor.

Mohri and Kawabata simulated the compaction in 10 layers of 0.3 meters. PLAXIS does not allow more than two load systems, then the compaction are only applied at the most critical levels. This is still realistic because in the current practice, from the city of Gatineau, they do not compact from the springline to 0.3 meters over the pipe's crown. The purpose of this method is to avoid damaging the pipe and to reduce the deformation caused by the compaction process. However, the compaction of the backfill over 0.3 meter from the pipe's crown is not simulated in the present analysis, and that has definitely an effect on the deflections of the pipe.

Displacement

As mentioned previously, Fukuoka (1995) shows that the effect of compaction by a tamper does not extend more than 0.3 meters below the level being compacted (Mohri and Kawabata, 1995). In order to reproduce the same effect in the finite element analysis, a

boundary condition for restraining the vertical displacement is set at the bottom of the ditch. It would have been more realistic to set another vertical displacement restriction on the geometry line below the second load system, but PLAXIS does not allow that. It would not be possible to activate the first traction load because prescribed displacements have priority over traction loads. Mohri and Kawabata were able to restrain the transmission of the compaction effect under every layer being compacted. This can explain some of the differences between their results and those of the present analysis.

Interface Elements

Interface elements are needed around the pipe to model the interaction between the soil and the pipe material. The roughness of the interface is modelled by choosing a suitable value for the strength reduction factor. This factor relates the friction angle and the adhesion of the interface to the friction angle and the cohesion of the soil. When using the fifteen-node soil element, five pairs of nodes and five stress points define the interface element.

An abrupt change in soil conditions may lead to high values of stress and strain. It is possible that finite element analysis using ordinary elements would not produce realistic results for stresses and strains at these locations. This phenomenon may be solved by the addition of interface elements that will enhance the flexibility of the mesh. In an analysis of buried pipeline considering compaction, an abrupt change in soil conditions occurs between the layer of soil being compacted and the side of the trench. The high values of stress and strain generated would cause the soil to fail if interface elements are not added. Thus, interface elements were added between the backfill material and the side of the trench. The best results were obtained with the interface added in the crushed stone with a reduction factor of 0.67. Input parameters for interface elements are related to the type of soil where they are located. Hence, the reduction factor is the same for all the interface elements in a specific type of soil. In this case, the reduction factor for the interface between the backfill and the side of the trench is the same as the reduction factor of 0.67 for the interface between the pipe and the backfill material. Several unsuccessful trials have been made in order to dissociate the two types of interface. They generated some instability causing the iteration procedure to fail.

Phreatic line

The water table is four meters below the soil surface and is relatively stable. It does not have an effect on the stress distribution around the pipe.

6.1.2 Input Parameters

Application of the Variable Moduli Model and Determination of the Parameters for the Hardening Soil Model

Stress-strain curves for standard drained triaxial tests have been produced using the Variable Moduli model, and the parameters given in the paper of Mohri and Kawabata. Three primary loading curves at a confining pressure of 100, 200 and 300 kPa have been generated for the Kanto Loam and the Crushed Stone type of soil, and are presented in the chapter about soil models. These curves allow finding the failure envelope, as presented in a Chapter 5, and therefore obtaining the strength parameters such as friction angle and cohesion. The friction angles for Kanto Loam and Crushed Stone are 17.86° and 28.19° respectively, and the cohesion values are 13.13 and 40.61 kPa respectively. These values seem to be abnormal; however, they provide comparable calculated deformations in the finite element analysis using the Hardening-Soil model.

The primary deviatoric loading curve at a confining pressure of 100 kPa is needed for the determination of an important parameter for the Hardening Soil Model. This parameter is the reference modulus E_{50}^{ref} , corresponding to a confining reference pressure of 100 kPa, and is defined by a slope of a segment passing by the origin and the point on the stress-strain curve where 50% of the failure value is reached. It is also called secant modulus, and it is found to be 11554 kPa for the Kanto Loam, and 5602 kPa for the Crushed Stone.

Because the friction angles of the Kanto Loam and the Crushed Stone are below 30° , the dilation angles are realistically assumed to be zero (PLAXIS, 1998). The volumetric strain versus axial strain curves from the variable moduli model show that the soil samples have only undergone compression as shown in Chapter 5. Also, the results obtained from the analysis of the stone dust, used for the interface tests, show that the magnitude of the dilation

angle is $\psi = \phi - 30^\circ$. These results support the assumption of the dilation angle of zero for the Kanto Loam and the Crushed Stone.

Unloading and reloading curves have also been produced. However, the default value of the reference Young's modulus for unloading and reloading E_{ur}^{ref} in PLAXIS, corresponding to the reference pressure of 100 kPa, is used. The default elastic unloading/reloading modulus is equal to $3E_{50}^{ref}$.

The tangent stiffness for primary oedometer loading E_{oed}^{ref} is chosen to be equal to the secant modulus in standard drained triaxial test E_{50}^{ref} . This is the default set up of PLAXIS. The values found from the Variable Moduli model are presented in the Soil Model's chapter.

The PLAXIS user manual recommends a value of $m = 0.5$ for stress-level dependency of stiffness used in the hardening soil model. All the other settings such as the Poisson's ratio for unloading-reloading, K_0 value for normal consolidation (based on Jacky's formula, $1 - \sin\phi$), and the failure ratio R_f are set to the default value. A summary of the input parameters is presented in Table 6.2 shown below.

Table 6.2: Summary of the input parameters for the Hardening Soil Model

Parameters	Kanto Loam	Crushed Stone
Unit weight, γ_{dry} (kN/m ³)	12.27	16.35
Wet unit weight, γ_{wet} (kN/m ³)	13.72	20.08
Friction angle, ϕ°	17.86	28.19
Cohesion, c (kPa)	13.13	40.61
Dilation angle, ψ°	0.0	0.0
Secant modulus E_{50}^{ref} (kPa)	11 632.08	5604.86
Primary compression modulus E_{oed}^{ref} (kPa)	11 632.08	5604.86
Elastic unloading/reloading modulus E_{ur}^{ref} (kPa)	34 896.24	16 814.4
Stress-level dependency, m	0.5	0.5

Failure ratio, R_f	0.9	0.9
Poisson's ratio for unloading/reloading, ν	0.2	0.2
$K_o (1 - \sin \phi)$	0.693	0.528

Pipe Material

The pipe used in this analysis is an FRP (fibre reinforced plastic) pipe. The characteristics of the pipe are given in the summary of the Mohri and Kawabata's paper. PLAXIS use curved beam elements based on the Mindlin's theory (Plaxis, 1998) to simulate a pipe. The input parameters for the beam elements are:

- Normal stiffness, EA
- Flexural rigidity, EI
- Equivalent thickness, d_{eq}
- Weight, W
- Poisson's ratio, ν

The weight of the beam, or the FRP pipe, is not considered in this analysis. From the user manual of PLAXIS, the beam thickness d_{eq} is calculated from $d_{eq} = \sqrt{12 \frac{EI}{EA}}$. The Poisson's ratio is set to an arbitrary value of 0.3. Table 6.3 shows the input parameters of the pipe material.

Table 6.3: Input parameters of the pipe material.

Parameters	FRP Pipe
Normal stiffness, EA (kN/m)	435 550
Flexural Rigidity, EI (kNm ² /m)	8.37
Equivalent thickness, d_{eq} (m)	0.0152
Weight, W (kN/m ²)	0.0
Poisson's ratio, ν	0.3

Interface strength

Interface tests have been conducted on PVC surfaces with stone dust. The results show that the friction angle of the interface is about 30% less than the friction angle of the soil itself. There is no proof that it would be the same for the fibre reinforced plastic surface. However, it is realistic to assume that the interface reduction factor of 0.67 for the PVC would be in the same order for the FRP.

The results of the interface experiments also show that the adhesion of the soil on the PVC surface is close to zero. The graph of the normal stress versus the maximum shear stress is a straight line starting from the origin. Although it is realistic to assume that the adhesion is zero for the FRP/crushed stone interface, it is not possible to set a reduction factor for the adhesion different than the reduction factor for the friction angle. Therefore, the adhesion has the value of the cohesion of the soil multiplied by the same reduction factor as for the friction angle.

6.1.3 Types of Analysis

Three different types of analyses have been done. The first analysis is a complete analysis including interface elements, staged construction, and compaction simulation. The second analysis did not include the interface elements in order to find out their effect on the deflection, the stress distribution around the pipe and in the pipe material. The third analysis includes only the stage construction with interface elements. These three analyses allowed the simulation and reproduction of the experimental and numerical results of Mohri and

Kawabata, as well as the evaluation of the effect of the interface elements on the numerical results.

6.1.4 Calculation

PLAXIS offers the possibility to choose different types of calculation. The most suitable type for the present analyses is the *Plastic Calculation*, because the analyses include elastic-plastic behaviour. The *Load Advancement Ultimate Level* algorithm within PLAXIS is the second option to choose. It determines the step size automatically, and terminates the calculation when the following criteria are satisfied:

- The maximum specified number of additional calculation steps has been applied.
- The total specified load has been applied.
- A collapse load has been reached. Collapse is assumed when the applied load reduces in magnitude in two successive calculation steps.

Table 6.4 below shows the calculation phases used to simulate the staged construction and the compaction process in Analysis 1 (PipeAnalysis1).

Table 6.4: Calculation phases for the analysis one.

No.	Phase	Loading input	First step	Last step
0	Initial phase	-	0	0
1	Excavation 1	Staged Construction	1	2
2	Excavation 2	Staged Construction	3	6
3	Excavation 3	Staged Construction	7	7
4	Bedding, Crushed Stone	Staged Construction	8	11
5	Pipe	Staged Construction	12	12
6	Backfill, Crushed Stone	Staged Construction	13	13
7	Compaction A	Total Multipliers	14	16
...	...	...	...	...
28	Unloading A	Total Multipliers	61	63

29	Backfill, Crushed Stone	Staged Construction	64	64
30	Compaction B	Total Multipliers	65	68
...	...	...	...	...
51	Unloading B	Total Multipliers	152	158
52	Backfill, Crushed Stone	Staged Construction	159	159
53	Backfill, Crushed Stone	Staged Construction	160	160
54	Backfill, Crushed Stone	Staged Construction	161	162
55	Backfill, Kanto Loam	Staged Construction	163	163
56	Backfill, Kanto Loam	Staged Construction	164	164
57	Backfill, Kanto Loam	Staged Construction	165	166

6.1.5 Results

The compaction in PLAXIS is simulated by the residual strain resulting from the unloading/reloading process. Figure 6.4 shows the deformations of the ground caused by the construction stages. The solid lines indicate the original mesh and the red lines correspond to the deformed state. These deformations are significant and cannot be neglected in an analysis of a buried flexible pipe.

Pipe Deflection

The results of the Analysis 1 and 3 shown in Figure 6.5 follow exactly the same trend as the numerical results of Mohri and Kawabata. The pipe elongated because of the compaction of the side fills and then it deflected because of the earth pressure over the pipe's crown. The elongation is defined as an increase in the vertical or horizontal diameter of the pipe and the deflection is defined as a decrease in vertical or horizontal diameter. The elongations are considered positive and the deflections are considered negative. In Analysis 1, the elongation reached a peak of 17.9-mm when the backfill was at the crown, and then decreased to 12.9 mm when the backfill reached the ground surface. In the case of the horizontal diameter, it deflected to a final value of -14 mm. It reached a peak of -18.7 mm when the backfill was at 1.43 m from the bed, and then started to increase because of the overburden pressure. The FRP pipe, in the field experiment of Mohri and Kawabata,

elongated vertically for about 12.5 mm, and deflected horizontally for about -10.5 mm at the completion of the backfill.

The behaviour of the pipe in Analysis 3, without the simulation of the compaction, can be summarized as follow. There was elongation of the vertical diameter followed by a deflection caused by the earth pressure on the pipe's crown. However, the overall values of deflections and elongations in Analysis 3 are very small and the pipe enters a flattening mode. Thus, this method did not reproduce the experimental curve very well. The conclusion of Mohri and Kawabata, which confirmed that it is important to consider the compaction process in the analysis of a buried flexible pipe, is verified.

The output from the analysis without interface elements (Analysis 2) shows a vertical elongation about 30% greater than the analysis with interface elements. The vertical elongation reached a peak of 20.7 mm at the crown, and then decreased to 16.9 mm. The horizontal deflection reached a peak of -22.2 mm, and then decreased to -18.35 mm. The final horizontal deflection was again about 30% higher than that of Analysis 1. As a result, the analysis without interface elements did not predict the behaviour of a buried flexible pipe adequately.

Stresses

The fibre-reinforced-plastic (FRP) pipe is represented by curved beams. From the final construction stage in Analysis 1, it is observed that the FRP pipe carries a negative bending moment of -0.196 kNm/m at the crown and -0.426 kNm/m at the invert, and a positive bending moment of 0.460 kNm/m at the springline. These results with those of normal forces and shear forces are presented graphically in Figures 6.6, 6.7, and 6.8. The highest bending moment of 0.551 kNm/m, in Analysis 1, was generated at the springline during the compaction simulation. The maximum shear force of 1.961 kN/m is located at the haunch of the pipe and was also generated during the compaction simulation. Moreover, the maximum compressive normal force of -17.8 kN/m is located at the invert, and obtained at the completion of the backfill. Figures 6.9, 6.10, and 6.11, show the total displacement, the normal stress and the shear stress in the interface between the pipe and the crushed stone.

Analysis 2 shows the effect of interface elements. Without interface elements, the maximum compressive normal force at the completion of the backfill is now at the springline, and the value is 40% higher than the analysis with interface elements. The maximum bending moment is 20% higher, and it is also obtained during the compaction process. The shear forces are distributed slightly differently because of the changes in earth pressure distribution.

The distribution of stresses in the ground in Analysis 1 shown in Figure 6.12 is typical for a flexible pipe. The normal effective stresses applied on the pipe are illustrated in Figure 6.16. The stress at the crown is relatively low (-14 kPa), and arching effect is observed. There is a concentration of stress, in the order of -28 kPa, at the springline. A stress of about -23 kPa is found at the invert of the pipe, and decreased down to -20 kPa toward the haunch of the pipe where the compaction simulation tended to reduce the normal pressure on the pipe. The maximum earth pressure was about -80 kPa at the springline during the compaction process. A comparison between the results of Analysis 1 and Analysis 2 shows that the normal effective stress in the analysis without interface elements (Analysis 2) was lower at the invert, but significantly higher everywhere else. The mean earth pressure distributions are illustrated in Figures 6.12 through 6.15. In addition, Figures 6.17 and 6.18 present the earth pressure distribution in the same way as Mohri and Kawabata. Therefore, it is easier to compare the results.

6.1.6 Discussion

In the light of the numerical results, it can be stated that the interface elements have a significant influence on the deformations of the pipe, the stresses generated in the pipe and the stresses in the ground surrounding the pipe. The final calculated deformations of the pipe in Analysis 1 are closer to the experimental curve than the results of Analysis 2. It is observed that the first step of the compaction simulation produced a smaller vertical elongation and horizontal deflection in the case without interface. This is related to the interface elements attached to the beam elements and generating tension limiting vertical elongations and horizontal deflections. On the other hand, the absence of interface elements generated an increase of earth pressure at the pipe's haunch during the second step of the

compaction simulation. This may explain the higher vertical elongation and horizontal deflection in Analysis 2. The pipe's deformations in both analysis are approximately the same when backfilling above the springline.

The arching effect described by Marston does not take into consideration the effect of the friction of the backfill material on the pipe's surface. Marston relates the arching effect to the relative movement of the backfill directly above the pipe with respect to the soil on both sides of the trench. The rigidity of the pipe and the density of the backfill have a considerable effect on the arching. For instance, a loose soil placed above a pipe will settle more than the side fills. The friction between the side fills and the loose soil will reduce the total load applied on top of the pipe resulting in arching. However, the numerical analysis with interface elements generated a greater arching in the ground than the analysis without interface elements. The relative movements of the interface elements on the side of the pipe may have relaxed the stresses at the pipe's crown. It is difficult to draw a conclusion from this result because the variations of the soil density were not taken into account throughout the study.

The state of stresses in the FRP pipe shown in Figures 6.6 through 6.8 indicate that the pipe was subjected to negative bending moments at the invert and at the crown, and to compressive normal stresses. The higher values obtained in the case without interface elements (Analysis 2) were caused by the soil elements directly connected to the structural elements. Interface elements allow the soil elements to slip, or separate, from the pipe surface when the shear stresses and the normal stresses in the interface reach a threshold. Analysis 2 does not include interface elements and then the stresses in the ground are directly transferred to the pipe increasing the normal stresses and the bending moments.

The interface elements have also an effect on the earth pressure distribution. The use of interface elements reduces the normal effective stress on the pipe by more than 50% at the springline. However, it increases the normal effective stress by 30% at the pipe's invert. These results are shown in Figure 6.16. The slip displacements of the crushed stone along the pipe during the compaction simulation may result in high residual strains below the pipe

and increase the normal effective stress. It is observed in Figures 6.9 and 6.11 that the soil moved from the side of the pipe toward the invert. Analysis 1 seems to reproduce the compaction problem of the haunch encountered in the field. Gap displacements occur at the haunch of the pipe creating relatively low normal effective stress and reducing the support of the pipe. That may also explain the high stress at the invert.

The results of Analysis 1 are comparable to those of Mohri and Kawabata (1995). Mohri and Kawabata obtained higher deformation of the pipe and stresses in the ground, but they simulated the compaction for every layer of 0.3-m from the bottom to the top of the trench. This procedure increases the vertical elongations and horizontal deflections due to construction stages. In addition, the prescribed displacement, 0.3-m below the layer of soil being compacted, caused some fluctuations in the earth pressure distribution as shown in Figure 2.30. Generally, the distributions of earth pressure are similar, but Mohri and Kawabata obtained higher results and higher extreme values in the results than those obtained in Analysis 1 and 2. Figures 6.17 and 6.18 illustrate the earth pressure distribution in the same way as shown in Figure 2.30 reproduced from the paper of Mohri and Kawabata. The vertical and horizontal earth pressures measured in the field test did not completely agree with the normal effective stress distribution obtained in the finite element analysis. The compaction of the backfill above the pipe, its relatively low height and the non-oval deformation of the FRP pipe may have caused the high vertical earth pressure of 32 kPa and the low horizontal earth pressure of 20 kPa measured by Mohri and Kawabata (1995).

6.2 Finite element analyses of a 900 mm PVC pipe

The analyses were carried out for a smooth PVC pipe, which is a type of pipe commonly used in practice for water distribution systems. The input data required for interfaces in the finite element analyses were determined from the interface experiments using the interface testing device C3DI (Fakharian and Evgin, 1996). There is no experimental data to compare the deflections. It is assumed that the finite element results are close to the reality. The first objective of these analyses is to compare the behaviour of the FRP pipe used in Mohri and Kawabata's work and a PVC pipe, which is stiffer and smaller. The second objective is to

determine the effect of the interface reduction factor on the pipe deformation and the earth pressure distribution around the pipe.

6.2.1 Type of Analysis and Input Parameters

Analyses with interface reduction factors varying from 0.5 to 1.0 were conducted. These analyses are used to evaluate the effect of the interface strength on the deformation of the pipe and on the earth pressure distribution around the pipe. The characteristics of the PVC pipe are presented in Table 6.5 (IPEX Products). Although the width of the trench is smaller for the analysis with the 900-mm PVC pipe, the depth of soil cover is still 1.8 meters in order to be able to compare the results with the FRP pipe analysis. Moreover, the compaction simulation, the soil parameters and the procedure of the analysis were the same as in the analysis based on Mohri and Kawabata's paper presented in chapter 6. Note that interface elements between the backfill and the native soil were not essential in the present analyses. Figure 6.19 illustrates the trench schematically.

Table 6.5: PVC Pipe Characteristics

Parameter	Value
Type	IPEX Centurion, Pressure Pipe
D: Nominal Diameter (mm)	900
E: Elastic Modulus (GPa)	2.76
DR: Dimension Ratio	DR25
t: Thickness (mm)	38.9
A: Area (mm ²)	38.9
I: Moment of inertia (mm ⁴ /mm)	4905.32
EI: Flexural Rigidity (kNm ² /m)	13.53
EA: Axial Stiffness (kN/m)	107 364.0

The soil properties are highly dependent on the state of compaction (Boscardin et al., 1990). In the analyses presented in this thesis, the soil properties should vary throughout the compaction simulation. However, varying the soil properties during the calculation process brought instability in the calculation. It is then impossible to include this important

characteristic of the soils into the present analyses. This drawback might explain the over estimation of the deflection compared to the experimental values of Mohri and Kawabata.

6.2.2 Results

Deflections of the pipe

The deformations of the PVC pipe, in the range of 5-mm, were much smaller than the deformations of the FRP pipe. This observation is quite normal considering that the PVC pipe is 40% smaller and that its flexural rigidity, EI , is about 40% higher. On the other hand, the behaviour of both the FRP and the PVC pipes under the applied loads was similar, as shown in Figures 6.20 and 6.5. Vertical elongation occurred during the compaction simulation and then decreased when backfilling from the pipe's crown up to the ground surface. Figure 6.20 also illustrates that the influence of the interface reduction factor on the deformation of the pipe was not significant. In fact, the vertical elongations and the horizontal deflections were almost the same in the four analyses using interface reduction factors of 0.5, 0.67, 0.85 and 1.0. The deformations in the interface with a reduction factor of 0.5 and 1.0 are presented in Figures 6.21 and 6.22. As it is expected, the total deformations are higher in the case of the weaker interface. The interface is particularly deformed at the haunch of the pipe

Stress distribution in the ground

It appears that the interface reduction factors have an effect on the normal effective stress distribution. The normal effective stress at the pipe's crown was about -14 kPa at the completion of the backfill with an interface reduction factor of 0.5. This stress was increased to -19 kPa with an interface reduction factor of 1.0. A similar behaviour is observed at the invert of the pipe. The normal effective stress varied from -11 kPa to -16 kPa as illustrated in Figures 6.23 and 6.24. There is a slight increase of the normal effective stress at the springline, which is about -40 kPa, but not as significant as the differences observed at the crown and at the invert. The mean effective stress distributions are illustrated in Figures 6.25 and 6.26. Figures 6.27 through 6.30 show the variation of shear and normal effective stresses in the interface caused by the variations in interface strength. Again, the analyses with higher interface reduction factors produced higher stresses.

6.2.3 Discussion

The interface strength has an influence on the normal effective stresses applied on the buried PVC pipe. However, the deformation of the pipe did not reflect this increase of stress. The overburden pressure may be too low to generate loads that would have a considerable effect on the deformation of the pipe related to the interface strength. Figure 6.25 shows that the relative earth movements along the top of the pipe relax the pressure at the pipe's crown. This phenomenon is increased as the interface reduction factor, or the interface strength, is decreased.

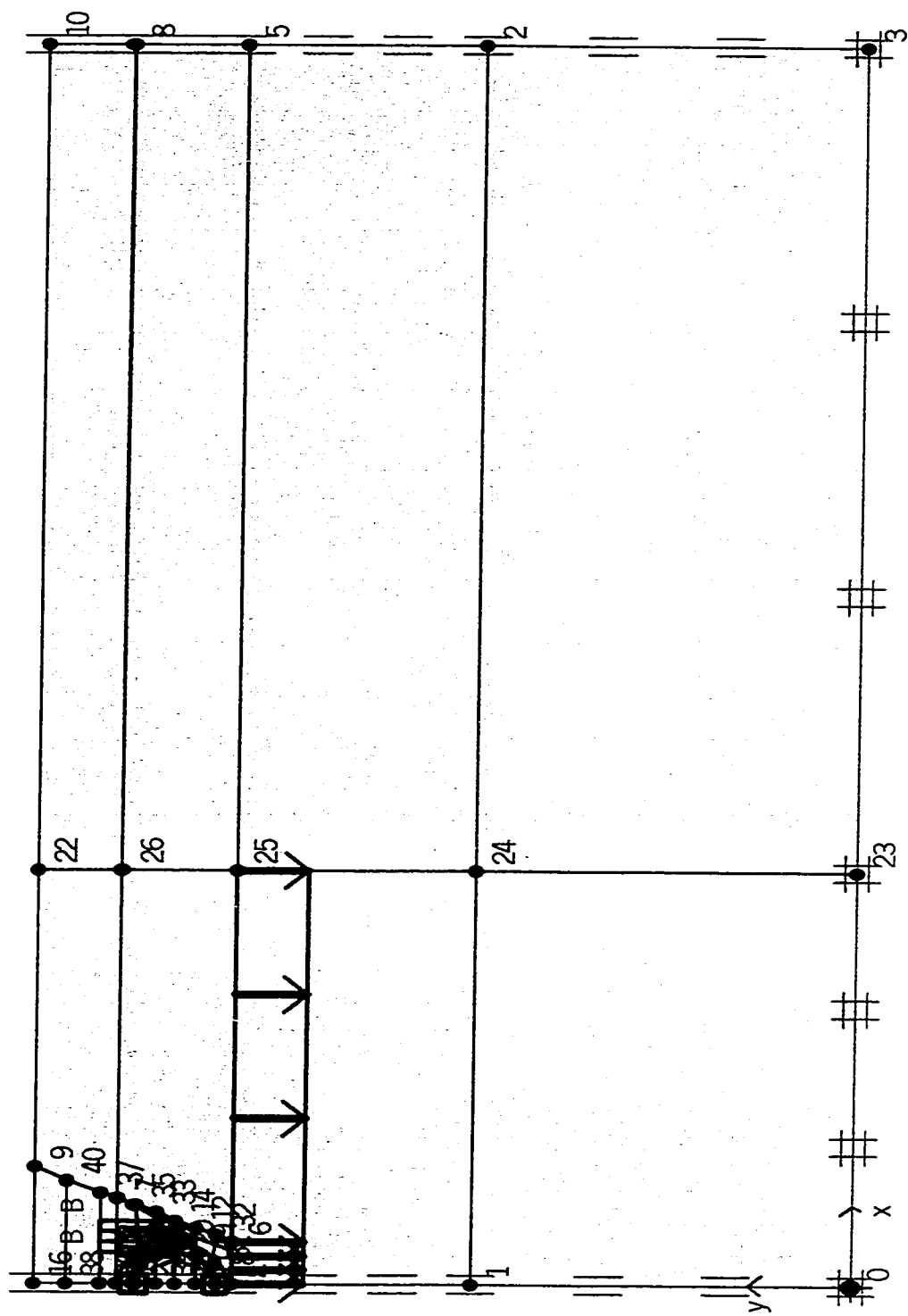


Figure 6.2: Geometry and boundary conditions

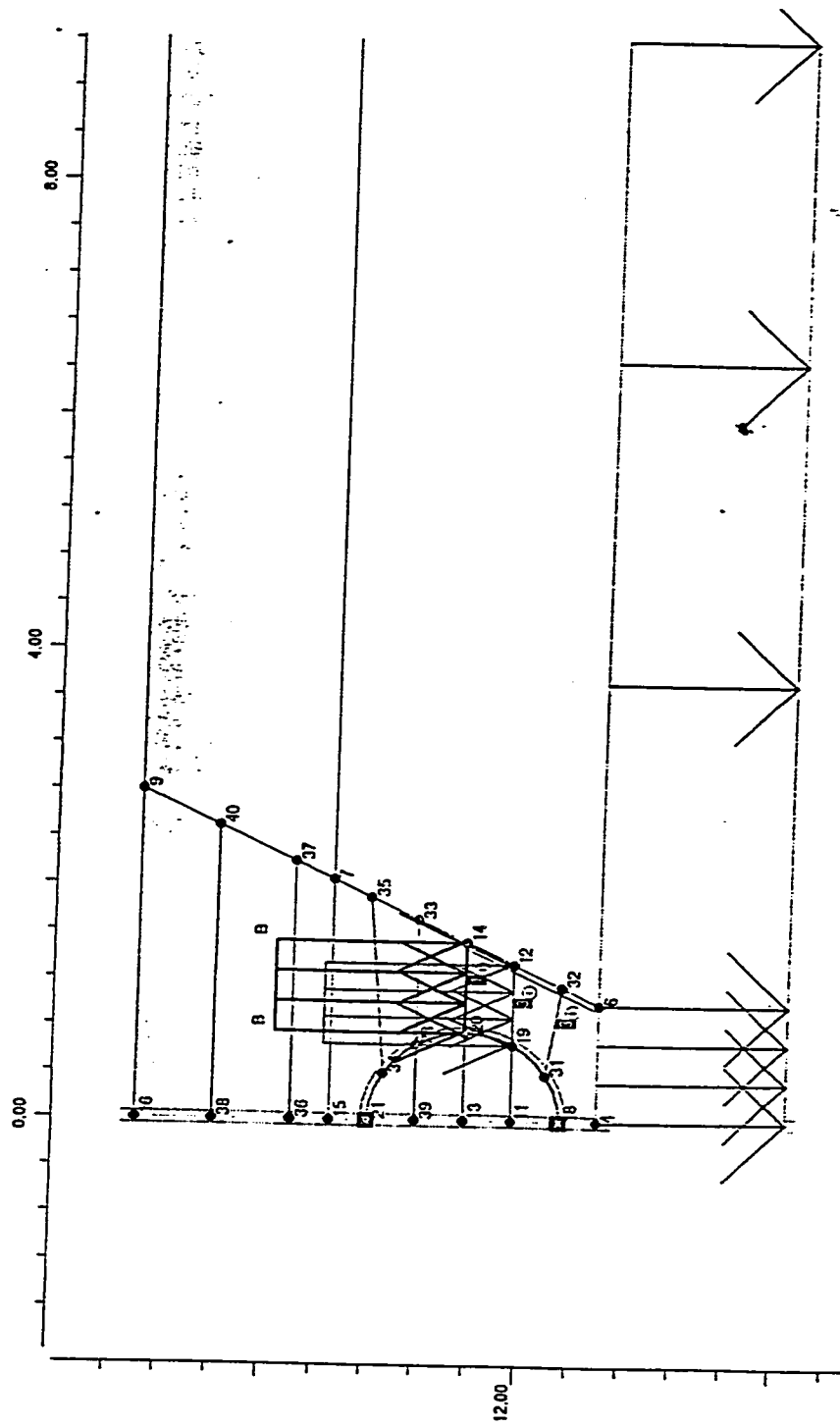


Figure 6.3: Enlargement of the geometry and boundary conditions in the analysis of the FRP pipe

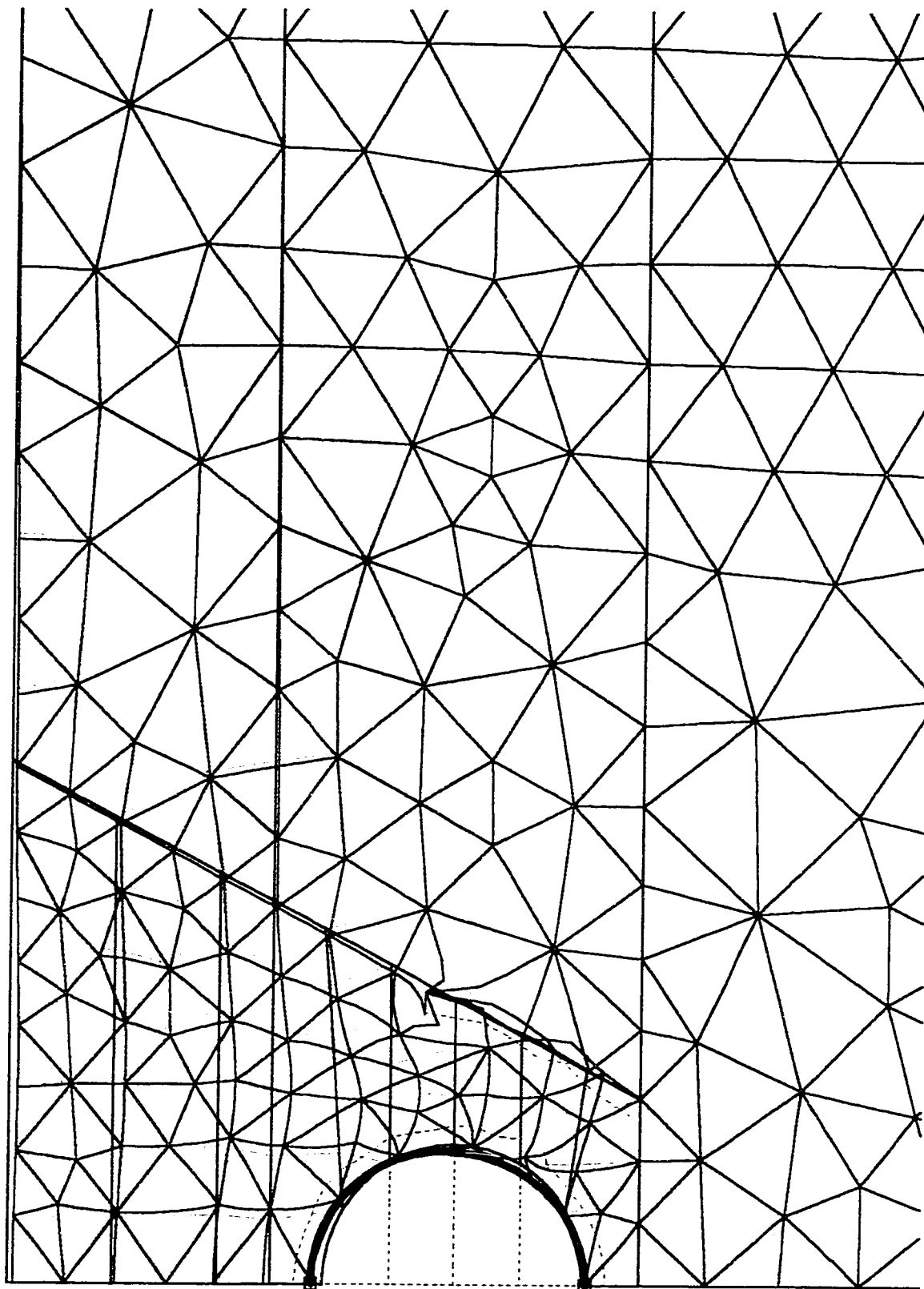


Figure 6.4: Deformed mesh in Analysis 1 (Extreme total displacement: 38.41×10^{-3} ; displacements scaled up 5 times)

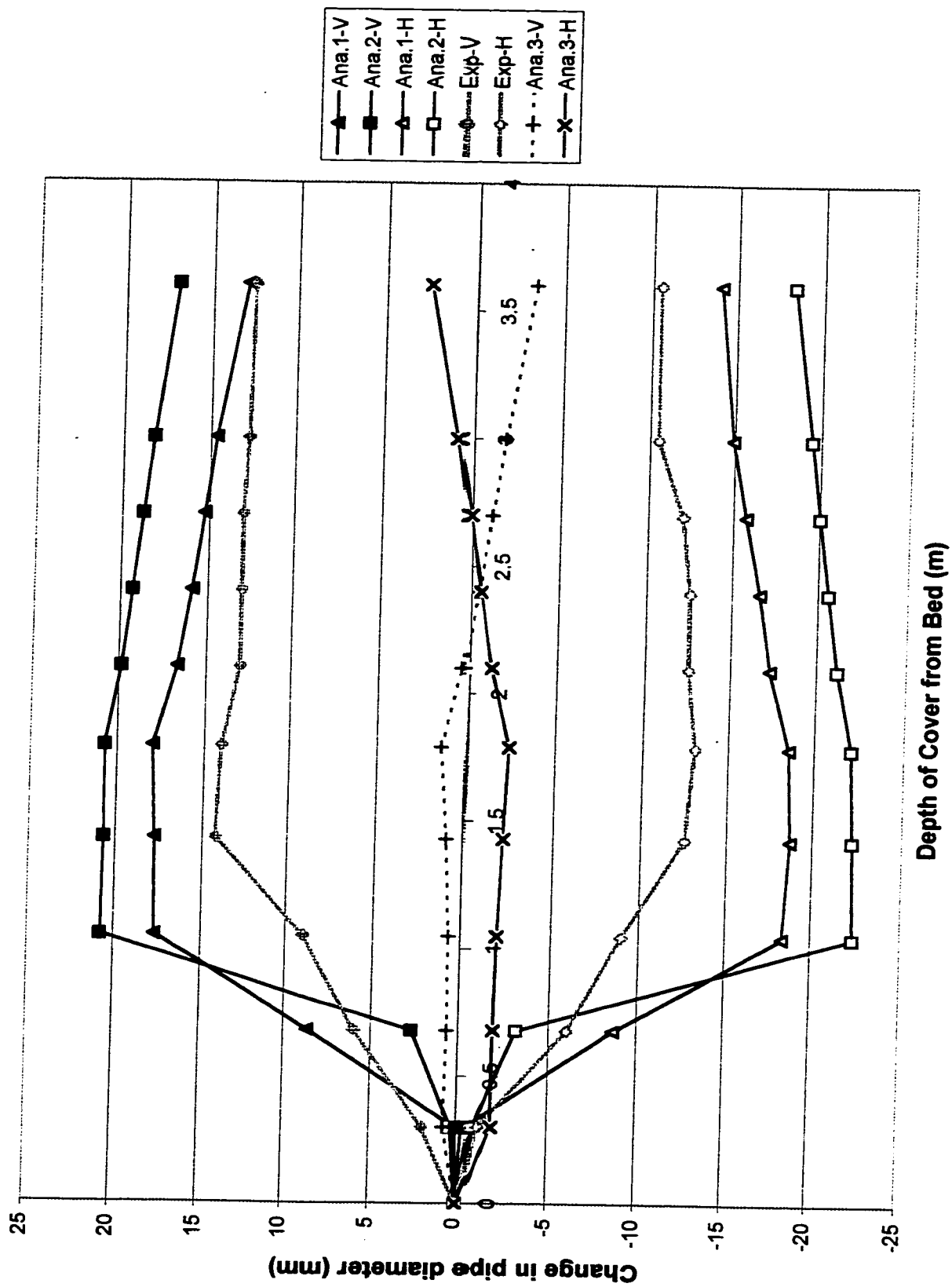


Figure 6.5: Elongation and deflection of the FRP pipe

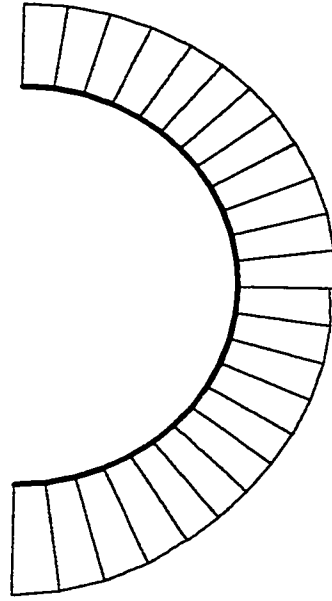


Figure 6.6: Normal forces in the FRP pipe at the final stage in Analysis 1
Extreme axial force: -17.80 kN/m (See chapter 5 for the sign convention)

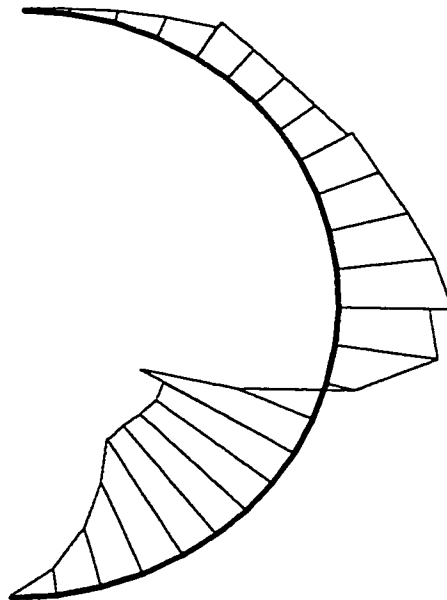


Figure 6.7: Shear forces in the FRP pipe at the final stage in Analysis 1
Extreme shear force: 1.60 kN/m (See chapter 5 for the sign convention)

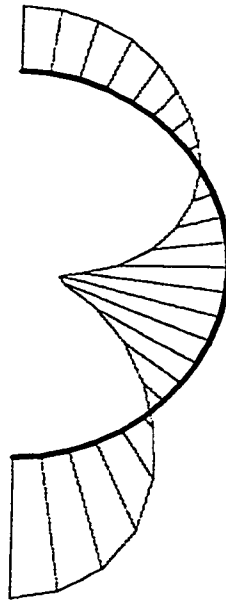


Figure 6.8: Bending moments in the FRP pipe at the final stage in Analysis 1
 Extreme bending moment: $459.81 \text{E}10^{-3} \text{ kNm/m}$ (See chapter 5 for the sign convention)



Figure 6.9: Total deformation in the interface at the final stage
 Extreme total displacement: $-24.02 \text{E}10^{-3} \text{ m}$

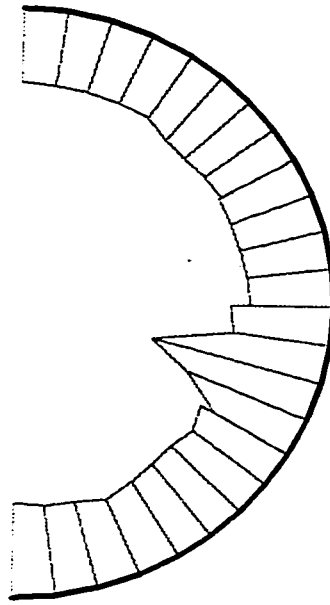


Figure 6.10: Normal stresses in the interface at the final stage
 Extreme effective normal stress: -37.40 kN/m^2 (See chapter 5 for the sign convention)

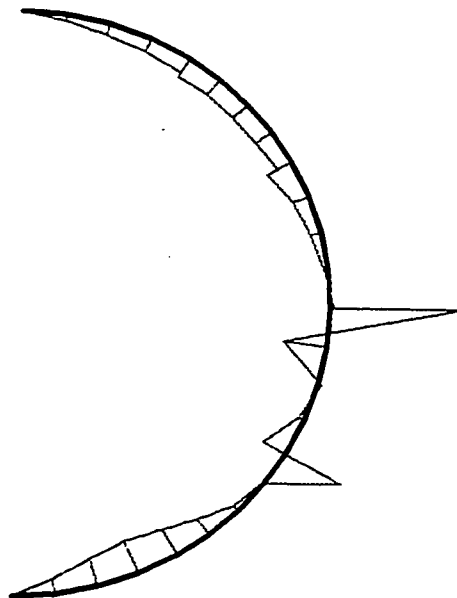


Figure 6.11: Shear stresses in the interface at the final stage
 Extreme shear stress: -17.26 kN/m^2 (See chapter 5 for the sign convention)

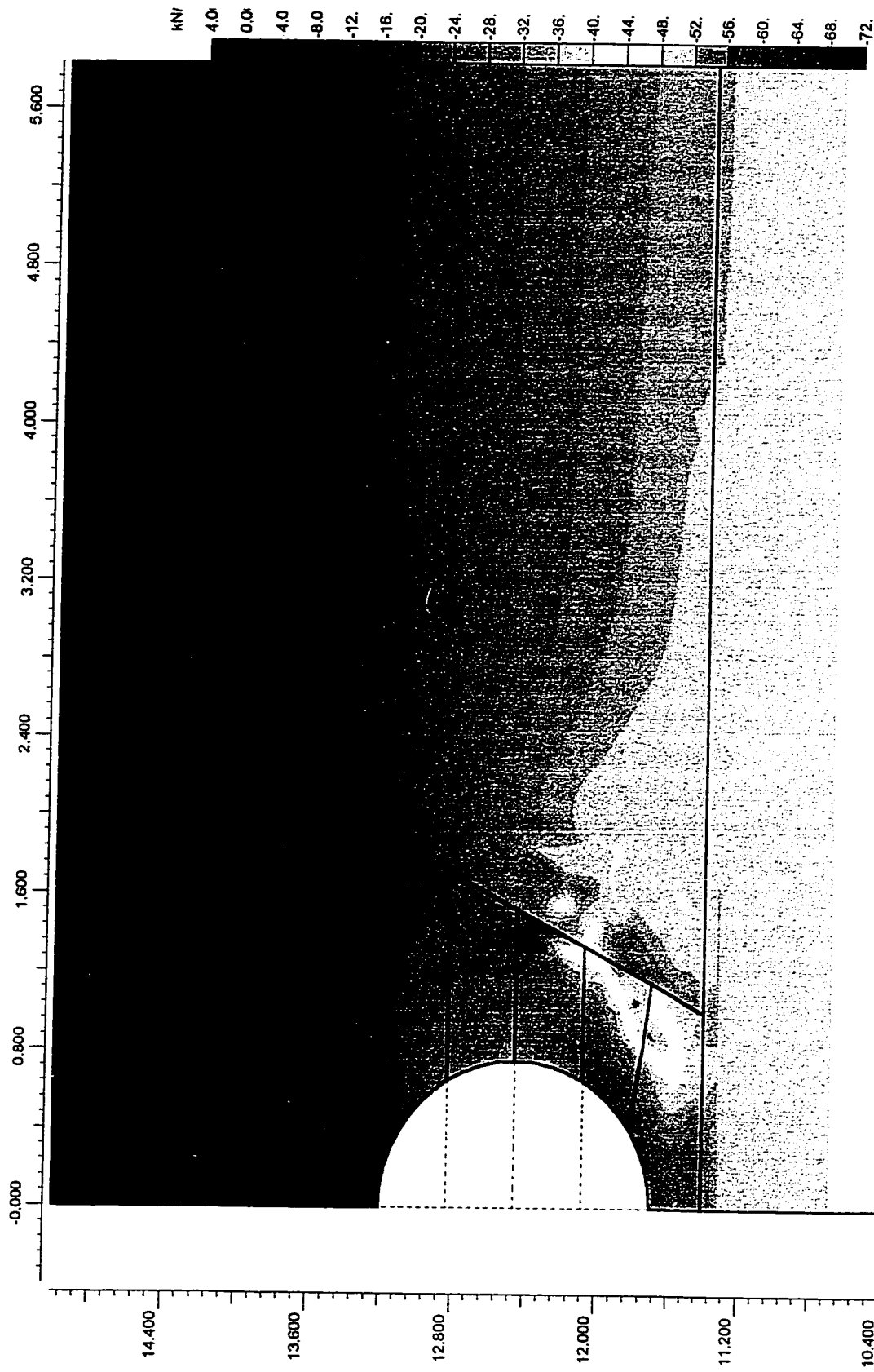


Figure 6.12: Distribution of earth pressure around the pipe at the final stage in Analysis 1

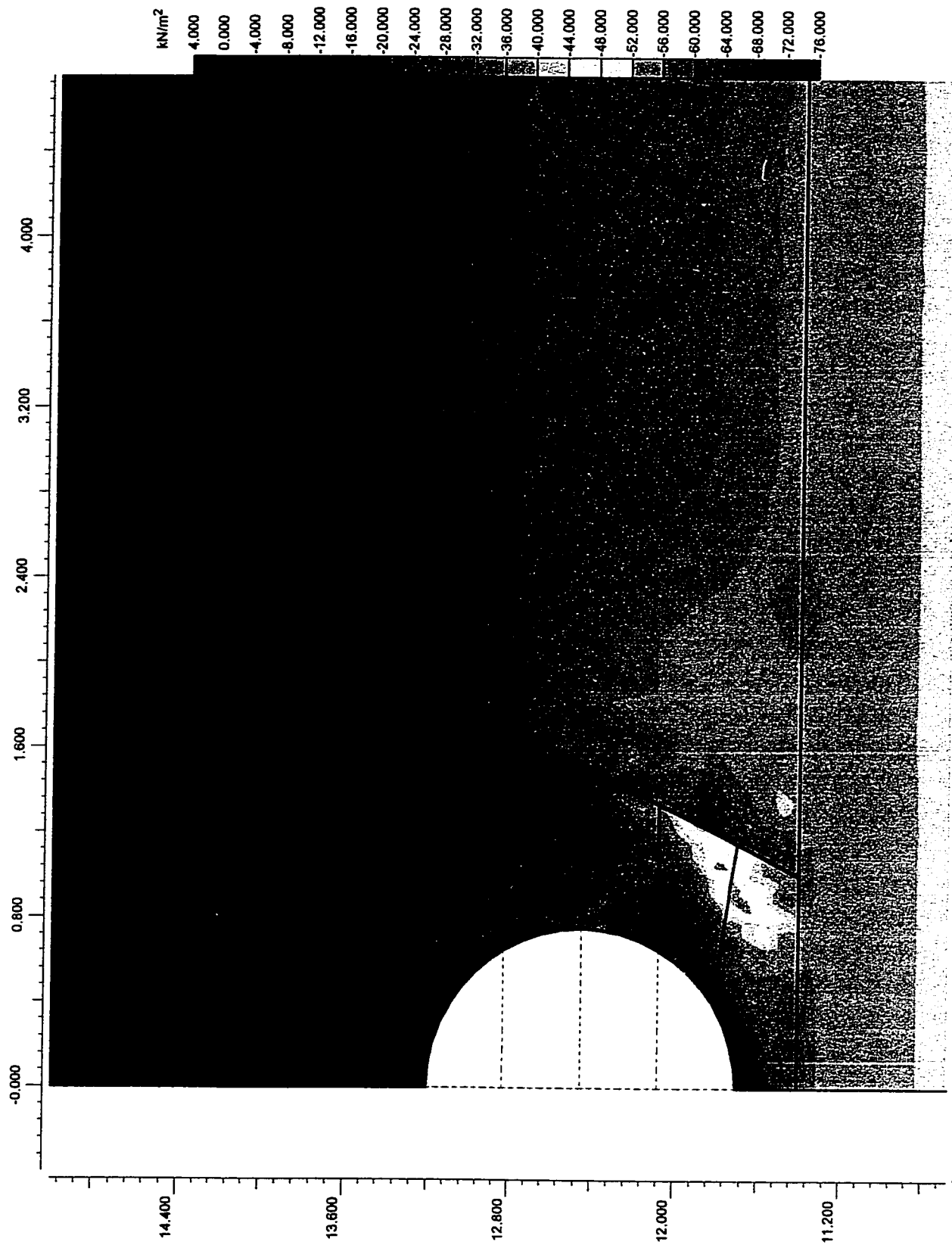


Figure 6.13: Distribution of earth pressure around the pipe at the final stage in Analysis 2

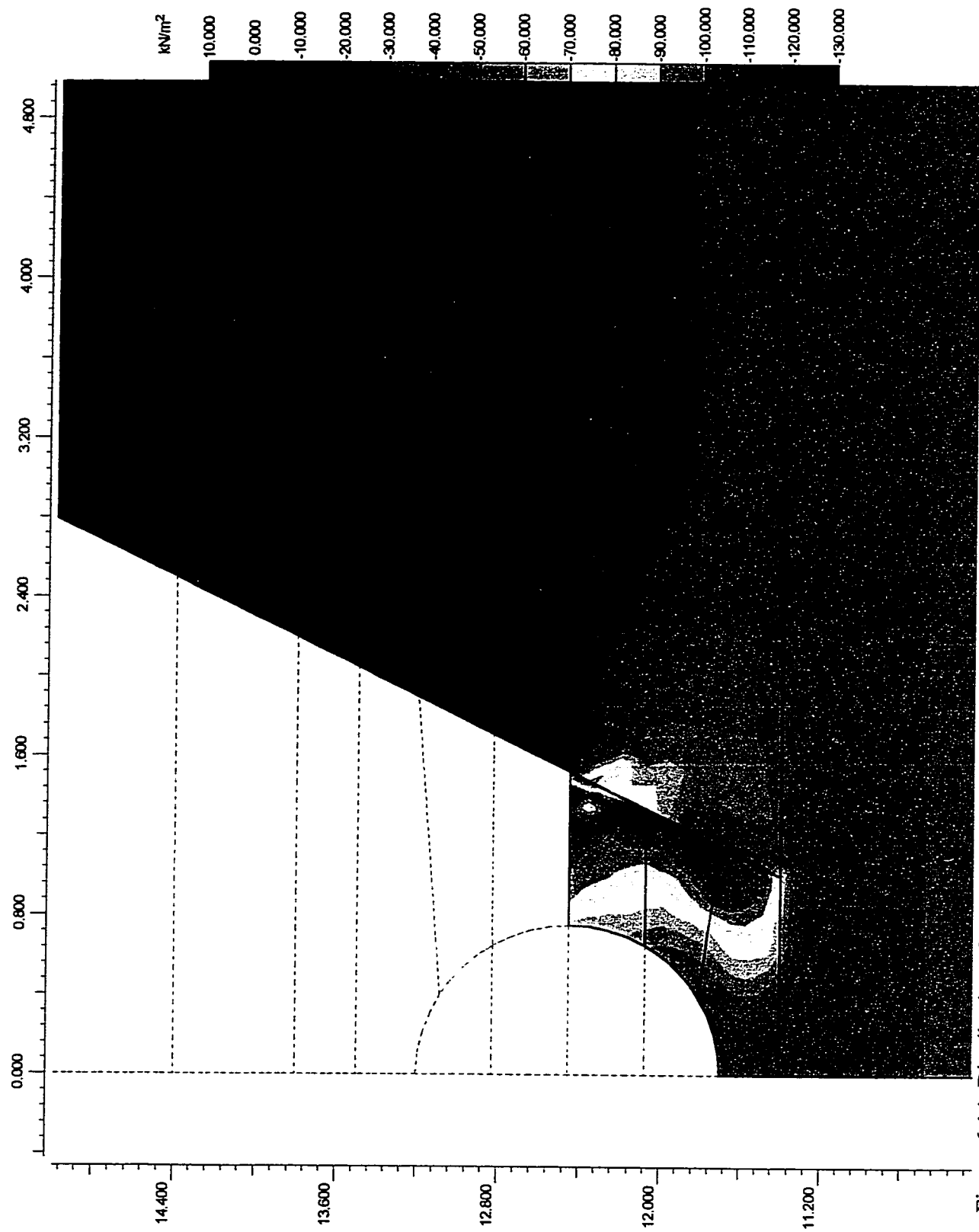


Figure 6.14: Distribution of earth pressure at the completion of the compaction simulation in Analysis 1

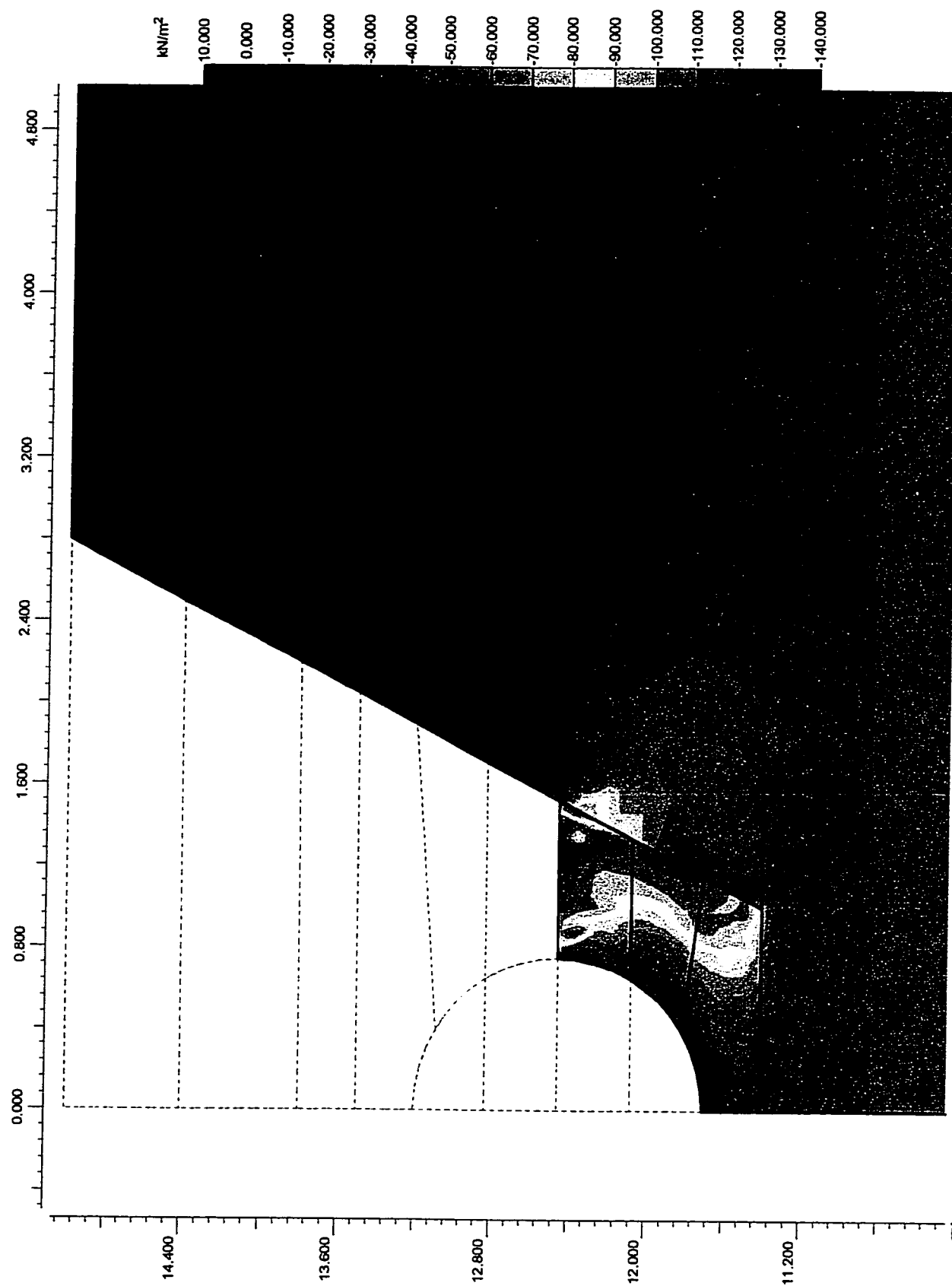


Figure 6.15: Distribution of earth pressure at the completion of the compaction simulation in Analysis 2

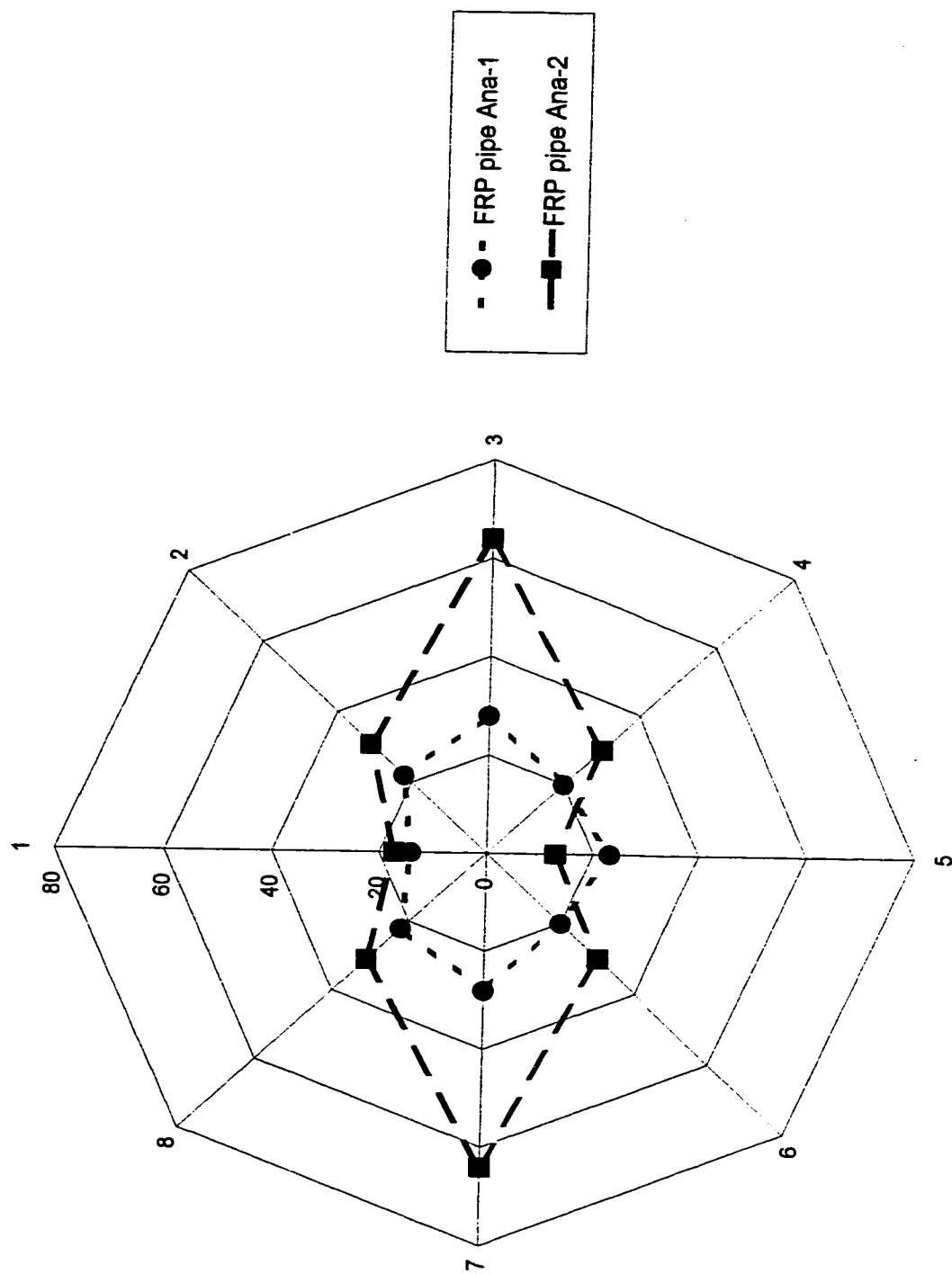


Figure 6.16: Normal stress distributions around the FRP pipe in Analysis 1 and 2 (kPa)

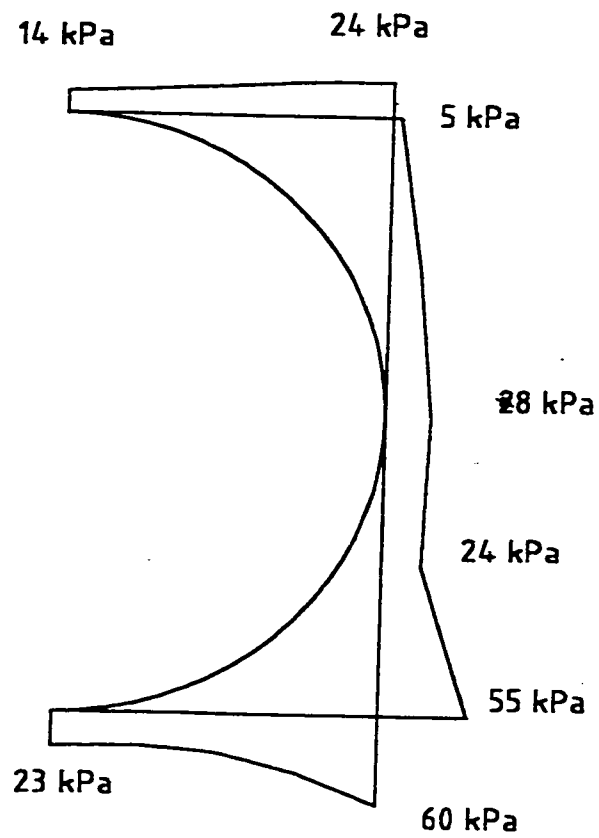


Figure 6.17: Earth pressure distribution around the FRP pipe at the completion of the backfill in Analysis 1

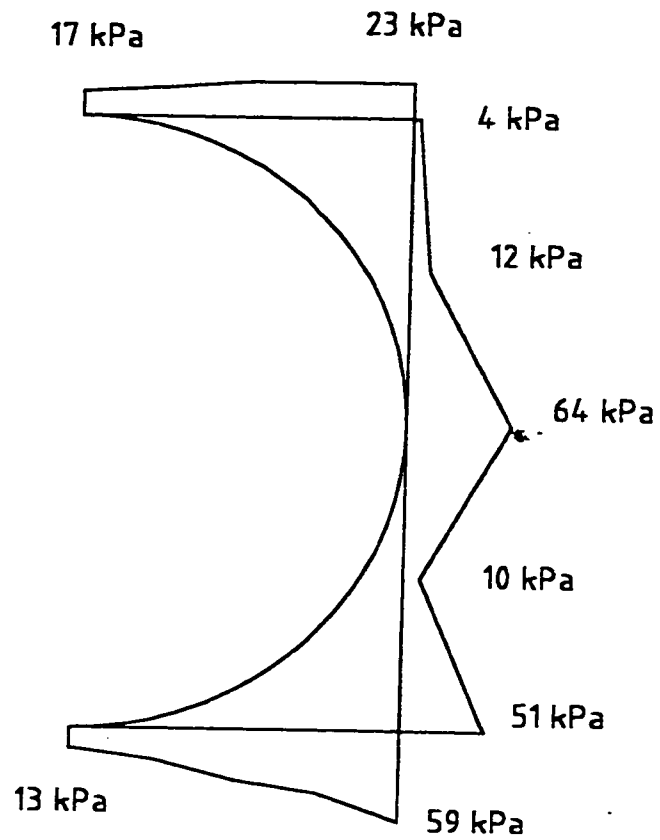


Figure 6.18: Earth pressure distribution around the FRP pipe at the completion of the backfill in Analysis 2

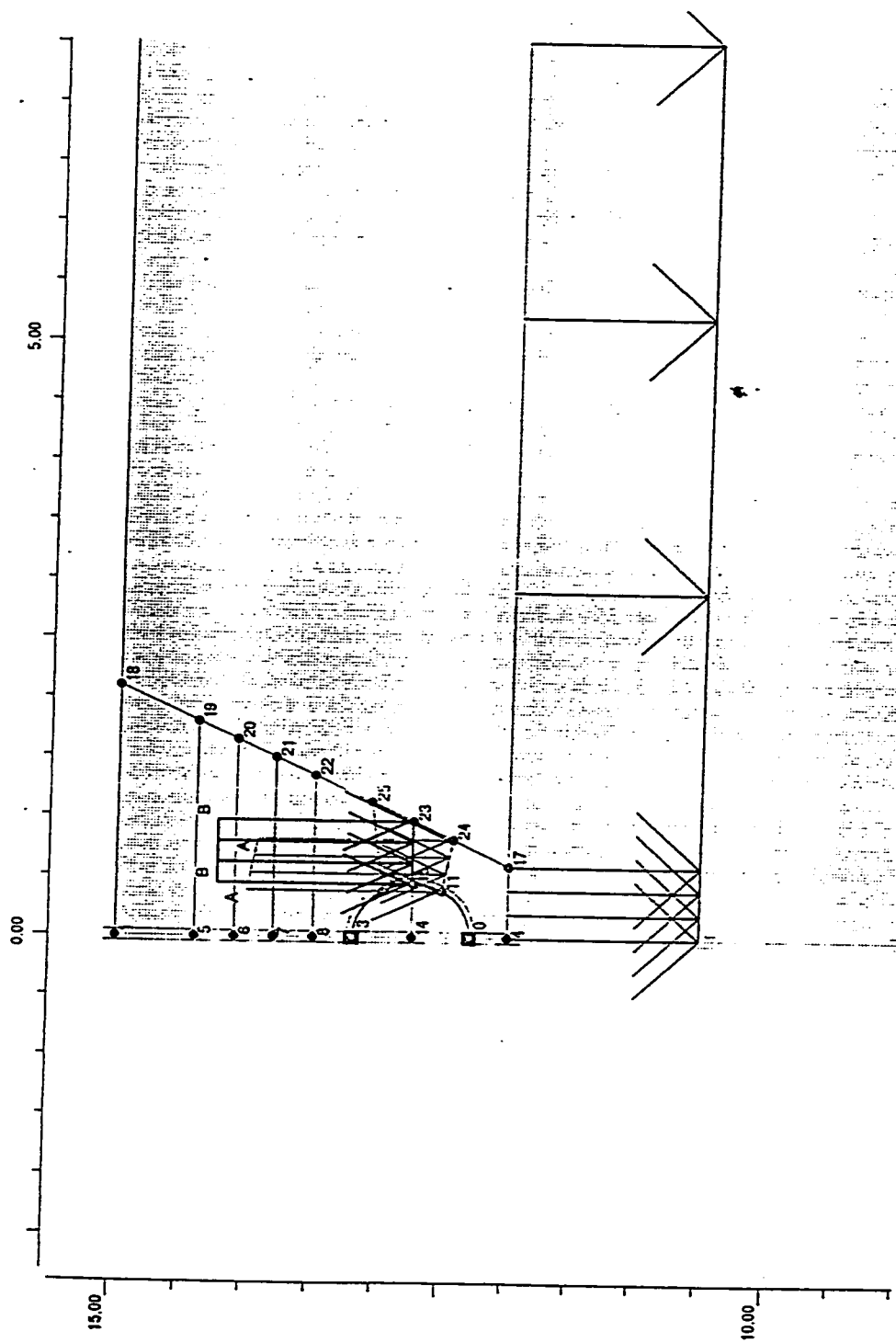


Figure 6.19: Enlargement of the geometry and boundary conditions in the analysis of the PVC pipe

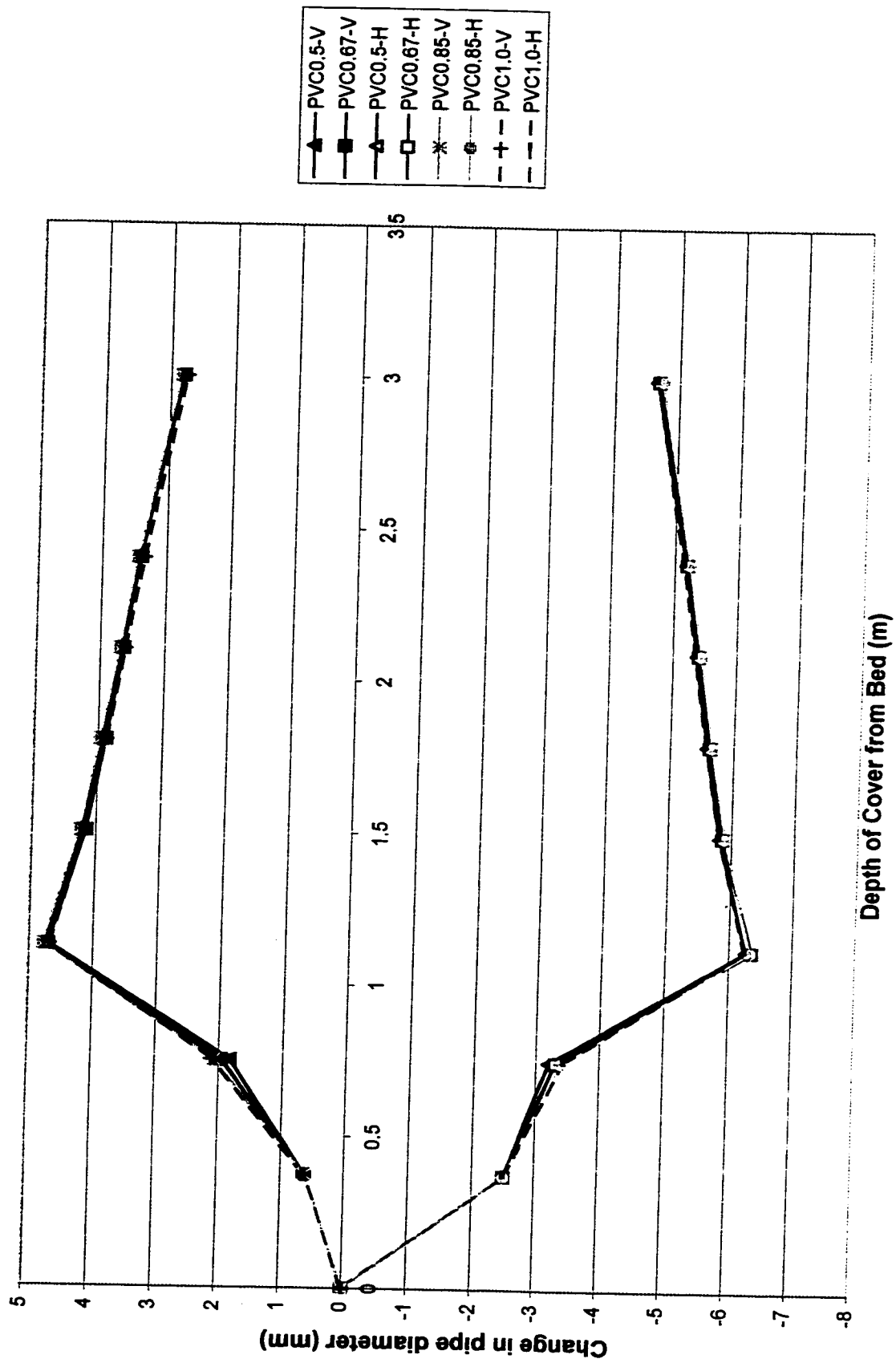


Figure 6.20: PVC pipe deformations with interface reduction factors of 0.5, 0.67, 0.85, 1.0.

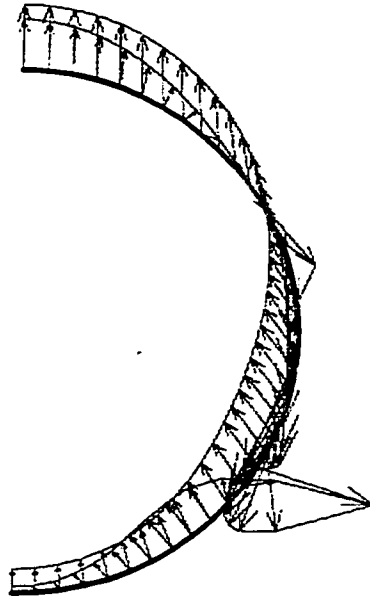


Figure 6.21: Total deformation in the interface at the completion of the backfill with a reduction factor of 1.0. Extreme total displacement: $-9.9\text{E}10^{-3}\text{m}$

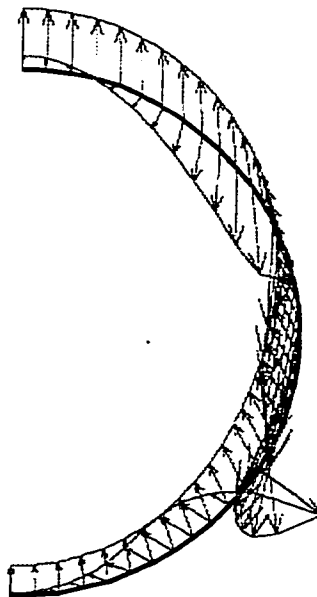


Figure 6.22: Total deformation in the interface at the completion of the backfill with a reduction factor of 0.5. Extreme total displacement: $-13.37\text{E}10^{-3}\text{m}$

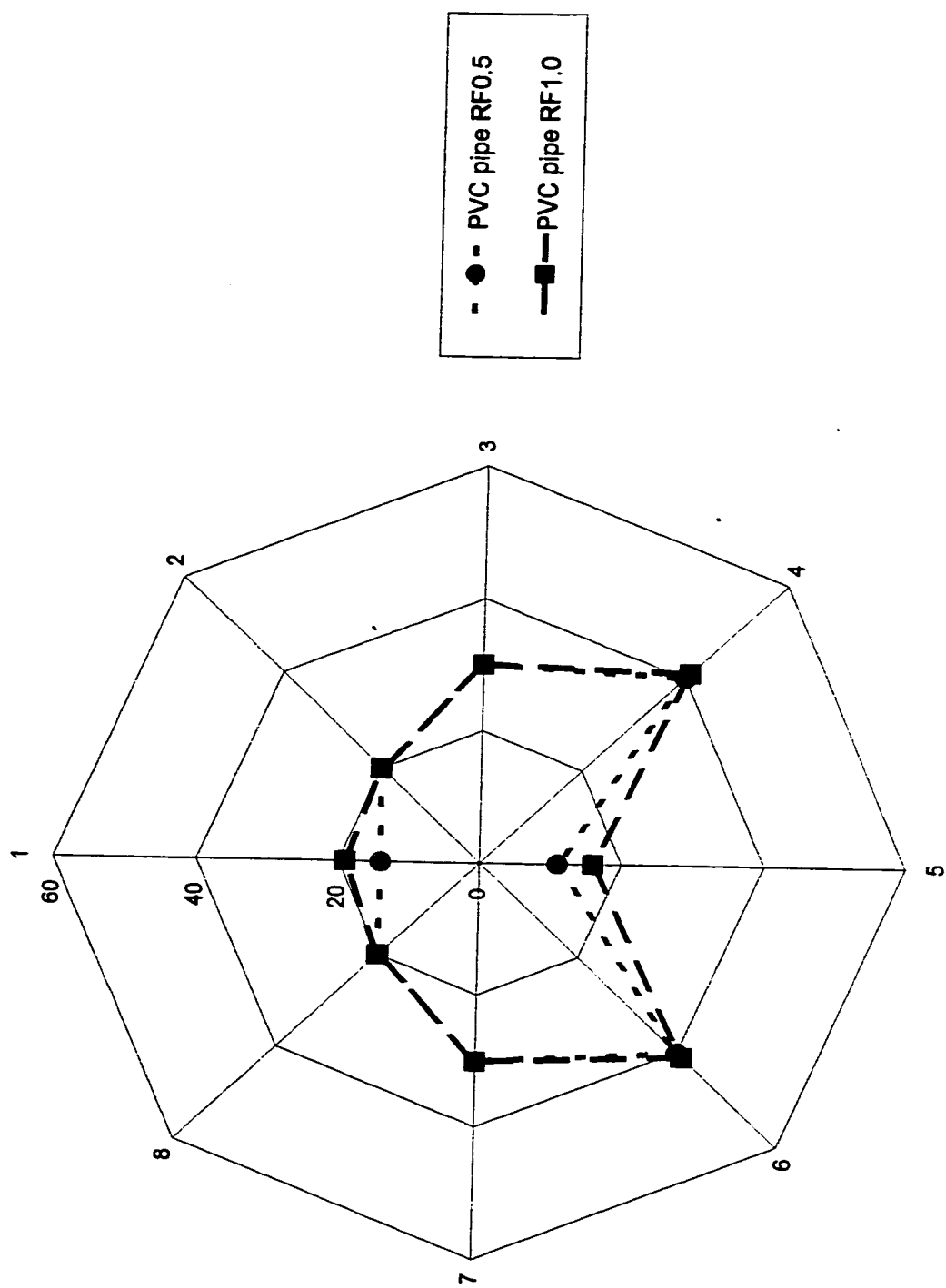


Figure 6.23: Normal effective stress distributions around the PVC pipe with an interface reduction factor of 0.5 and 1.0 (kPa)

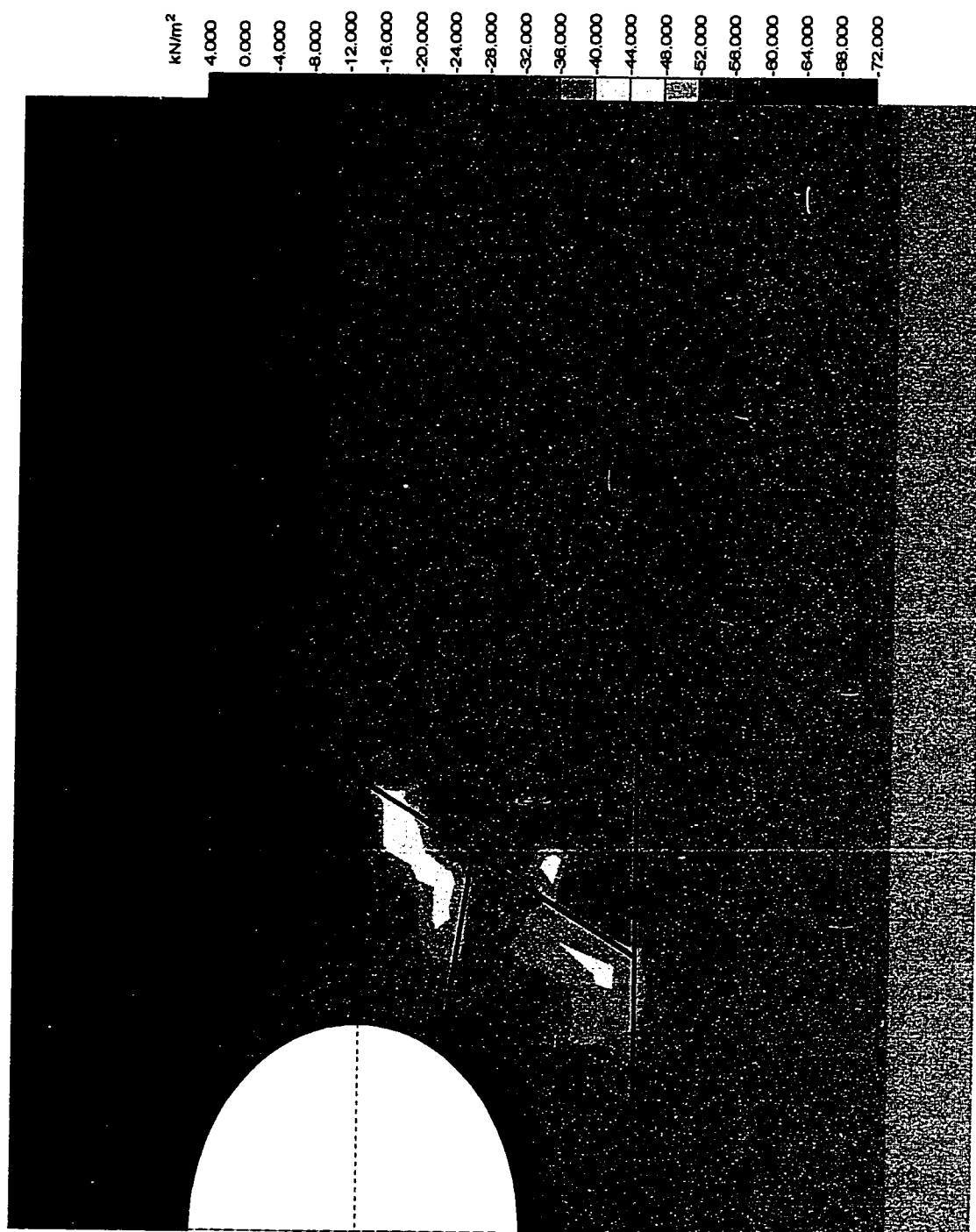


Figure 6.24: Earth pressure distribution around the PVC pipe, at the completion of the backfill, with an interface reduction factor of 0.85.

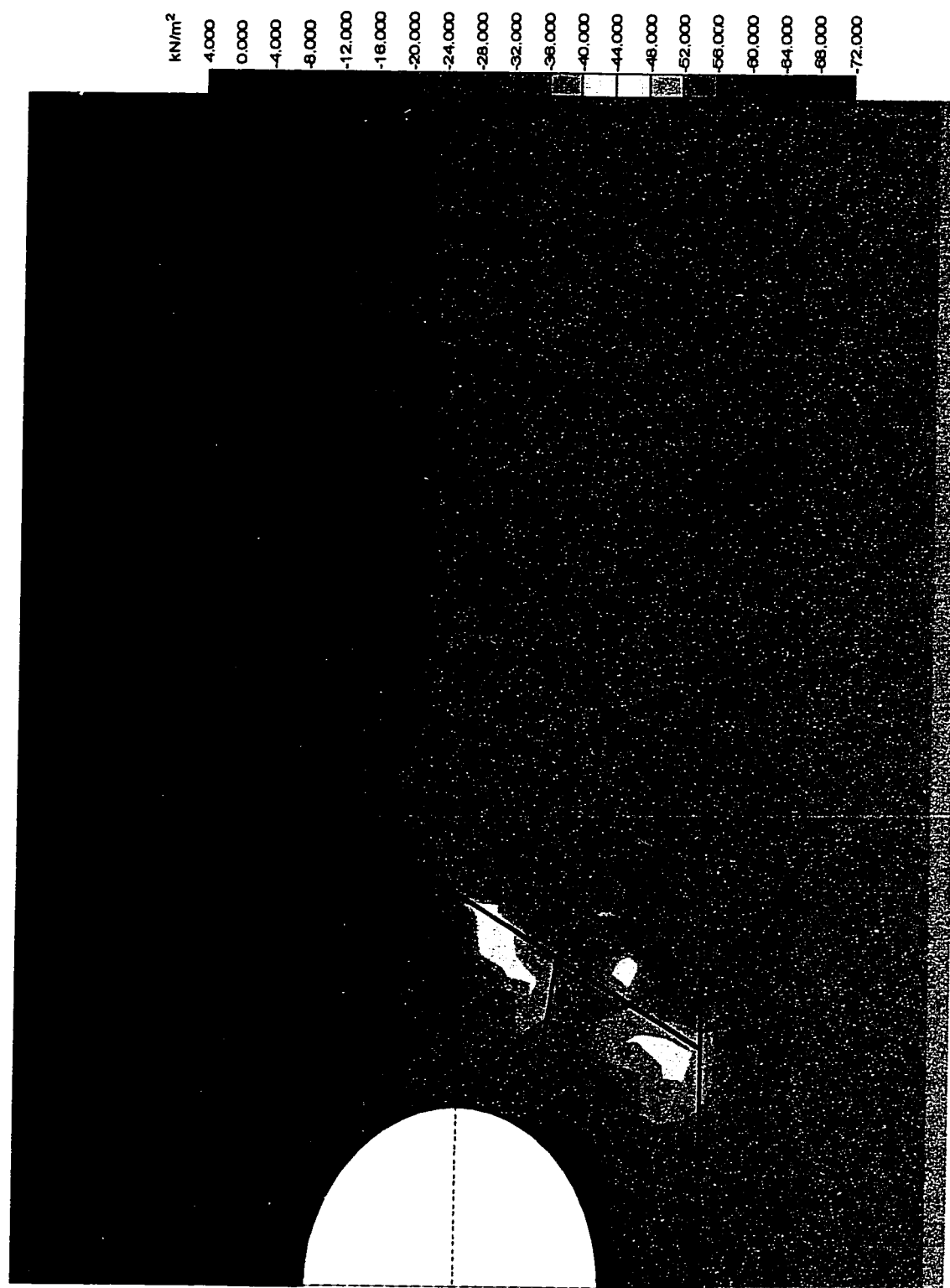


Figure 6.25: Distribution of earth pressure around the PVC pipe, at the completion of the backfill, in the analysis with an interface reduction factor of 0.5.

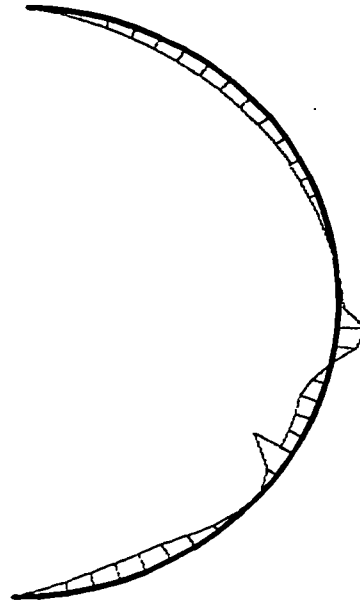


Figure 6.26: Shear stress in the interface at the completion of the backfill with a reduction factor of 1.0. Extreme shear stress: -4.66 kN/m^2 (See chapter 5 for the sign convention)

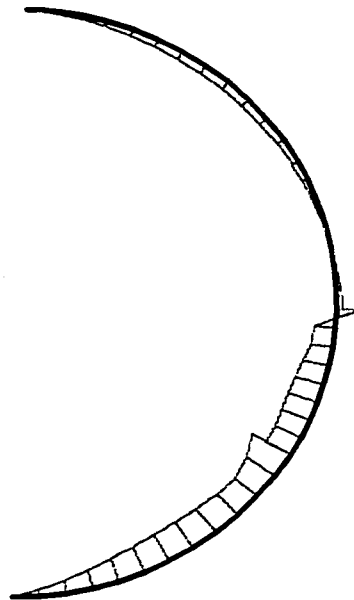


Figure 6.27: Shear stress in the interface at the completion of the backfill with a reduction factor of 0.5. Extreme shear stress: -2.06 kN/m^2 (See chapter 5 for the sign convention)

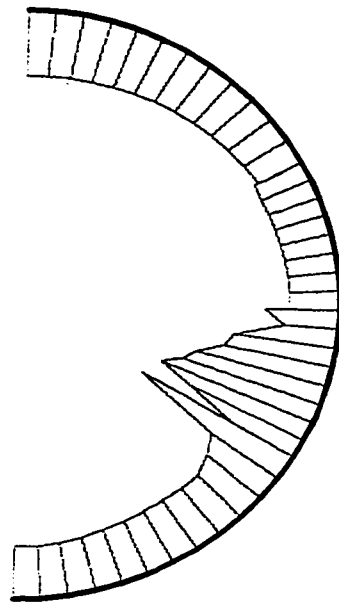


Figure 6.28: Effective normal stresses in the interface at the completion of the backfill with a reduction factor of 1.0. Mean effective normal stress: -20 kPa (See chapter.5 for the sign convention)

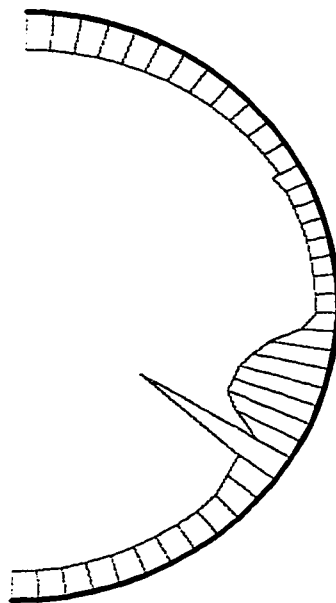


Figure 6.29: Effective normal stresses in the interface at the completion of the backfill with a reduction factor of 0.5. Mean effective stress: -15 kPa (See chapter 5 for the sign convention)

CHAPTER 7

EVALUATION OF CURRENT ANALYTICAL METHODS

The results of the theoretical deformations of the FRP pipe and the PVC pipe are presented in this section. The elastic solution by Hoëg, the modified Iowa formula of Watkins and the modified Iowa formula of Greenwood and Lang are used to predict the deformations. The results are compared with those of the finite element analyses and the field experiment of Mohri and Kawabata in the case of the FRP pipe. The prism load is used as vertical applied loads in the three methods.

7.1 The modified Iowa formula of Watkins

The parameters used in the modified Iowa formula are

- D_I : 1.0
- K : 0.096 (90°)
- r^3 : 0.75 m (FRP); 0.45 m (PVC)
- EI : 8.908 kNm²/m (FRP); 13.53 kN/m²/m (PVC)
- E' : 13 790 kPa (Howards, 1977)
- γH : 23.31 kPa (Prism load)
- $\Delta X / \Delta Y$: 0.913 (Spangler, 1941)

The modified Iowa formula of Watkins gave acceptable results for the deflection of the pipe when backfilling above the pipe's crown. However, this method did not consider the initial vertical elongation of the pipe caused by the compaction of the side fills. Table 7.1 presents the results. If the initial vertical elongations and the initial horizontal deflections of the pipes are subtracted from the total deformations, the results are comparable. In the case of the FRP pipe, the experimental results of Mohri and Kawabata give a horizontal elongation of 2.5 and a vertical deflection of -1.75, which are close to the predictions by the Watkins' formula. In the case of the PVC pipe, the adjusted FE results give a horizontal elongation of 1.672 and a vertical deflection of -1.951.

Table 7.1: Deformations, in mm, of the pipe predicted by the Watkins' Iowa formula

Type of pipe	Watkins' formula		FE Analysis		Field experiment	
	ΔX	ΔY	ΔX	ΔY	ΔX	ΔY
FRP	2.6	-2.8	-14.0	12.9	-10.5	12.5
PVC	2.3	-2.5	-4.7	2.8	-	-

7.2 The modified Iowa formula of Greenwood and Lang

The pipe-soil interaction coefficient, the Leonhardt relationship and the initial ovalization are added to the modified Iowa formula in order to provide more accurate predictions of the pipe deformations. The inclusion of the pipe-soil interaction coefficient and the Leonhardt relationship into the formula reduced the elongations and the deflections given by the standard modified Iowa formula. The results are presented in Table 7.2. Note that an arbitrary initial ovalization of 1.0% for the FRP pipe and 0.5% for the PVC pipe have been chosen. The value proposed by Greenwood and Lang (1990) does not reflect the initial ovalization obtained in the field experiment of Mohri and Kawabata (1995) and in the FE analysis with the PVC pipe.

The pipe-soil interaction coefficients are

- 1.13 for the FRP pipe
- 1.82 for the PVC pipe

The Leonhardt factors are

- 1.28 for the FRP pipe
- 1.25 for the PVC pipe

Table 7.2: Deformations, in mm, of the pipe predicted by the modified Iowa formula of Greenwood and Lang

Type of pipe	G. and L. formula		FE Analysis		Field experiment	
	ΔX	ΔY	ΔX	ΔY	ΔX	ΔY
FRP	-13.2	13.0	-14.0	12.9	-10.5	12.5
PVC	-3.4	3.3	-4.7	2.8	-	-

7.3 The elastic solution of Høeg

The elastic solution of Høeg does not take into account the initial ovalization, but it allows computing pipe deformations with or without free slippage at the interface. In addition, stresses and displacements are available at any point around the pipe. Table 7.3 presents the deformation predictions, and Figures 7.1 and 7.2 present the normal stress predictions. The table considers only the deformation of the pipes when backfilled above the crown.

The parameters used in the equations are

- p : 23.31 kPa
- k : 0.528
- R/r : 1
- ν : 0.2
- ν_c : 0.3
- M : 5605 kPa
- E_c : 28050 MPa (FRP); 2760 MPa (PVC)
- D : 1.516 m (FRP) 0.972 m (PVC)
- t : 0.0155 m (FRP) 0.0389 m (PVC)

Table 7.3: Deformations, in mm, of the pipe predicted by the elastic solution of Høeg

Pipe	Free slippage		No slippage		FE interface		FE no interface		Field exp.	
	ΔX	ΔY	ΔX	ΔY	ΔX	ΔY	ΔX	ΔY	ΔX	ΔY
FRP	4.9	-5.0	2.3	-2.4	4.7	-5.0	4.0	-3.8	2.5	-1.8
PVC	2.5	-2.6	1.1	-1.3	1.7	-2.0	-	-	-	-

7.4 Discussion

Although the modified Iowa formula of Watkins did not consider the initial ovalization, it gave a fairly good prediction of the horizontal elongation caused by the overburden pressure in the field experiment with the FRP pipe. There was a slight difference in the prediction of the vertical deflection due the non-elliptical deformation of the pipe. The standard ratio $\Delta X / \Delta Y$ from Spangler's experiments is 0.913. However, the field measurements gave a deformation ratio of 1.4. In the case of the PVC pipe, it is only possible to compare the

results with the FE analysis. The modified Iowa formula estimated a higher deformation by about 0.06% of the diameter, which is very small. Generally, the Marston-Spangler theory gave acceptable results, and the FE analysis tends to overestimate.

The additions of Greenwood and Lang (1990) made the deformations readily comparable with the field measurements and the FE analysis. This formula overestimates the deformation of the FRP pipe by about 0.17% of the diameter, which is considered good. The prediction for the PVC pipe is about 0.14% of the diameter compared to the FE analysis. Again, differences were caused by the non-elliptical deformations. The pipe-soil interaction coefficient is a function of the diameter and the flexural rigidity of the pipe. An increase of the diameter and a decrease of the flexural rigidity result in an increase of the horizontal elongation of the pipe. Regarding the Leonhardt relationship, an increase of the ratio of the embedment modulus to the native soil modulus and the ratio of the trench width to the pipe diameter result in an increase of the horizontal elongation of the pipe.

The solution of Høeg is based on the elastic theory and implies assumptions that simplify the analysis. The soil material is assumed linearly elastic, isotropic and homogenous. The loads applied on the pipe are symmetrical with respect to the horizontal axis and the vertical axis. The horizontal load is also dependent on the coefficient relating the vertical load to the horizontal load. The boundaries are assumed to be at least one pipe diameter away from the buried pipe. The mathematical solution is expressed in terms of the compressibility ratio, C , and the flexibility ratio, F . The compressibility ratio relates the compressibility of the pipe material to that of a solid soil cylinder. This ratio is very small in the current study; thus, it did not have a major effect. The flexibility ratio relates the flexibility of the pipe material to the compressibility of a solid soil cylinder. The flexibility ratio influenced the displacements and the stresses of the pipe.

The predictions of the FRP pipe deformations using the Høeg's solution with free slippage are very close to that of the FE analysis with interface elements. Therefore, this method tends to overestimate the deformation of the pipe measured in the field experiment by about 15%. The results without slippage at the interface are more representative of the field test

results. Calculated results are also about 15% lower than the predictions using the FE method without interface elements. In the case of the PVC pipe, the estimations are about 5% higher than that of the FE analysis.

As shown in Figures 8.1 and 8.2, the normal stress distribution using the Hoeg solution did not reflect the distribution using the FE analysis. The compaction simulation in the FE analysis, which considerably influenced the stress distributions, may cause this difference. Moreover, the normal stress distributions in the analysis by Hoeg and the present analysis did not reflect the results obtained in the field experiment of Mohri and Kawabata (see Figure 6.29).

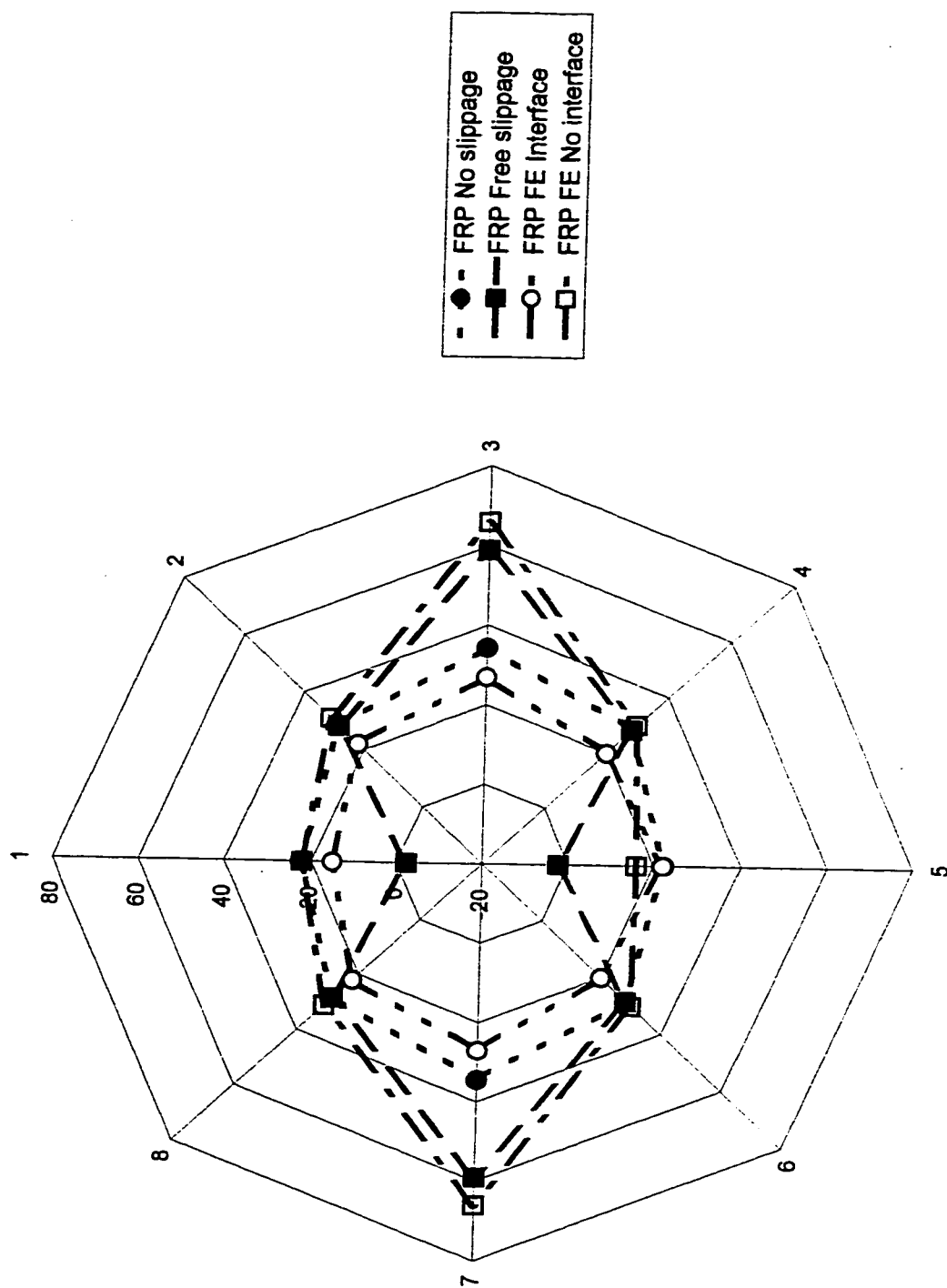


Figure 7.1: Normal stress distribution around the FRP pipe using the elastic solution of Hoeg (kPa)

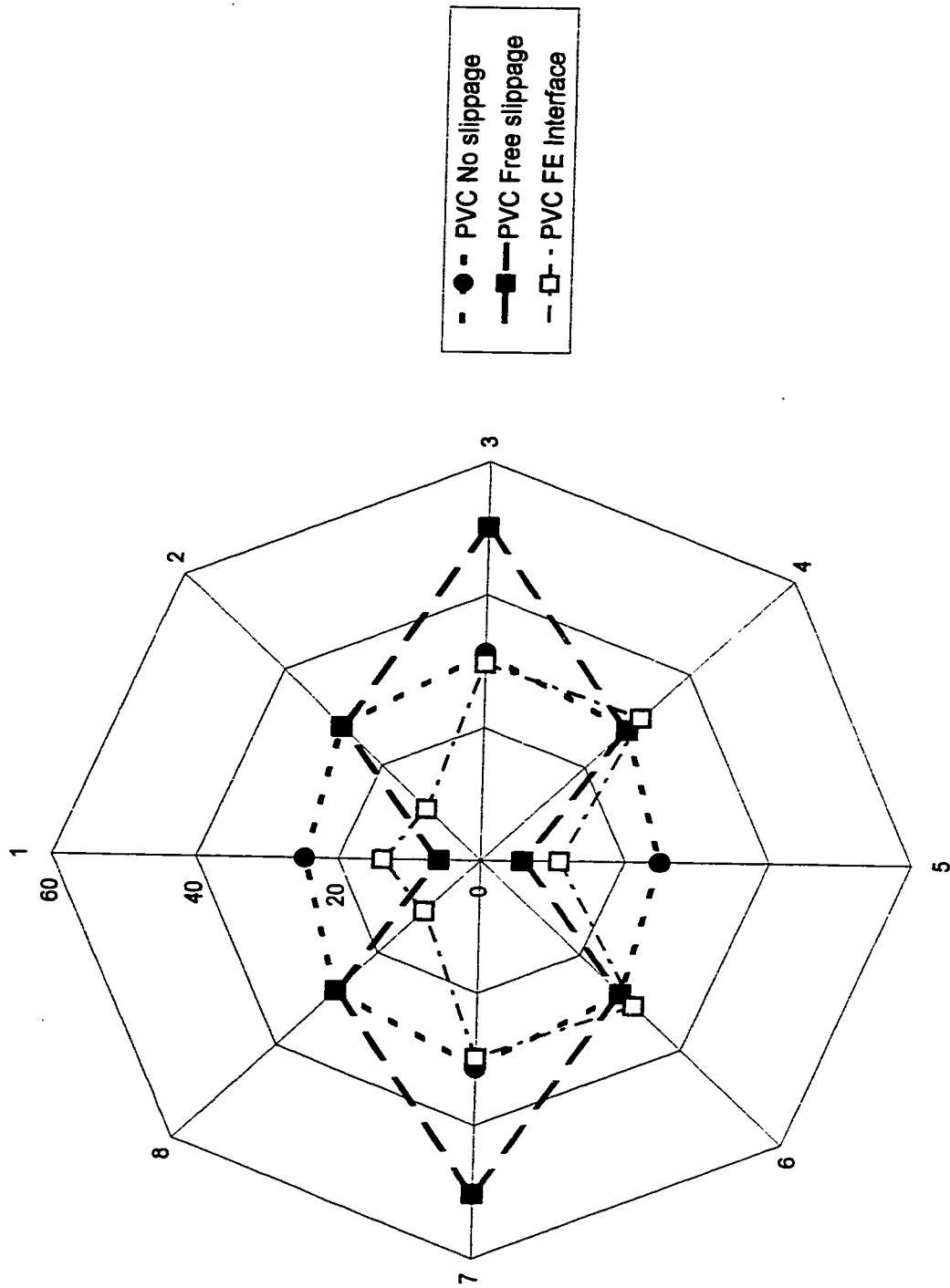


Figure 7.2: Normal stress distribution around the PVC pipe using the elastic solution of Hoëg (kPa)

CHAPTER 8

SUMMARY AND CONCLUSIONS

The behaviour of a buried Fiber Reinforced Plastic (FRP) pipe and a buried Poly Vinyl Chloride (PVC) pipe, with diameters of 1500-mm and 900-mm respectively, were analysed with the PLAXIS finite element code and with some analytical methods. The analyses were based on the field experiment and numerical analysis of Mohri and Kawabata (1995) presented in the literature review.

The literature review identified the most frequently used analytical methods to analyse buried flexible pipes. These methods are the Marston (1913) theory, the Iowa formula by Spangler (1941), the modified Iowa formula by Watkins (1958), the modified Iowa formula by Greenwood and Lang (1990), the elastic solution by Burns and Richard (1964), the elastic solution by Høeg (1968), the viscoelastic solution by Chua and Lytton (1989) and the finite element method. The finite element method is proven to be superior because of its capability to consider non-linear soil behaviour, non-homogeneous backfill material and stages of construction. In addition, the literature review presented the failure modes of flexible pipes and an investigation on soil-structure interface behaviour.

An experimental program was undertaken on a PVC-Stone dust interface using the Cyclic Three-Dimensional Interface (C3DI) device (Fakarhian and Evgin, 1996). It was found that the shear strength of the interface is not significantly influenced by the surface roughness of the PVC surface with the normal stress varying from 100 to 300 kPa. However, the shear strength tended to decrease as the number of cycles increased. The results also showed that there is no adhesion at the PVC-Stone dust interface and that the ratio of the interface friction angle to the soil friction angle is about 0.67. Further studies should be done to analyse the behaviour of different kinds of soils under various moisture content, and under constant normal stresses below 100 kPa.

Two numerical analyses were performed on a Fiber Reinforced Plastic (FRP) pipe in order to point out the effect of interface elements on the numerical results. The first analysis included

interface elements and gave a good approximation of the deformations of the pipe in the field experiment of Mohri and Kawabata (1995). The second analysis did not include interface elements. Consequently, the vertical elongation was 30% greater. The interface elements have a significant influence on the deformations of the pipe when backfilling from the bedding to the crown. However, they did not seem to influence the deformation of the pipe during backfilling from the the crown to the ground surface.

The finite element analyses on the FRP pipe showed that the interface elements have also an influence on the earth pressure distribution around the pipe and the forces in the pipe wall. In the analysi without interface elements, there was an increase of 40% of the normal force in the pipe, and an increase of 20% of the bending moment at the springline. Arching effect was observed in both analyses. However, it was more pronounced in the analysis with interface due to the relative movement of the backfill along the pipe wall. An increase of earth pressure of 50% at the springline, and a decrease of 50% at the invert were observed in the case of the analysis where the soil elements were directly connected to the beam elements. The earth pressure calculated and measured by Mohri and Kawabata (1995) were higher than the results obtained in the present analysis. The PLAXIS finite element code should be improved in order to simulate the compaction more adequately. The method used in this thesis caused some iteration problems. In addition, it would be useful and more realistic to be able to modify the soil properties throughout an analysis.

The analyses on the PVC pipe measured the influence of the interface shear strength on the deformation of the pipe, and on the earth pressure distribution around the pipe. It was observed that the reduction of the interface shear strength by 50% did not influence the deformation of the pipe. However, the weaker interface caused a decrease of 26% of the earth pressure at the crown, and 30% at the invert. Further study should be undertaken on the effect of the interface shear strength on a deep buried flexible pipe.

The analytical methods used to predict the behaviour of the FRP and the PVC pipe were the modified Iowa formula by Watkins (1958) and Greenwood and Lang (1990), and the elastic solution of Høeg (1968). The Watkins' modified Iowa formula gave results somewhat close

to the field experiment of Mohri and Kawabata (1995) if the deformation due to the stages of construction are not taken into account. Moreover, it is assumed in the Iowa formula that the pipe deforms as an ellipse, which is not the case for low stiffness pipe. The modification brought by Greenwood and Lang (1990) made the results more readily comparable with those of the field experiment because it takes into account the stages of construction. The predictions overestimated the horizontal deflection by about 25%. The elastic solution of Høeg (1968) with no slippage at the interface provided good approximations of the field experiment. The analysis with free slippage overestimated the horizontal deflection by almost 100%. This analytical solution does not consider the stages of construction. Therefore, the results of the earth pressure distribution are not comparable with the finite element analyses and the field experiment.

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APPENDIX

DATA OF THE INTERFACE TESTS

First series of tests

Test #A

Type of test: Monotonic test in the Y direction

Boundary condition: Constant normal stress

Soil sample: New stone dust

Plate: Scratched PVC

Roughness (Ra): Y: 0.574 μm X: 0.334 μm

$\sigma_n = 100 \text{ kPa}$

Initial normal displacement: -0.62 mm

Maximum shear stress: -54.5 kPa

Normal displacement: Z: -0.16 mm

Relative density of the soil sample: 81.57 %

Friction angle of the interface ϕ_i : 28.6°

Coefficient of friction: 0.55

Test #B

Type of test: Monotonic test in the Y direction

Boundary condition: Constant normal stress

Soil sample: New stone dust

Plate: Scratched PVC

Roughness (Ra): Y: 0.88 μm X: 0.65 μm

$\sigma_n = 300 \text{ kPa}$

Initial normal displacement: -0.87 mm

Maximum shear stress: 169.5 kPa

Normal displacement: Z: 0.09 mm

Relative density: 97.25 %

Friction angle of the interface ϕ_i : 29.5°

Coefficient of friction: 0.57

Test #C

Type of test: Monotonic test in the Y direction

Boundary condition: Constant normal stress

Soil sample: New stone dust

Plate: Scratched PVC

Roughness (Ra): Y: 1.454 μm X: 1.516

$\sigma_n = 500 \text{ kPa}$

Initial normal displacement: - mm

Maximum shear stress: -281.9 kPa

Normal displacement: Z: -0.47 mm

Relative density: 93.0 %

Friction angle of the interface ϕ_i : 29.4°

Coefficient of friction: 0.56

Second series of tests (+ / - are the direction of the test as shown in Figure 8.12)

Test #1

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: New PVC

Roughness (Ra): 0.0 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.69 mm

Maximum shear stress: X+: 106.6 kPa X-: -118.8 kPa

Normal displacement: Z: -0.09 mm Z: -0.22 mm
Relative density: 87.64 %
Friction angle of the interface ϕ_i : 30.7°
Coefficient of friction: 0.59

Test #2

Type of test: Cyclic test in the X direction
Boundary condition: Constant normal stress
Soil sample: New crushed stone
Plate: Scratched PVC from the previous test
Roughness (Ra): X: 0.492 μm Y: 0.75 μm
 $\sigma_n = 200 \text{ kPa}$
Initial normal displacement: -0.69 mm
Maximum shear stress: X+: 108.6 kPa X-: -107.3 kPa
Normal displacement: Z: -0.12 mm Z: -0.3 mm
Relative density: 94.37 %
Friction angle of the interface ϕ_i : 28.5°
Coefficient of friction: 0.54

Test #3

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: 0.852 μm Y: 0.886 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.8 mm

Maximum shear stress: X+: 100.5 kPa X-: -115.4 kPa

Normal displacement: Z: -0.08 mm Z: -0.22 mm

Relative density: 88.6 %

Friction angle of the interface ϕ_i : 30°

Coefficient of friction: 0.58

Test #4

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: $1.37 \mu\text{m}$ $0.956 \mu\text{m}$

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.86 mm

Maximum shear stress: X+: 107.3 kPa X-: -109.3 kPa

Normal displacement: Z: -0.14 mm Z: -0.3 mm

Relative density: 84.24 %

Friction angle of the interface ϕ_i : 28.7°

Coefficient of friction: 0.55

Test #5

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: $1.082 \mu\text{m}$ Y: $1.396 \mu\text{m}$

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.65 mm

Maximum shear stress: X+: 108.0 kPa X-: -112.7 kPa

Normal displacement: Z: -0.09 mm Z: -0.22 mm

Relative density: 93.35 %

Friction angle of the interface ϕ_i : 29.4°

Coefficient of friction: 0.56

Test #6

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: 0.99 μm Y: 1.568 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.86 mm

Maximum shear stress: X+: 105.9 kPa X-: -203.1 kPa

Normal displacement: Z: -0.07 mm Z: -0.22 mm

Relative density: 91.08 %

Friction angle of the interface ϕ_i : 27.9°

Coefficient of friction: 0.53

Comment: For this test, I did zero the load cell X1 for the second part of the cycle. That explains the high shear stress value of -203.1 kPa. A load cell computes the load from the strain increment between an interval of time. In this particular case, there is tension in the cell during the first part of the cycle. At the beginning of the second part, or when the plate going back to the starting point, the load cell will still read tension from the strain gage, and then the tension will start to reduce and get into the compression phase. However, if the value of the load cell is zeroed after the first part, the strain gages will compute the increment from zero. Therefore, the load cell will indicate compression even if there is still tension in the load cell.

Test #7

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: 1.154 μm Y: 2.244 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.82 mm

Maximum shear stress: X+: 102.6 kPa X-: 118.8 kPa X+: 108.6 kPa

Normal displacement: Z: -0.07 mm Z: -0.22 mm Z: -0.37 kPa

Relative density: 86.81 %

Friction angle of the interface ϕ_i : 30.7°

Coefficient of friction: 0.59

Test #8

Type of test: Cyclic test in the Y direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: From the previous test

Roughness (Ra): X: 1.158 μm Y: 2.346 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.75 mm

Maximum shear stress: Y+: -106.2 kPa Y-: 114.4 kPa

Normal displacement: Z: 0.05 mm Z: -0.31 mm

Relative density: 86.61 %

Friction angle of the interface ϕ_i : 29.8°

Coefficient of friction: 0.57

Test #9

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: New crushed stone

Plate: New PVC

Roughness (Ra): 0.0 μm

$\sigma_n = 200 \text{ kPa}$

Initial normal displacement: -0.61 mm

Maximum shear stress: X+: 112.0 kPa X-: 125.5 kPa

Normal displacement: Z: -0.07 mm Z: -0.22 mm

Relative density: 92.61 %

Friction angle of the interface ϕ_i : 32.1°

Coefficient of friction: 0.63

Comment: The displacements were not constant. It may be due to the installation of the LVDTs. However, the phenomenon did not happen in the next tests.

Test #10

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the test #9

Plate: Scratched PVC from the test #9

$\sigma_n = 200 \text{ kPa}$

Maximum shear stress: X+: 114.0 kPa X-: -121.5 kPa

Normal displacement: Z: -0.31 mm Z: -0.39 mm

Friction angle of the interface ϕ_i : 31.3°

Coefficient of friction: 0.61

Comment: There is an increase in the shear stress at the end of the test in both directions.

Test #11

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200$ kPa

Maximum shear stress: X+: 112.7 kPa

X-: -118.8 kPa

Normal displacement: Z: -0.43 mm

Z: -0.48 mm

Friction angle of the interface ϕ_i : 30.7°

Coefficient of friction: 0.59

Test #12

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200$ kPa

Maximum shear stress: X+: 109.3 kPa

X-: -117.4 kPa

Normal displacement: Z: -0.51 mm

Z: -0.53 mm

Friction angle of the interface ϕ_i : 30.4°

Coefficient of friction: 0.59

Test #13

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200$ kPa

Maximum shear stress: X+: 114.0 kPa

X-: -115.4 kPa

Normal displacement: Z: -56 mm

Z: -0.58 mm

Friction angle of the interface ϕ_i : 30.0°

Coefficient of friction: 0.58

Test #14

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200$ kPa

Maximum shear stress: X+: 108.6 kPa X-: -113.4 kPa

Normal displacement: Z: -0.6 mm Z: -0.62 mm

Friction angle of the interface ϕ_i : 29.6°

Coefficient of friction: 0.57

Test #15

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200$ kPa

Maximum shear stress: X+: 107.3 kPa X-: -112.7 kPa

Normal displacement: Z: -0.63 mm Z: -0.63 mm

Friction angle of the interface ϕ_i : 29.4°

Coefficient of friction: 0.56

Test #16

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

$\sigma_n = 200 \text{ kPa}$

Maximum shear stress: X+: 108.6 kPa

X-: -112.7 kPa

Normal displacement: Z: -0.65 mm

Z: -0.66 mm

Friction angle of the interface ϕ_i : 29.4°

Coefficient of friction: 0.56

Test #17

Type of test: Cyclic test in the X direction

Boundary condition: Constant normal stress

Soil sample: Same as the previous test

Plate: Same as the previous test

Roughness (Ra): X: 1.37 Y: 2.346

$\sigma_n = 200 \text{ kPa}$

Maximum shear stress: X+: 106.6 kPa

X-: -111.3 kPa

Normal displacement: Z: -0.68 mm

Z: -0.68 mm

Friction angle of the interface ϕ_i : 29.1°

Coefficient of friction: 0.56

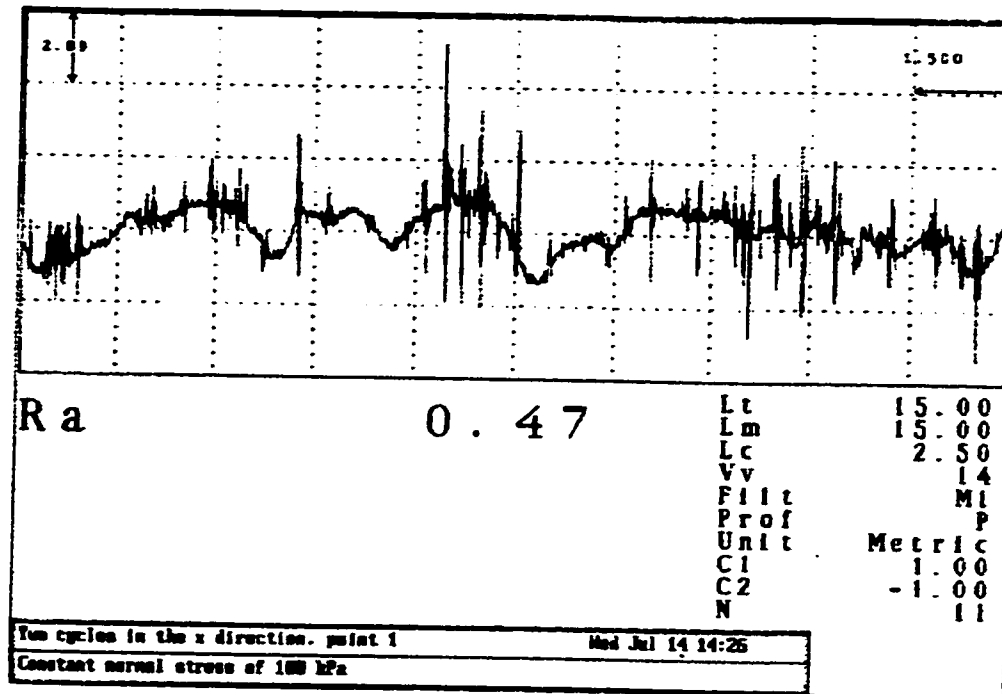


Figure A.1: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 100 kPa in test A.

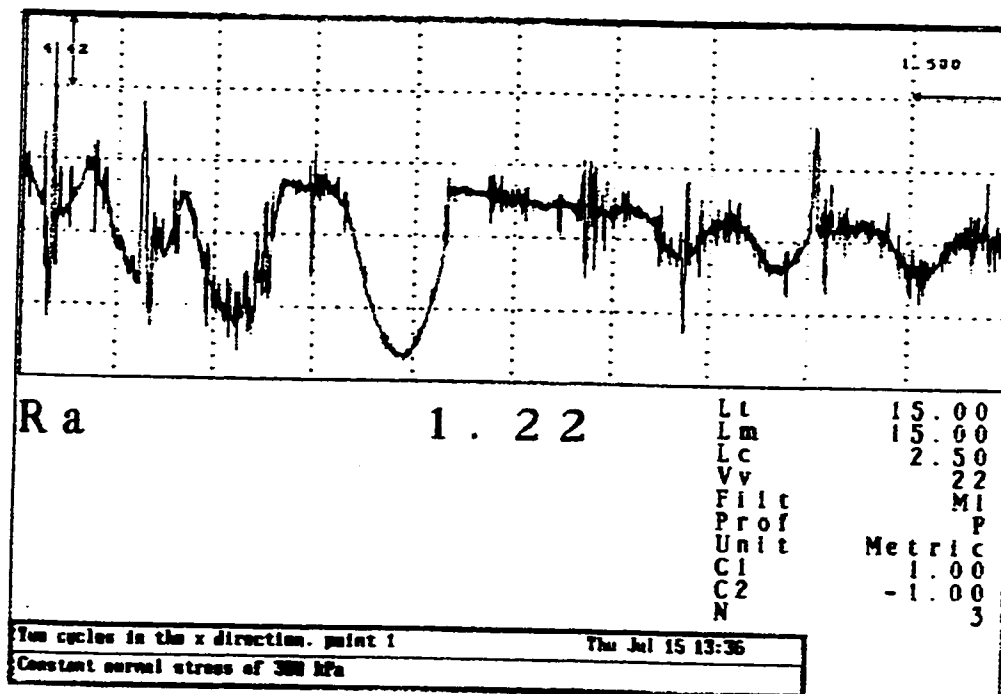


Figure A.2: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 300 kPa in test B.

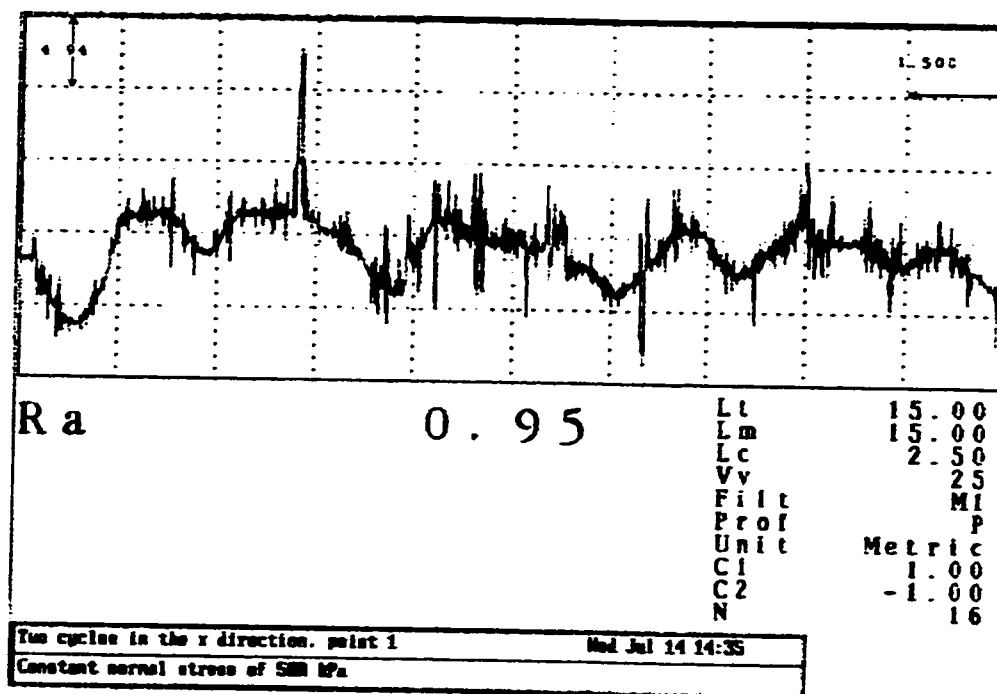


Figure A.3: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 500 kPa in test C.

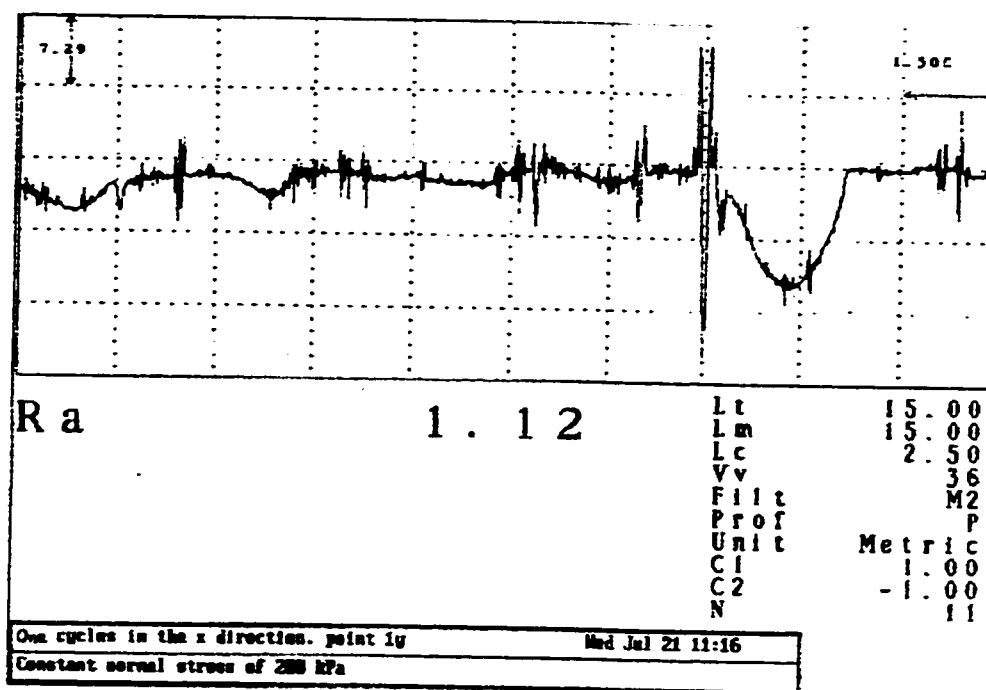


Figure A.4: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #2.

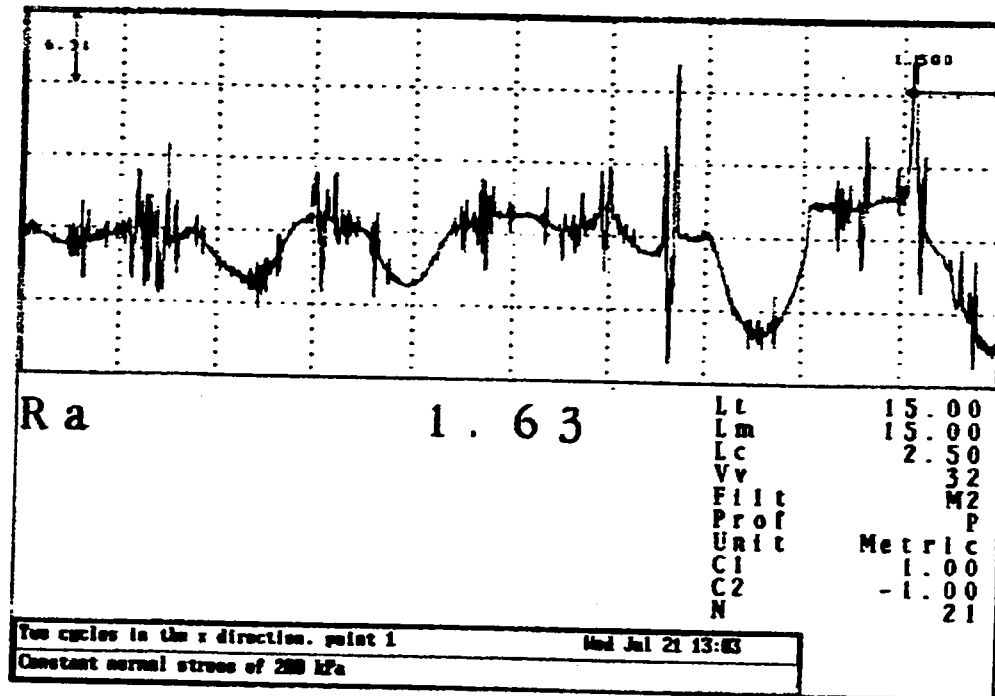


Figure A.5: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #3

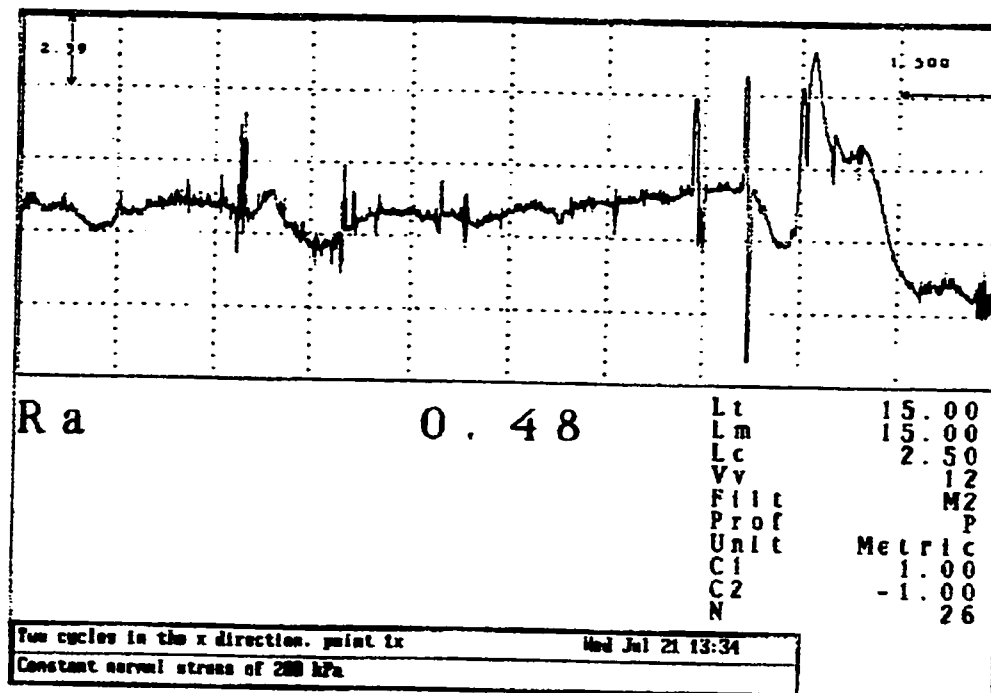


Figure A.6: Initial surface roughness at point 1X (See Figure 8.12) of the PVC plate tested with a constant normal stress of 200 kPa in test #3.

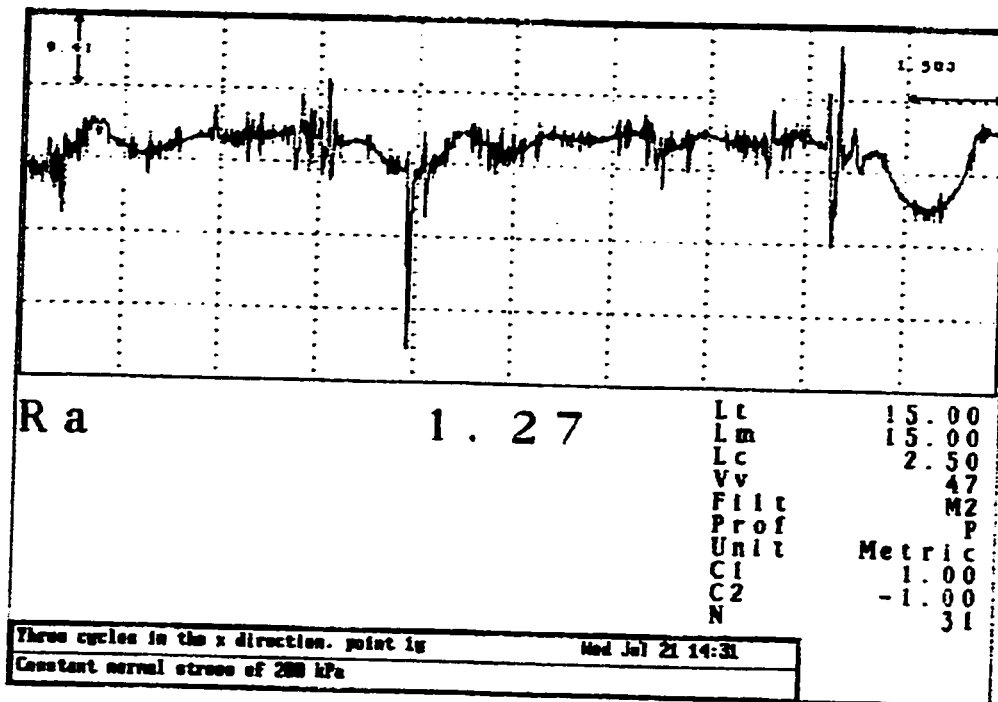


Figure A.7: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #4.

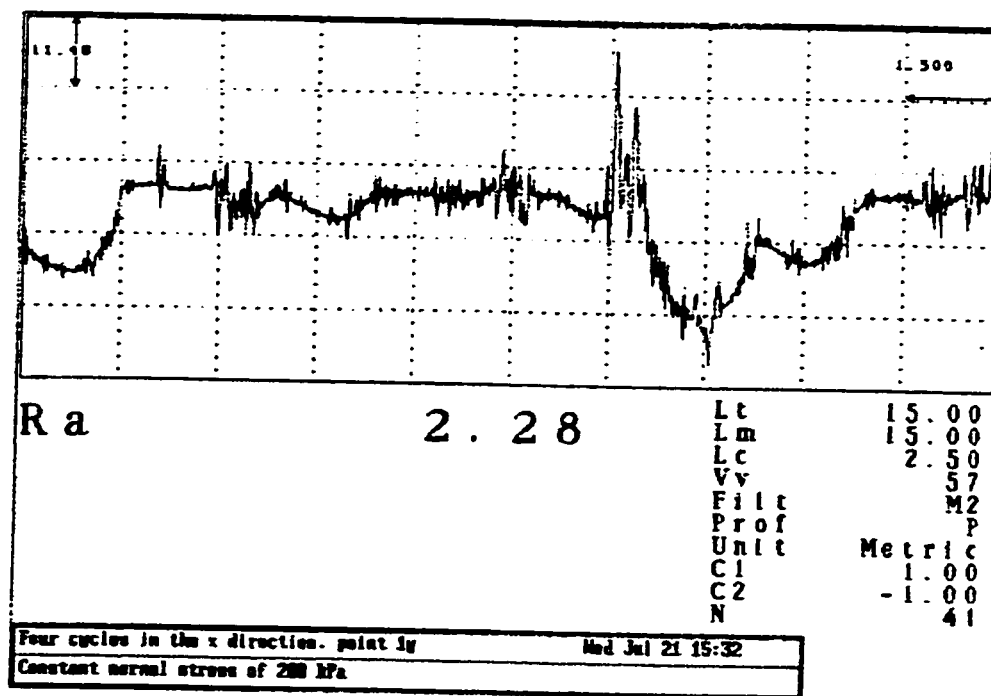


Figure A.8: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #5.

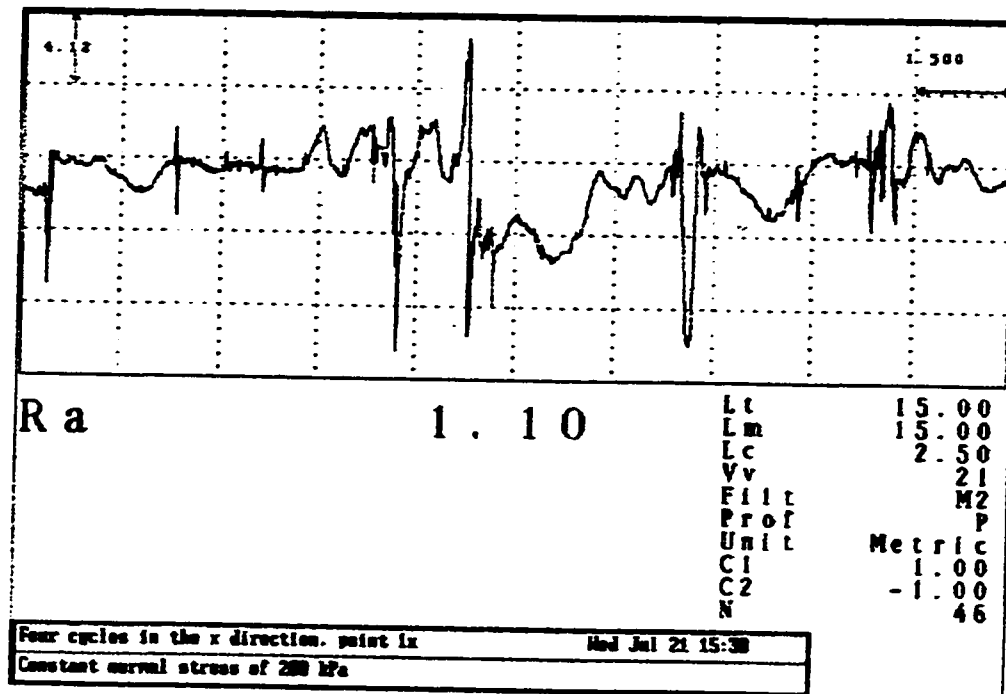


Figure A.9: Initial surface roughness at point 1X (See Figure 8.12) of the PVC plate tested with a constant normal stress of 200 kPa in test #5

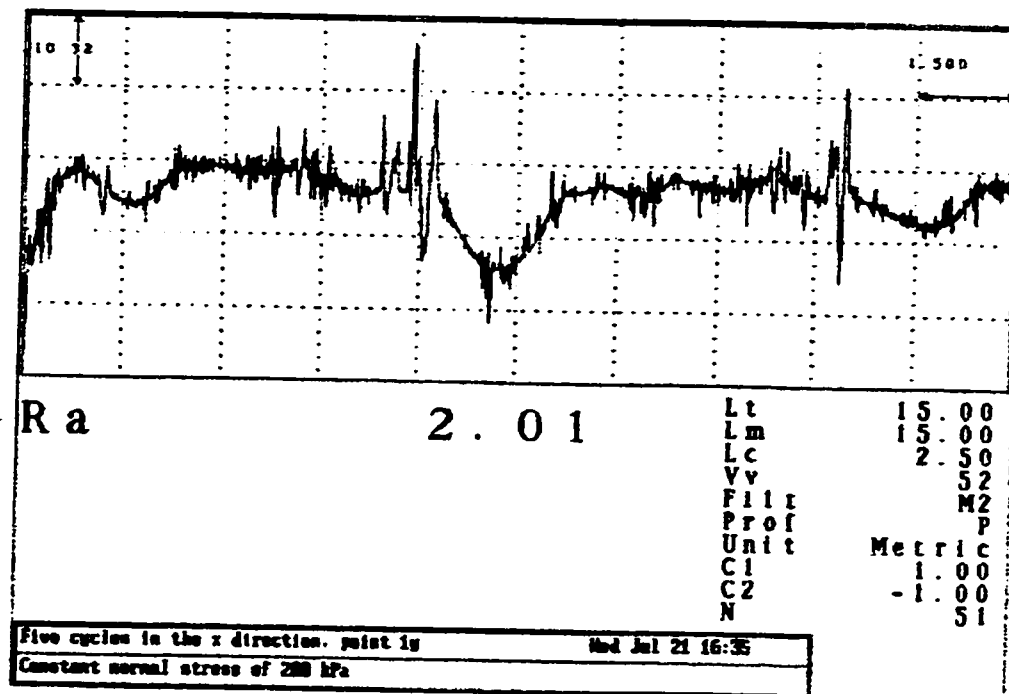


Figure A.10: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #6

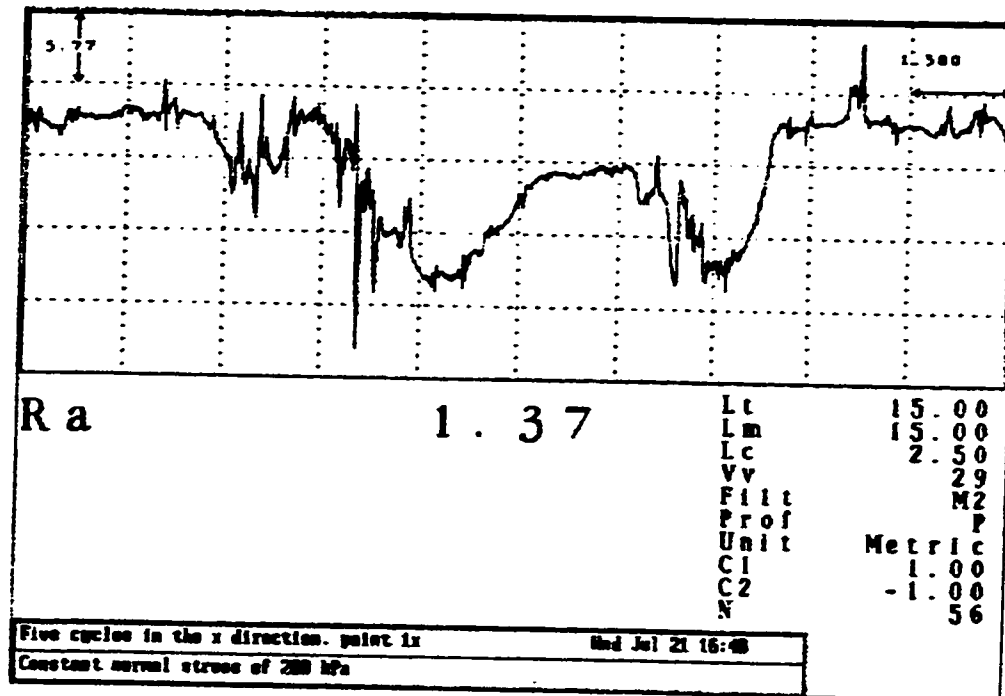


Figure A.11: Initial surface roughness at point 1X (See Figure 8.12) of the PVC plate tested with a constant normal stress of 200 kPa in test #6.

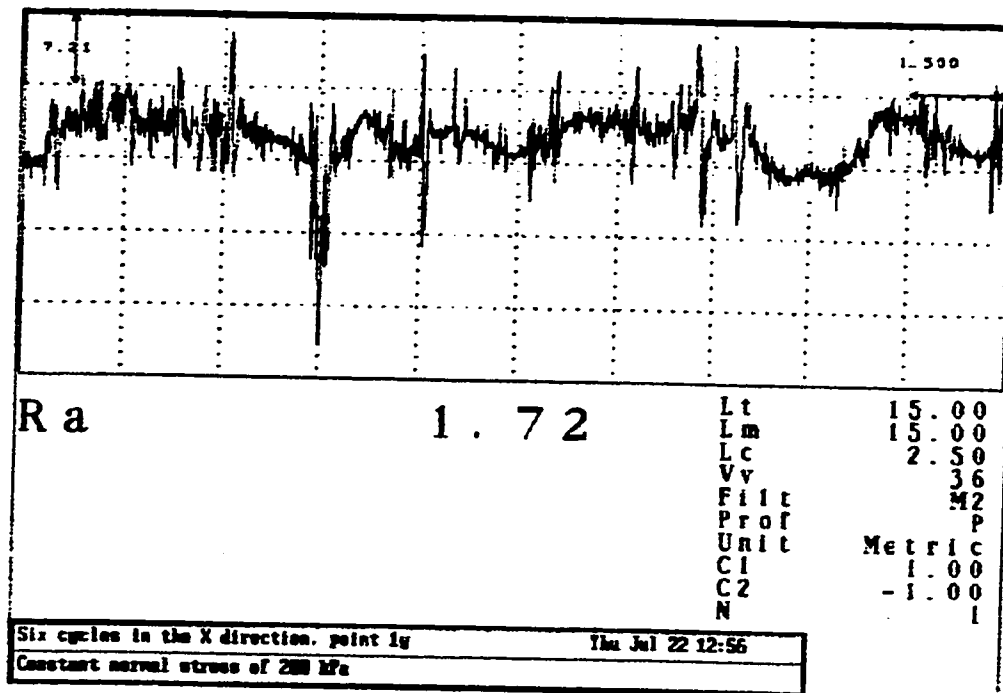


Figure A.12: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #7

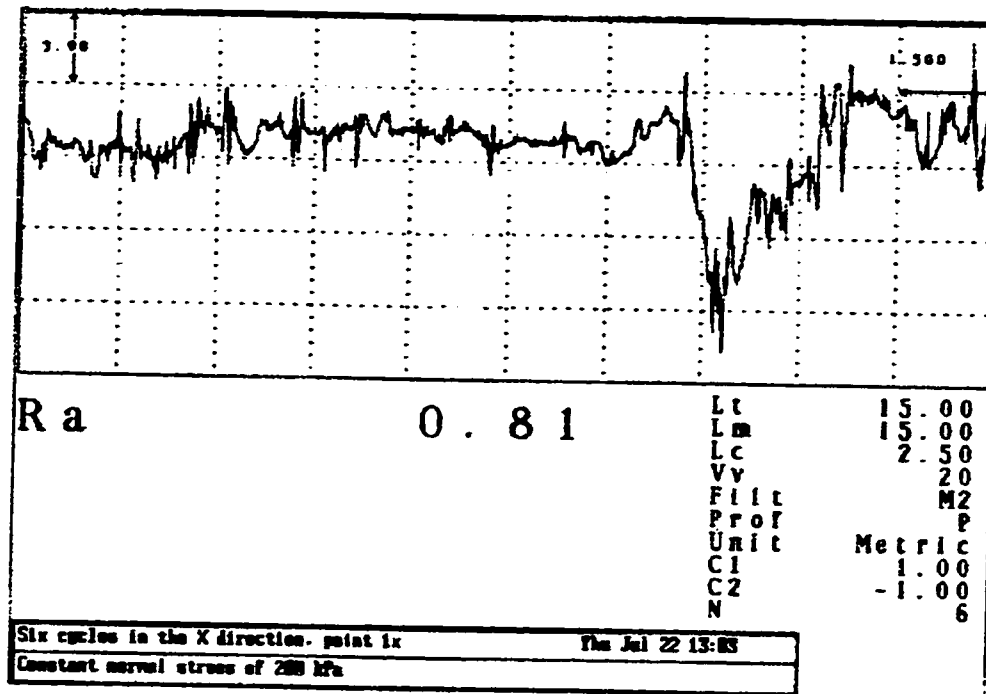


Figure A.13: Initial surface roughness at point 1X (See Figure 8.12) of the PVC plate tested with a constant normal stress of 200 kPa in test #7.

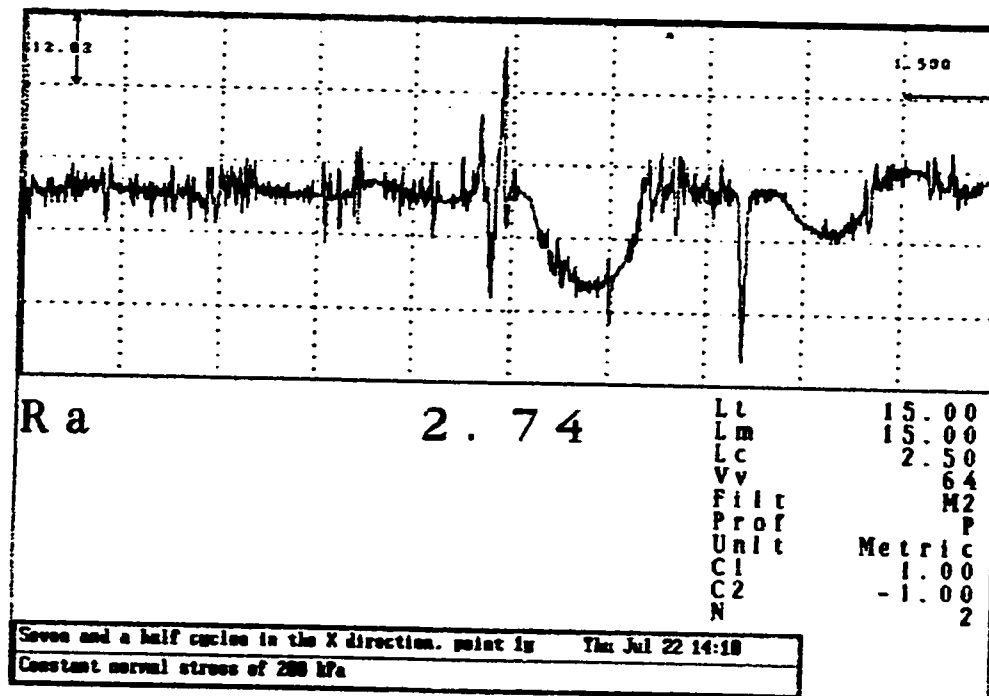


Figure A.14: Initial surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #8

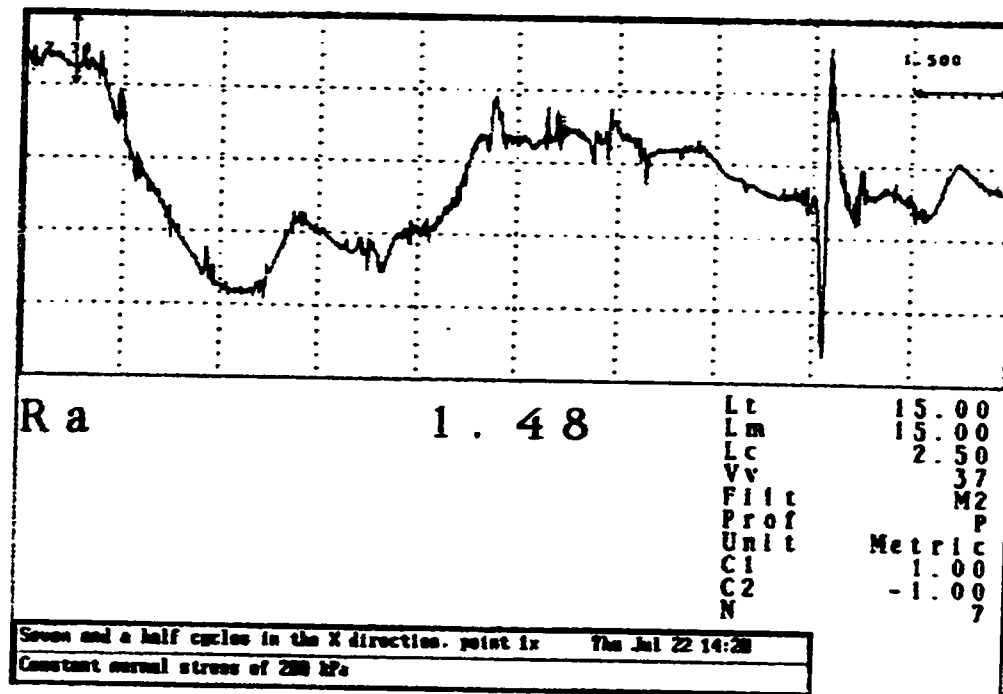


Figure A.15: Initial surface roughness at point 1X (See Figure 8.12) of the PVC plate tested with a constant normal stress of 200 kPa in test #8.

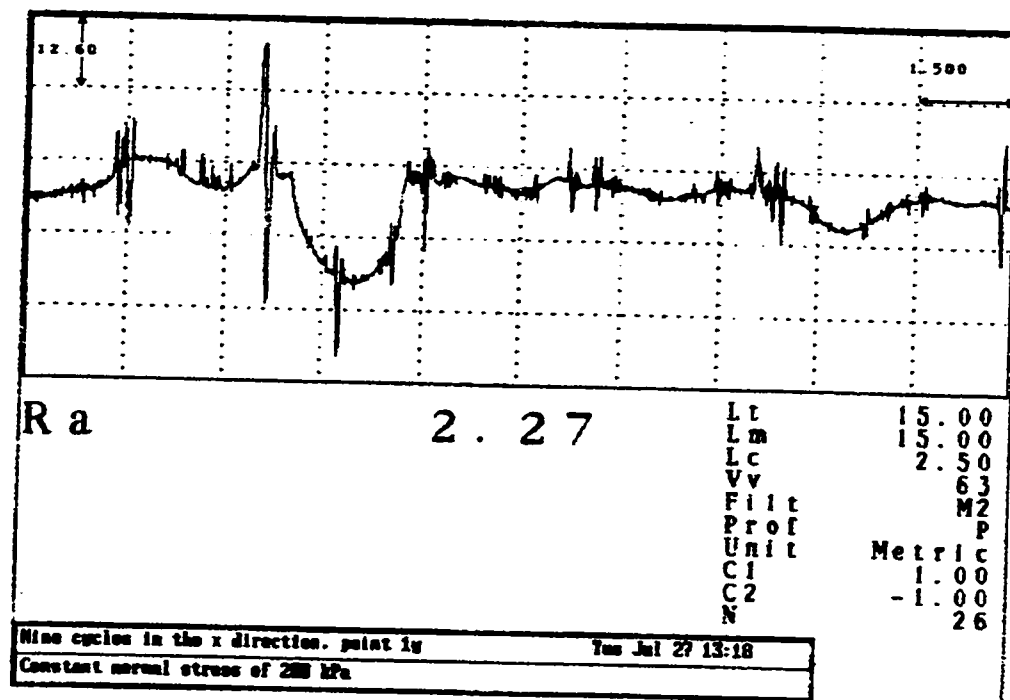


Figure A.16: Surface roughness at point 1Y (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #9

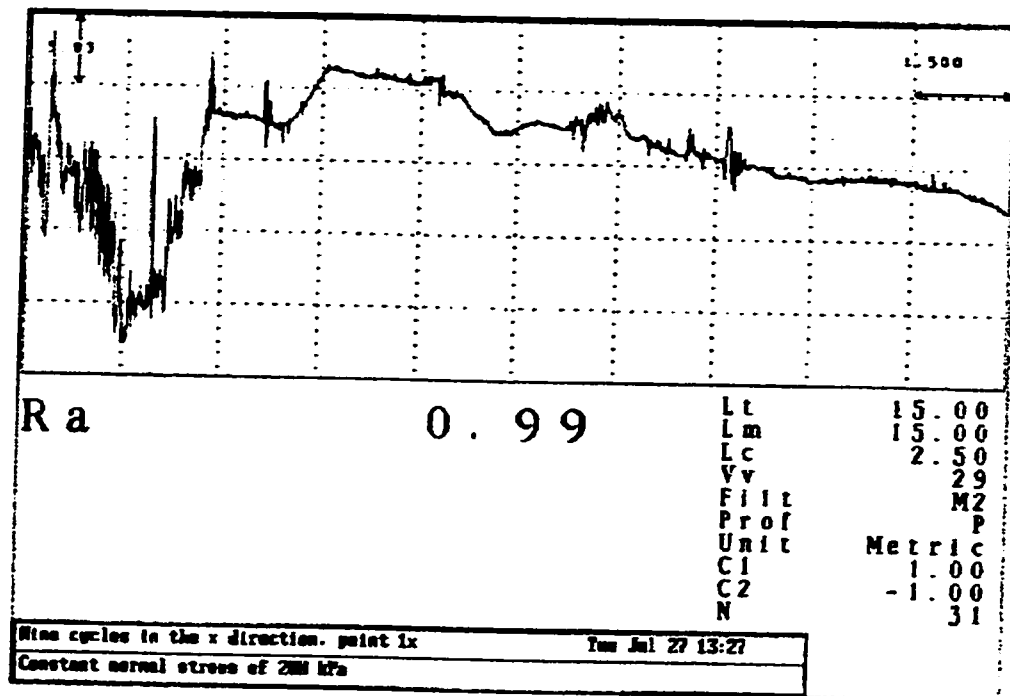


Figure A.17: Surface roughness at point 1X (See Figure 8.12) of the PVC plate sheared with a constant normal stress of 200 kPa in test #9