Behaviour of buried pipes and bored tunnels in sand

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BEHAVIOUR OF BURIED PIPES AND BORED TUNNELS IN SAND

by

ROBERT TALBY, B.Eng.

A Doctoral Thesis submitted in partial fulfilment of the requirements for the award of
Doctor of Philosophy of Loughborough University.

September 1997

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ABSTRACT

This thesis essentially reports an investigation of the behaviour of buried (0.12 to 0.25m diameter) single-walled PVC-U and vitrified clay pipes during installation in a uniform sand surround and when subjected to applied surface loading. An additional simple study of tail void displacements due to tunnelling in sand is also presented.

Controlled laboratory tests were conducted in a glass-faced, steel-sided box. The buried pipes were installed perpendicular to the glass face and were subjected to static and cyclic loading, simulating increasing overburden stress and the passing of traffic over a shallow buried pipe respectively. The simulated shallow tunnel tests were also conducted perpendicular to the glass and involved withdrawal of the outer of two concentrically placed tubes.

Photographs were taken of the sand particles and the buried structure in the plane of the cross section together with strain gauge readings on the pipe or tunnel wall throughout installation and loading/shield withdrawal. The resulting sand displacements are presented in the form of horizontal and vertical contour plots. Pipe deflections and volumetric and shear strain contours of the sand were also determined for the buried pipe tests. The shape of the deformed pipe and the imposed stress at the pipe springline were inferred from the pipe wall strains.

During the PVC-U pipe tests, the deformation of the pipe caused the applied stress to be transferred to the sidefill via arching in the surrounding soil. This was associated with a reduction of applied stress reaching the pipe. Increasing the initial soil stiffness reduced the magnitude of the pipe and soil displacements and the stress carried by the pipe. Use of a vitrified clay pipe however, caused the soil surround to settle relative to the pipe. Soil shear strain contour plots are used to highlight the mechanisms of the transfer of applied stress onto, or away from, the buried pipes, and are related to the shape of the deformed pipe in the PVC-U pipe tests.
The test data also allowed standard buried pipe design methods and installation procedures to be critically appraised. The soil movements recorded during the tunnel tests were shown to be similar to those recorded during the buried PVC-U pipe tests, indicating a similar soil loading transfer mechanism.
ACKNOWLEDGEMENTS

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Finally, I would like to thank my friends, particularly Karen, family and those researchers, particularly Mr E. Faragher, who have offered help and understanding throughout the project.
NOTATION

CAPITAL LETTERS

A  Cross sectional area of pipe/tunnel
A_r  Degree of Arching
B_d  Width of trench
C  Cover depth
C  Compressibility ratio (Hoeg, 1968)
C_e  Positive projection load coefficient
C_d  Narrow trench load coefficient
C_e  Height of equal settlement
D  Outside diameter of pipe/tunnel or width of trapdoor
D_1  Effective width between shear planes
D_l  Deflection lag factor
D_o  Original outside diameter of pipe/tunnel
E  Elastic modulus
E_1  Initial pipe modulus after one minute in parallel plate test
E_p  Elastic modulus of pipe
E_s  Elastic modulus of soil
E'  Modulus of soil reaction
F  Flexibility Ratio
I  Second moment of area of pipe
K  Empirical constant (Equation 3.30)
K  Ratio of the horizontal and vertical stress
K_a  Rankine's active earth pressure coefficient
K_b  Bedding constant
K_cr  Critical horizontal and vertical stress ratio separating Mode I and Mode II (Wong and Kaiser, 1991)
K_d  Ratio of the horizontal and vertical pressure due to dead load
          (Gumbel, 1983)
K_o  Ratio of the horizontal and vertical stress at rest
L_R  Load reduction factor
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N )</td>
<td>Number of independent readings from sampling (Equation 5.1)</td>
</tr>
<tr>
<td>( N )</td>
<td>Hoop thrust in pipe wall</td>
</tr>
<tr>
<td>( N_y )</td>
<td>Distortional component of hoop thrust in pipe wall</td>
</tr>
<tr>
<td>( N_z )</td>
<td>Uniform component of hoop thrust in pipe wall</td>
</tr>
<tr>
<td>( M )</td>
<td>Refraction index of glass (Appendix 1)</td>
</tr>
<tr>
<td>( M )</td>
<td>Bending moment in pipe wall</td>
</tr>
<tr>
<td>( M_y )</td>
<td>Distortional component of bending moment thrust in pipe wall</td>
</tr>
<tr>
<td>( P )</td>
<td>Applied vertical stress</td>
</tr>
<tr>
<td>( P_i )</td>
<td>Support pressure inside tunnel</td>
</tr>
<tr>
<td>( P_{ic} )</td>
<td>Support pressure inside tunnel at collapse</td>
</tr>
<tr>
<td>( P_a )</td>
<td>Vertical pressure above tunnel (soil and surcharge)</td>
</tr>
<tr>
<td>( R )</td>
<td>Radius of outside of pipe/tunnel</td>
</tr>
<tr>
<td>( R )</td>
<td>Stiffness-geometry parameter (Equation 2.6)</td>
</tr>
<tr>
<td>( S_c )</td>
<td>Compression stiffness of pipe</td>
</tr>
<tr>
<td>( S_f )</td>
<td>Flexural stiffness of pipe</td>
</tr>
<tr>
<td>( S_x )</td>
<td>Standard deviation from sampling (Equation 5.1)</td>
</tr>
<tr>
<td>( S_{\bar{x}} )</td>
<td>Standard error of sample mean (Equation 5.2)</td>
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<tr>
<td>( T )</td>
<td>Predicted hoop thrust at the pipe/tunnel springline</td>
</tr>
<tr>
<td>( U_s )</td>
<td>Surface surcharge load per unit length</td>
</tr>
<tr>
<td>( W )</td>
<td>Surface soil settlement</td>
</tr>
<tr>
<td>( W_c )</td>
<td>Tunnel crown soil settlement (Equation 3.33)</td>
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<tr>
<td>( W_s )</td>
<td>Ground surface soil settlement (Equation 3.33)</td>
</tr>
<tr>
<td>( W_{\text{max}} )</td>
<td>Maximum surface soil settlement</td>
</tr>
<tr>
<td>( W_e )</td>
<td>Imposed load on buried pipe per unit length</td>
</tr>
<tr>
<td>( \Delta R )</td>
<td>Radial displacement of tunnel lining</td>
</tr>
<tr>
<td>( \Delta X )</td>
<td>Horizontal pipe deflection</td>
</tr>
<tr>
<td>( Y )</td>
<td>Flexural stiffness ratio (Gumbel, 1983)</td>
</tr>
<tr>
<td>( Z )</td>
<td>Compression stiffness ratio (Gumbel, 1983)</td>
</tr>
<tr>
<td>( Z )</td>
<td>Depth from the ground surface to pipe/tunnel axis</td>
</tr>
</tbody>
</table>
c \hspace{1cm} \text{cohesion} \\
\text{d}_{10} \hspace{1cm} \text{Particle size for which 10\% of total passes} \\
\text{d}_{30} \hspace{1cm} \text{Particle size for which 30\% of total passes} \\
\text{d}_{60} \hspace{1cm} \text{Particle size for which 60\% of total passes} \\
\text{d}_c \hspace{1cm} \text{Shortening of the pipe in the vertical dimension} \\
e_a \hspace{1cm} \text{modulus of passive resistance} \\
i \hspace{1cm} \text{Trough width parameter (Error function curve)} \\
m \hspace{1cm} \text{Power law constant} \\
n \hspace{1cm} \text{Number of restrictions from sampling (Equation 5.1)} \\
o \hspace{1cm} \text{Overcut dimension in tunnel tests} \\
p \hspace{1cm} \text{Projection ratio} \\
p_v \hspace{1cm} \text{Vertical soil pressure} \\
p_h \hspace{1cm} \text{Horizontal soil pressure} \\
p_y \hspace{1cm} \text{Distortional component of pressure} \\
p_z \hspace{1cm} \text{Uniform component of pressure} \\
r \hspace{1cm} \text{Radial distance from centre of pipe/tunnel} \\
r_{sd} \hspace{1cm} \text{Settlement ratio} \\
s_f \hspace{1cm} \text{Settlement of the pipe invert} \\
s_g \hspace{1cm} \text{Settlement of the original surface of natural ground} \\
s_m \hspace{1cm} \text{Settlement of sidefill} \\
t \hspace{1cm} \text{structure wall thickness} \\
t \hspace{1cm} \text{time (Equation 3.30, in minutes)} \\
q \hspace{1cm} \text{Uniformly distributed surcharge} \\
u \hspace{1cm} \text{Radial deformation} \\
v \hspace{1cm} \text{Circumferential (or hoop) deformation} \\
x \hspace{1cm} \text{Mean of actual observations from sampling (Equation 5.1)} \\
x_i \hspace{1cm} \text{Recorded value from sampling (Equation 5.1)} \\
x \hspace{1cm} \text{Horizontal Cartesian co-ordinate} \\
y \hspace{1cm} \text{Vertical Cartesian co-ordinate} \\
z \hspace{1cm} \text{Longitudinal Cartesian co-ordinate}
GREEK SYMBOLS

\( \alpha \) Bedding angle (Iowa formula)
\( \alpha \) Arching factor (Gumbel, 1983)
\( \alpha_z \) Uniform thrust coefficient
\( \beta \) Distortional thrust coefficient
\( \delta \) Vertical deflection of buried structure
\( \delta_h \) Horizontal deflection of buried pipe
\( \delta_y \) Uniform diametral pipe strain
\( \delta_z \) Distortional diametral pipe strain
\( \varepsilon_i \) Circumferential internal wall strain
\( \varepsilon_0 \) Circumferential external wall strain
\( \phi' \) Angle of shearing resistance
\( \gamma \) Unit weight of soil
\( \lambda_z \) Magnification factor
\( \mu \) Coefficient of sliding friction
\( \mu \) True mean from sampling (Equation 5.3)
\( \mu \varepsilon \) Microstrain \( (1 \times 10^{-6}) \)
\( \theta \) Angle from horizontal axis of pipe/tunnel
\( \sigma_{1} \) Major principal stress
\( \sigma_{3} \) Minor principal stress
\( \sigma_v \) Vertical stress
\( \sigma_h \) Horizontal stress
\( \sigma_r \) Radial stress
\( \sigma_s \) Uniform surface stress
\( \sigma_t \) Tunnel support pressure
\( \sigma_{yi} \) Distortional stress component for the first load case
\( \sigma_{yi} \) Distortional stress component for the second load case
\( \sigma_{zi} \) Uniform stress component for the first load case
\( \sigma_{zi} \) Uniform stress component for the second load case
\( \sigma_\theta \) Circumferential (or hoop) stress
\( \tau_{\theta} \) Shear stress
\( \nu \) Poisson’s ratio

\( \nu_p \) Poisson’s ratio of pipe

\( \nu_s \) Poisson’s ratio of soil

\( \psi' \) Dilation angle of soil

**ABBREVIATIONS**

FS Free or full slippage

GW Groundwater level

HDS Horizontal pipe diametral

L External loading case (Figure 2.15)

LHS Left hand side

NS No slippage

O.D. Outside diameter

PP Poorly placed

PPL Poorly placed loose

R_d Relative density

RHS Right hand side

U Unloading excavation condition (Figure 2.15)

VDS Vertical pipe diametral strain

WP Well placed

WPD Well placed dense

WPL Well placed loose
MEASUREMENTS

A negative pipe wall strain (thus inferred pipe wall stress) denotes compression. To maintain consistency between positive and negative degree of loading, the resulting measured and applied load at the springline of the buried structures are quoted with positive and negative signs, where a negative signs denotes compression. The applied surface stress from the water bag arrangement is by its nature a compressive stress and is thus stated as a magnitude only (i.e. 150kPa).

A pipe wall and sand particle displacement to the right and upwards is positive.

A positive VDS or HDS denotes a decrease in pipe diameter and both are expressed as a percentage.

On the LHS of the vertical pipe axis the occurrence of positive and negative arching strains is denoted by positive and negative signs respectively, whereas on the RHS the notation of the shear strain is reversed. A negative volumetric soil strain denotes compression.
CHAPTER ONE
INTRODUCTION

1.1 BACKGROUND TO BURIED PIPES AND BORED TUNNELS

Buried pipes and bored tunnels are two of the most common forms of buried structure. They typically consist of a long circular passageway that allows the transportation of a medium from one point to another to occur beneath the ground surface. Both buried pipes and bored tunnels are used to transport fluids, for example water and sewage, whilst tunnels are additionally constructed to permit the movement of people. Correspondingly, they represent an important part of the infrastructure of a society.

A buried structure can be constructed by three methods. The first of these is a "cut and cover" technique, which initially involves digging a trench, constructing the structure within it and then covering the structure over with soil. The second method of construction involves boring a passageway through the natural soil or rock. The third type is an "immersed tube" construction, which is similar to the "cut and cover" except below water. The first method is the most commonly used form as it is primarily used to install smaller sized buried pipelines. The distinction between buried pipes and bored tunnels has become blurred by the use of bored tunnelling techniques to construct pipelines. These methods are commonly referred to as 'trenchless pipelaying techniques' and have developed over recent decades. Prior to installing or renewing a buried passageway, an engineer therefore has a choice of employing trench or trenchless methods. The primary consideration in this selection will be cost. Factors influencing this decision will include the surface access to conduct trenching, the depth of burial which may restrict the use of trenching on safety or economic grounds, and the social cost of disruption of trenching.

The selected method of construction will subsequently influence the method of design used for the structure, and importantly, the determination of the applied loads (hence stresses). The scope of this project has been limited to consider the behaviour of buried non-pressurised pipes that are used for drainage applications. These types of pipes are installed in either a trench or beneath a soil embankment. Placed in either installation condition, the pipe will be subjected to a dead soil load. Additional loading may arise...
on completion of the installation in the form of surface loading. These can include a
dead surface surcharge, for example from buildings or materials, and dynamic cyclic
traffic loading. In addition, the effect of the viscoelastic behaviour of the buried pipe
installation needs to be assessed to ensure that it performs satisfactorily throughout its
lifetime. To achieve an economical design, the timing of these different loading
conditions should be related to short and long-term performance limits. For a drainage
pipe, these performance limits ensure that the pipe maintains (within limits) its size,
shape, line and level, so that no ingress or egress of fluids occurs. The exception to this
are land drains which are designed to permit the ingress of fluids.

Traditionally, the design of a buried drainage pipe has been characterised according to
whether the pipe is 'rigid' or 'flexible'. Brick lined sewers and lengths of small
diameter pipe manufactured from clay and concrete are typically classified as rigid.
These types of pipes and sewers were originally used by Victorian engineers in the UK
in the nineteenth century. Accordingly, they represent the oldest form of drainage with
a proven record of long-term behaviour. The use of plastic for the manufacture of
pipes has in the past two decades increased the use of flexible pipes. The earliest
plastic pipes were manufactured from unplasticised polyvinylchloride (PVC-U) with a
single smooth pipe wall. More recently the need to produce more economical and
larger diameter plastic pipes has led to the development of structural profile walled
pipes. The most commonly used plastics for these types of pipe are High Density
Polyethylene (HDPE) and Polypropylene (PP).

Rigid pipes are laid on a rigid compacted granular bedding such that the inherent
strength of the pipe is sufficient to withstand the applied loads by bending of the pipe
walls. Constructed beneath an earth embankment, the settlement of the soil above the
pipe crown is resisted by the rigid nature of the pipe. This resistance generates a
redistribution of the applied overburden stress such that the rigid pipe appears to
attract a greater proportion than the surround material. The imposed stress is
subsequently transmitted to the underlying bedding and underlying subsoil. Rigid pipes
are subject to brittle fracture if the imposed stress exceeds the strength of the pipe or if
longitudinal differential settlement occurs. The structural requirement of a rigid pipe is
therefore to resist the imposed stress without surface cracking. Application of increased magnitudes of imposed stress would therefore lead to the development of cracking throughout the pipe wall and subsequently, failure. In such circumstances the primary role of a drainage pipe, to stop the ingress and egress of fluid, could not be ensured.

Flexible pipes behave differently to rigid pipes as they have little inherent stiffness. Equilibrium is reached as the pipe deforms which is resisted by an interaction between it and the surrounding soil. The action of the pipe deforming induces two beneficial actions: arching and the mobilisation of lateral passive soil pressure. Due to the lower stiffness of the pipe compared to that of the surrounding soil, the application of overburden soil stresses causes the pipe crown to settle relative to the surrounding soil. This relative displacement induces shear stresses within the overlying soil, reducing the imposed soil stress on the pipe to a magnitude less than that of the overburden soil stress. The transfer of stress from above the pipe to the soil adjacent to the pipe forms an arch shape. For this reason, the shear stresses that cause the redistribution are referred to as "arching" stresses. The second beneficial action is derived by the lateral expansion of the pipe generating lateral passive pressures within the surrounding soil. This mobilisation of passive pressure prevents the pipe from deforming excessively and indirectly supports the vertically applied overburden.

Failure for a flexible pipe installation is classified as either excessive deformation, buckling or excessive hoop compression. Excessive deformation is the most difficult of these structural requirements to define. Field studies of large-diameter thin-walled corrugated steel pipe culverts buried beneath soil embankments measured deformations of up 20% of the culvert diameter. Applying a factor of safety of four to account for variations in workmanship and material quality, resulted in a criteria of a maximum permissible vertical deflection of 5% of the pipe diameter. In the UK this limit of 5% is still used (BSI, 1980) and applies when backfilling has been completed. More recently a long-term limit of 6% has been recommended by the Water Research Centre (WRc, 1986) which states that this limit must not be exceeded on completion of a 12 month
maintenance period. For pipes of sufficient inherent strength, failure will occur by excessive deflection and so is often the most critical performance parameter.

The second type of buried structure was constructed by boring through the natural soil or rock. The scope of this project has been limited to consider the behaviour of tunnels bored through soft ground. Excavation of the natural soil induces a redistribution of stress within the overlying soil, resulting in the generation of arching stresses. In an analogous manner to the behaviour of a buried flexible pipe, these arching stresses reduce the applied stresses to a magnitude less than that of the overburden. The redistribution of soil stress and the convergent soil displacement pattern induced within the overlying soil by the unloading process are arrested in the short-term by the construction of a permanent tunnel lining. Continued excavation without the provision of tunnel support will eventually lead to soil collapse. Longer-term soil settlements and soil stress changes can be induced by the consolidation of the soil as any excess pore-water pressures generated during the construction of the tunnel dissipate. The resulting stresses imposed on the lining are modified, in an analogous manner to a buried pipe, by the relative stiffness of the tunnel lining and the soil. In addition to considering earth pressures on the lining, the design of the tunnel lining must incorporate the effects of the jacking stresses created by the shield.

1.2 OBJECTIVES OF THE RESEARCH

Fundamentally, the behaviour of a buried pipe-soil system is governed by the relative stiffness of the pipe and the soil. It is conceivable therefore for a clay pipe surrounded by rock to behave as a flexible pipe system, whilst conversely, a plastic pipe surrounded by a very soft soil to behave as a rigid pipe system. Understanding the structural behaviour of a buried pipe-soil system requires an understanding of the behaviour of the pipe, the surrounding soil and the interaction between them. For a rigid pipe system the interaction between the pipe and soil will cause an increase of stress on the pipe above the magnitude of the overburden soil stress, whilst for a flexible pipe system, the converse will occur. The most severe load case for a rigid pipe will therefore occur after redistribution of the soil stress. However, for a flexible pipe system the most severe load case will occur prior to redistribution as application of the
initial load causes the pipe to deform. Investigating the interaction between a buried flexible pipe and the surrounding soil is important because the pipe and soil act as a single structural unit. A better understanding of the interaction between the compliant flexible pipe and the surrounding soil will permit the installation of these pipes under larger vertical and horizontal stresses without inducing prominent pipe deflections or pipe wall strains.

The objectives of this research project were to provide a better understanding of buried pipe-soil interaction under static and dynamic load conditions. The pipes used in the research related to those currently in use within the UK for drainage and ducting applications. Testing of two different PVC-U pipes and a vitrified clay pipe was conducted using a glass-sided laboratory test tank. They were installed in an embankment condition to eliminate the influence of a reduction of applied overburden stress through the development of shear stresses between the sides of a trench and the natural soil. To simplify the surround conditions, a uniformly sized Leighton Buzzard sand was used as the complete surround material throughout the study. In addition to the application of a soil overburden stress, a uniform surcharge was applied at the surface of the installation. The static case represented a short-term application of an additional height of fill material of up to 8m. The dynamic case simulated the behaviour of a buried pipe installed beneath a highway. The uniform surface surcharge was applied a maximum of 1000 times to simulate the application of road traffic loading which creates additional transient stresses on the pipe.

To achieve the objective of the project it was necessary to record the behaviour of both the pipe and the soil during these load conditions. The behaviour of the buried pipe and the surrounding fill material can be investigated by recording its displacement. This technique has been widely used in the field of tunnelling. The buried pipes were placed between the glass face and the opposite steel wall of the test tank to allow the behaviour of the buried pipe-soil system to be recorded in the perpendicular plane to the length of the pipe. From the recorded displacements soil strain distributions were determined. In particular, the volumetric and shear soil strain distributions provided a greater understanding of the interaction process. In addition, the behaviour of the pipe
was measured by use of strain gauges attached around the circumference (circumferential direction) and along the length of the pipe (axial direction) on the internal and external face of the pipe wall. These strain gauge measurements indicated how the pipe derived its support from the surround material and the degree of overburden stress imposed on the pipe.

The investigation into buried pipe-soil behaviour was provisionally compared to that of a bored tunnel. Whilst the construction methods differ between these two types of buried structure, there appears to be similarities in behaviour and particularly the manner in which soil stress redistribution occurs. The laboratory tests simulated the construction of a tunnel within a uniform Leighton Buzzard sand by using a previously adopted tail void method. This entailed withdrawing an outer tube (representing the shield) away from the inner tube (representing the lining) whilst the lining remained butted against the glass face. Throughout the withdrawal process the behaviour of the lining and surround soil were recorded using the techniques previously adopted during the buried pipe tests. This involved measuring the soil movements and strain gauge measurements of the lining behaviour.

The research has been reported in eight chapters. A thorough review of the literature is presented in Chapters 2 and 3 to ascertain what information is available concerning soil stress redistribution around buried structures. The review has been separated to enable the principles of this redistribution to be described from small-scale laboratory experiments on simplified buried structures and theoretical solutions. This work is presented in Chapter 2. The review is continued in Chapter 3 to ascertain what information in the form of field observations and experimental investigations is available on soil stress redistribution around buried pipes and bored tunnels. This chapter also includes a description of the methods of design for buried pipes, which has previously been described in Chapter 2 for bored tunnels.

Chapter 4 discusses the philosophy adopted during this research project and explains why alternative approaches were not undertaken. The method of experimental investigation is described in Chapter 5, from which the results are presented and
discussed in Chapters 6 and 7 respectively. The results of the buried pipe tests are compared to design methods and the behaviour observed during the tunnel tests in Chapter 7. Conclusions are drawn from the project in Chapter 8 and suggestions are made for further work.

1.3 TERMINOLOGY

In order to avoid confusion in the use of varied terminology, standard terms have been used throughout this thesis. These terms are defined below:

The name and location of points around the pipe or tunnel circumference are shown in Figure 1.1a. The pipe or tunnel 'crown' refers to the upmost point on the cross-section, whilst the invert is the lowermost, both points lying on the vertical axis. The pipe or tunnel 'springings' lie on the horizontal axis at diametrically opposite points. The 'shoulders' lie between the 'crown' and 'springings', whilst the 'haunches' lie between the 'springings' and the 'invert'.

The definition of the Vertical Diametral Strain (VDS) of a pipe is shown in Figure 1.1b and relates to the change of the internal vertical diameter of a pipe compared to its external diameter. The Horizontal Diametral Strain (HDS) conversely relates to the change of the internal horizontal diameter of the pipe compared to its external diameter. A positive VDS or HDS denotes a decrease in pipe diameter and both are expressed as a percentage.

The soil surrounding a buried pipe installation is shown in Figure 1c. The 'bedding' is the soil lying directly beneath the pipe 'invert'. The 'backfill' is the soil placed between the pipe 'crown' and the ground surface. The 'sidefill' material lies between the 'bedding' and 'backfill' layers, adjacent to the pipe 'springings'.

'Insitu' soil is the soil found naturally on a site.

'Depth of cover' is the distance between the pipe crown and the ground surface.

'Positive Arching' describes the measured (or imposed) stress on the buried structure being less than the externally applied 'free-field' stress (i.e. the applied stress where the structure not present). 'Negative arching' conversely describes the opposite situation.

'Degree of Loading' relates the stress (hence load) measured (or imposed) on a buried pipe to the externally applied 'free-field' stress and is expressed as a percentage.
A degree of loading of less than 100% indicates the occurrence of positive arching. whilst greater than 100% indicates negative arching. The ‘Degree of Arching’ conversely relates the stress (hence load) distributed away from, or onto, the buried structure to the externally applied ‘free-field’ stress.

‘Jacking’ is the process of pushing a pipe train forwards into the ground using a hydraulic jacking equipment.

‘Overcut’ is the extra circumferential excavation that arises because of the greater diameter of the shield than that of the installed lining.

‘Prototype’ is the full scale buried pipe or tunnelling technique, as opposed to a scaled down version used for modelling purposes.
(a) **Pipe terminology**

V.D.S. = $\frac{\delta}{D_0}$

$\delta$ = change in internal diameter (mm)

$D_0$ = original external diameter (mm)

(b) **Definition of vertical diametral strain**

(c) **Installation terminology**

Figure 1.1 Definitions and Terminology
2 REVIEW OF LITERATURE ON ARCHING AND ELASTIC STRESS DISTRIBUTION

2.1 INTRODUCTION

The review of previous studies of the behaviour of buried pipes and bored soft ground tunnels has been separated into two sections. In Chapter One reference was made to the redistribution of stress in the overlying soil due to the presence of a buried structure, whether a buried pipe or a bored tunnel. Examination of the influence of this redistribution of stress has initially been assessed by describing studies conducted on idealised buried structures. These are described in Section 2.2 under the heading Arching Studies. Within this section a theoretical method to calculate the imposed stress on a buried structure is described, based upon the development of shear planes in the overlying soil. To calculate the redistribution of stress prior to the yielding of the overlying soil methods based upon elastic theory have been developed for buried pipes and bored tunnels. These are described in Section 2.3 under the heading Elastic Stress Distribution. Discussion regarding theoretical modelling has in the review of literature been limited to outlining relatively simple theoretical solutions which aim to simulate the behaviour of buried pipes and bored tunnels. A review of numerical modelling techniques, for example finite element analysis, has been omitted as these are deemed to be outside the scope of the project. Chapter Two concludes with a discussion of the similarity of the behaviour of buried pipes and bored tunnels. The aim of describing this work in Chapter Two is to aid the interpretation of field studies and previous laboratory studies conducted on buried pipes and bored tunnels which are described in Chapter Three. Chapter Three concludes with a summary of the main findings from the work reported in both Chapters Two and Three.

2.2 ARCHING STUDIES

2.2.1 Introduction

The aim of Section 2.2 is to describe studies undertaken to investigate the influence of the arching stresses induced in the soil overlying a buried structure. Section 2.2.2 describes two ‘trapdoor’ experiments, Section 2.2.3 presents the theoretical ‘shear-plane’ method and Section 2.2.4 outlines other laboratory based arching studies.
2.2.2 Experimental Trapdoor Studies

Terzaghi (1936) conducted probably the simplest and most famous study into the redistribution of stress in the soil overlying a buried structure. These are widely known as his trapdoor experiments. The laboratory arrangement, reproduced in Figure 2.1a, shows a horizontal stratum of dry sand, of unit weight $\gamma$ and depth $C$, initially supported on a rigid horizontal base containing a rectangular trapdoor of width $a_1$. The vertical stress on the base of the structure, including the trapdoor, at the start of the test was equal to the full soil overburden stress ($\gamma C$). Allowing the trapdoor to yield (i.e. move downwards) resulted in a reduction of stress on the trapdoor to a small fraction of the initial soil overburden stress, and a corresponding increase above the initial soil overburden stress on the structure adjacent to the trapdoor (see Figure 2.1b). This transfer of applied stress from above to the sides of the trapdoor is commonly called arching and because there is a reduction of stress on the trapdoor below the level of the initial soil overburden stress it is referred to in this study as 'positive arching'. Further trapdoor experiments are described later in this section in which the trapdoor was displaced into the overlying soil. Reversing the displacement of the trapdoor was shown to led to an increase of stress on the trapdoor above the level of the initial overburden stress. This form of stress transfer has been referred to in this study as 'negative arching'.

The mechanics of the stress transfer shown in Figures 2.1 a and b is shown in Figures 2.2 a and b. The initial deflection of the trapdoor resulted in a vertical expansion, and lateral contraction, of the soil body, denoted by area $aa_1bb_1$, directly above the trapdoor causing the two bodies of soil adjacent to it, to expand laterally and contract vertically. As a result of this settlement pattern, shearing stresses are formed along the two inclined zones, denoted by the lines $ac$ and $a_1c_1$. It is the formation of these shear stresses in the overlying soil, which are frequently termed arching stresses, that cause the transfer of stress away from, or in some cases onto, the buried structure. Figure 2.3 shows the results of the measured imposed load on the trapdoor against the deflection of the trapdoor for dense and loose sand, denoted by the curves $C_1$ and $C_2$ respectively. The formation of the shearing stresses shown in Figure 2.2a results in the imposed load on the trapdoor corresponding to the minimum value indicated on Figure 2.3. The lower minimum imposed load observed on the trapdoor overlain by dense sand was due to the larger angle of internal friction that would exist in a dense sand than a loose sand, resulting in an increased magnitude of shear stress and greater stress reduction on
the trapdoor. Continued yielding of the trapdoor caused the two previously inclined zones of shearing to become vertical (see Figure 2.2b) corresponding to the ultimate value of imposed load on the trapdoor shown in Figure 2.3. From these results it was concluded that because the ultimate stress, equivalent to an area of sand denoted ‘aa,e’ in Figure 2.1a, remains constant irrespective of continued yielding of the trapdoor and initial soil density, the soil above the trapdoor had reached a state of failure.

Figure 2.4a to c shows the measured horizontal and vertical pressures ($\sigma_h$ and $\sigma_v$ respectively) along a vertical section through the trapdoor and the resulting coefficient of lateral earth pressure ($K$) indicated by the dotted line. These lines are plotted for the states corresponding to the initial full soil overburden load (No trapdoor displacement), minimum (Trapdoor displacement = 1 mm) and ultimate load on the trapdoor (Trapdoor displacement = 7 mm). The results indicate that for the states corresponding to minimum and ultimate imposed load, the horizontal and vertical pressures were approximately 10% of the hydrostatic pressure immediately above the trapdoor but, importantly, were equal to the hydrostatic stress above a cover to trapdoor width (C/D) of 2.5. The corresponding value of $K$ was calculated as unity immediately above the trapdoor, increasing to a value of approximately 1.6 between a C/D ratio of 0.8 and 1.3 for Figures 2.4b and 2.4c respectively, and returning to a value of approximately 0.7 at a ratio of C/D of 2. From this it has been widely reported (e.g. Petroff, 1990) that a value of $K=1$ can be used to calculate the imposed load on a yielding rigid trapdoor. Experiments conducted to assess whether seepage of water through sand influenced the arching mechanism showed that after altering the applied stresses from total to effective stresses, the stress imposed on the trapdoor remained unaltered.

Adopting the trapdoor idea from Terzaghi (1936), McNulty (1965) described a series of laboratory tests using a circular trapdoor positioned centrally within a circular test rig. At a pre-selected surface stress and cover depth, the trapdoor was either displaced away from, or into (i.e. upwards), the overlying granular medium to induce positive or negative arching stresses respectively. Throughout each test the displacement of and load (hence stress) on the trapdoor was measured by Linear Variable Displacement Transducers (LVDTs) and load cells respectively.
The results of the positive and negative arching tests are reproduced in Figure 2.5. They are presented in terms of the degree of loading against the relative displacement of the trapdoor compared to its width for varying depths of cover (C/D). The results show that for a corresponding cover depth to trapdoor width ratio (C/D), the induced ultimate negative arching ratio (i.e. degree of loading greater than 100%) required larger relative trapdoor deflections than those required to achieve the ultimate positive arching ratio (i.e. degree of loading less than 100%). Prior to establishing these ultimate values however, it is clearly shown for a particular C/D value the rate of stress distribution decays exponentially as the relative displacement of the trapdoor increases. During positive arching, at comparable values of relative trapdoor displacement increasing the C/D ratio results in a reduction of imposed stress on the trapdoor, the magnitude of which reaches an approximately constant value for ratios exceeding unity. By contrast, the C/D ratio is a significant factor during negative arching particularly if large relative trapdoor displacements are produced.

2.2.3 ‘Shear Plane’ Method

The theory of arching was described in the previous section in terms of failure planes extending from the trapdoor into the overlying soil. Whilst real failure planes are curved in nature, by assuming they were vertical (see Figure 2.2b) Terzaghi (1943) was able to use a derivation originally presented by Janssen (1895) to determine the imposed stress on his rigid yielding trapdoor. The resulting analysis is referred to in this study as the ‘shear plane’ method. It assumes that the vertical stress on any horizontal section in the overlying soil is uniformly distributed and shear planes extend vertically from the two edges of the trapdoor (see Figure 2.6). Using the notation described in Terzaghi (1936), Terzaghi (1943) shows that assuming a soil stratum is subjected to a uniformly distributed surcharge (q), the vertical stress (σv) on the trapdoor can be found from:

\[ \sigma_v = \frac{D(y - 2c / D)}{2K\tan\phi} \left[ 1 - \exp\left(-K\tan\phi \frac{2C}{D}\right) \right] + q \exp\left(-K\tan\phi \frac{2C}{D}\right) \]  

(Eqn. 2.1)

where c and φ are the values of cohesion and the angle of friction between the two displacing masses. As Section 3.2.1 describes in further detail, this theoretical approach was also adopted by Marston (1930) to determine the external loads on buried pipes. It is clear that equation 3.5 relating to the case of a buried pipe installed in a narrow trench and subjected to a uniformly distributed load (Uw) is similar in nature to equation 2.1. The only distinction is that equation
3.5 is used to calculate an imposed load per unit length \((W_c)\) whilst equation 2.1 is used to calculate an imposed stress \(\left(\sigma_N\right)\). Equation 2.1 also shows that the influence of the surface surcharge stress diminishes with increasing depths of cover \((C)\) such that the vertical stress \(\left(\sigma_{vo}\right)\) on the trapdoor approaches the value:

\[
\sigma_{vo} = \frac{D(\gamma - 2c / D)}{2K\tan\phi}
\]

(Eqn. 2.2)

The shear plane method was also used to determine the imposed stress on an upward displacement trapdoor. An application of this is shown in equation 3.3 by Marston (1930) for the case of an embankment settling relative to a buried rigid pipe, thereby inducing negative arching stresses.

To validate this method Ladanyi and Hoyaux (1969) repeated the experimental approach described by Terzaghi (1936) to record both positive and negative arching stresses. Aluminium rods, rather than natural sands adopted by previous researchers, were used to represent the overlying granular material. This was because they exhibited simpler mechanical behaviour thereby allowing easier interpretation of the experimental results. During each experiment the stress on the trapdoor was measured and the displacement of the rods was recorded photographically. Photographs taken from a camera moving simultaneously with the structure, see Figures 2.7a and b, clearly show the formation of a fixed wedge during downward and upward displacement of the structure respectively. This area was indicated by Terzaghi (1936) as area 'aa\(_{1e}\)' in Figure 2.1a. As the results from McNulty (1965) had previously indicated (see Figure 2.5), the formation of the ultimate arching ratio is propagated by less displacement for positive arching when compared with that required for negative arching. This conclusion agrees with the photographic evidence shown by Ladanyi and Hoyaux as the failure wedge for the positive case is more clearly defined (Figure 2.7a) when compared with that for the negative case (Figure 2.7b).

Adapting equation 2.1 to represent the laboratory model experiments led to the assumptions of no cohesion \((c=0)\) within the aluminium rods and no applied uniform surface surcharge \((q=0)\). The value of \(K\) that was adopted was originally proposed by Krynine (1945) and is calculated from equation 2.3. This value differs from the average value of unity proposed by Terzaghi (1936) and \(K=K_a\) assumed by Marston (1930) during his use of the shear plane method to
calculate the external loads on buried pipes, see Section 3.2.1. Krynine argued that $K = K_s$ could not be used to determine the horizontal pressure ($\sigma_h$) at the ends of the horizontal section through the overlying soil, because $K_s$ is stated as the ratio of the minor ($\sigma_3$) and major ($\sigma_1$) principal stresses (i.e. $K_s = \sigma_3 / \sigma_1$). By definition, the principal stresses have no shear stress component associated with them, so the use of $K = K_s$ at a shear plane is consequently incorrect. The correct value of $K$ to be used in this example was derived by the use of a Mohr's circle (see Figure 2.8) in which the lengths $EF$ and $EG$, represented the minor and major principal stresses, $\sigma_3$ and $\sigma_1$, respectively. The ratio of the horizontal to vertical pressure ($\sigma_h / \sigma_v$) at the end of the horizontal section thus corresponded to the ratio of the lengths $AB$ and $CD$, leading to the value of $K$ of:

$$K = \frac{\sigma_h}{\sigma_v} = \frac{1 - \sin^2 \phi}{1 + \sin^2 \phi} \quad \text{(Eqn. 2.3)}$$

This correction has since been cited and described by, amongst others, Christensen (1967) and Handy (1985). Adopting a value of $K = K_s$ in equation 2.1 will therefore underestimate the supporting frictional stress, leading to an unsafe prediction of lateral pressure on grain bins and a conservative value of vertical stress on a pipe buried in a narrow trench.

A comparison of the measured and calculated values of positive arching stresses are shown against depth of cover in Figure 2.9. It indicates that the theory agreed well with the test results at low trapdoor displacements, which is described by the trapdoor deflection/width = 0 line. However, it was unable to model the increase of stress on the trapdoor (see Figure 2.3) as the trapdoor continues to yield simply by reducing the angle of friction from the peak to critical state value. An improved estimate of this increase of stress was obtained when the width of the yielding mass was increased to include the enlargement due to the lateral nature of the inclined shear plane, and is shown in Figure 2.9 by the trapdoor deflection/width = 0.5 line. To do this the authors adopted a proposal by Terzaghi (1943) shown in Figure 2.10. The effective width ($D$) replaces the $D$ term in equation 2.1 and is given by:

$$D_e = D + 2\delta \tan (45^\circ + \phi'/2) \quad \text{(Eqn. 2.4)}$$

By contrast, the experimental results obtained from the upward displacing trapdoor indicated that the method greatly overpredicted the measured vertical pressures. The authors explain that this was due to the assumption that the two failure planes are represented by vertical rigid walls which extend to the surface of the overlying soil stratum. Meyerhof (1969) states that
the prediction for the negative case can be improved by reducing the calculated shear stresses by assuming that they are inclined and do not necessarily reach the ground surface.

In summary, the results of the trapdoor experiments indicate that arching stresses are propagated by a relative displacement of the trapdoor into, or away, from the overlying soil. The formation of an ultimate stress imposed on the trapdoor was shown to be independent of the initial density of the soil, but required sufficient displacement of the trapdoor and height of fill to mobilise fully the shear stresses in the overlying soil. This propagated settlement is analogous to soil displacements induced during the construction of a bored tunnel. Indeed the importance of the trapdoor experiments was emphasised by Terzaghi when he stated that the understanding of the stress distribution above a yielding trapdoor represented the prerequisite for a clearer understanding of the stress distribution in sand around tunnels. Morgan (1961) repeated this by stating that it could be used to determine the lower limit for the vertical stress on a tunnel constructed in granular material. In addition, the method was adopted by Marston (1930) to determine the frictional support provided by a narrow trench to buried pipes. The advantages of the method lie in its simplicity, the clear definition of its assumptions and furthermore, when the results are compared with experimental results the limitations are apparent and can be considered.

2.2.4 Other Arching Studies

In their reviews of research aimed at modelling the arching mechanism, both Gill and True (1966) and Getzler et al (1968) produce similar criticisms of the trapdoor experiments. They argued that the adoption of the shear plane method for the analysis of arching was too simplistic a representation of the actual behaviour of the soil as it assumed vertical, and not curved, shear planes, and considered the forces associated with a failing soil, so disregarding the load changes during the elastic behaviour of a soil. In addition to questioning the appropriate value of $K$ to be adopted in design, Luscher and Hoeg (1964) state that a limitation of the trapdoor analysis is that it does not take into account the geometric configuration of the buried structure.

Noting both the theoretical and experimental deficiencies of the trapdoor experiments, Gill and True (1966) and Getzler et al (1968) conducted similar small-scale laboratory experiments to
induce arching stresses by the application of a surface surcharge stress to a buried structure-soil system of varying relative stiffness. This approach therefore differed from that of the trapdoor experiments in which the displacement of the trapdoor induced the arching stresses. Gill and True (1966) stated the need for this alternative approach was as a result of one of the most important findings from the trapdoor experiments being the establishment of a ultimate soil load which was independent of the magnitude of overpressure (see equation 2.2). They state that this conclusion was derived because an artificial mechanism, the trapdoor, was used to create the shearing stress within the overlying soil medium. It was argued that the soil-stress conditions arising from the relative displacement between a buried pipe and the soil immediately adjacent to it by the application of an overburden stress are quite different from those arising from a yielding trapdoor. The results from these tests vindicated this approach by emphasising the importance of the relative soil-structure stiffness in the behaviour of buried structures, an observation that was not possible from the trapdoor experiments.

To investigate the influence of positive arching in granular materials, Getzler et al (1968) conducted laboratory based experiments in a glass-walled, metal framed box measuring 100 x 200 x 400mm on buried rigid structures connected to a flexible support. By measuring the applied surface stress (σs) and the imposed load (hence stress, σv) on the structure, the degree of arching factor (A,) was defined as:-

\[ A_r = \frac{(\sigma_s - \sigma_v)}{\sigma_s} \]  
(Eqn. 2.5)

The vertical displacement of the surround at varying elevations and distances from the structure was determined by the use of thin discs connected by slender rods to deflectometers. Once the box had been filled with uniform sand poured in horizontal layers, a rubber diaphragm was used to apply a uniform surface surcharge pressure.

Figure 2.11 summarises the measured arching with respect to the applied surface stress for varying depths of cover. At shallow depths of cover the ultimate degree of arching was utilised immediately upon the application of surface stress, remaining approximately constant as the surface stress was increased thereafter. For deeper installations, the degree of arching was activated more gradually and was only fully achieved at very high surface stresses. It was concluded that under the same magnitude of surface stress the degree of positive arching
increased with cover depth, reaching an approximately constant value as the C/D ratio exceeded 1.0. Whilst the manner in which the arching stresses were induced within the overlying soil varied from that used in the trapdoor experiments, similar relationships were shown to be established between the degree of positive arching and cover depth, under an applied surface stress.

The results from model tests conducted by Gill and True (1966) were used to establish a relationship between the degree of arching (Ar) and a non-dimensional stiffness-geometry parameter (R) as:

\[ Ar = 0.87(1 - e^{-0.135R}) \]  

(Eqn. 2.6)

This relationship is shown in Figure 2.12 overlain by the results of the tests. The stiffness aspect of the parameter R concerns the relative stiffness of the buried structure and the surrounding soil. Assuming that the soil stiffness is the dominant factor for a positive arching case, the asymptotic relationship shown in Figure 2.12 indicates that the rate of increase in the degree of arching reduces with increased soil stiffness.

### 2.3 ELASTIC STRESS DISTRIBUTION

#### 2.3.1 Introduction
The aim of this section is to describe the philosophy behind the development of theoretical solutions used to determine the stress redistribution around buried structures prior to the formation of failure planes. Studies modelling the stress and displacement distributions around buried pipes and tunnels using elastic theory are described in Section's 2.3.2 and 2.3.3 respectively.

#### 2.3.2 Buried Pipes
Burns and Richards (1964) presented a set of expressions to determine the elastic stress and displacement distribution around a buried pipe subjected to an applied surface stress at a distant boundary. Similar expressions were later derived and presented by Hoeg (1968), which as Section 3.4 describes in further detail were compared to results from his experimental study. Due to the breadth of work undertaken by Hoeg it has been discussed in this study.
In order to develop an elastic solution, it was assumed that the complete circumference of the pipe wall was fully in contact with the surround so that slippage would only occur locally to the pipe-soil interface. The plane strain analysis assumed to represent the behaviour of a buried pipe is shown in Figure 2.13. The analysis assumes that the buried elastic circular cylinder interacts within an elastic, homogeneous and isotropic medium. The vertical (P) and horizontal stresses (KP), where K is the ratio of horizontal to vertical stress, were applied at distant boundaries so they would not modify the stresses and displacements around a buried cylinder. For this condition to be met by the use of an elastic analysis, the boundaries were assumed to be a minimum of one diameter away from the cylinder. In addition, the unit weight of the medium around the pipe was neglected. To determine the behaviour of the buried pipe-soil system according to the nature of the applied stresses and the relative stiffness of the pipe and the soil, two stiffness ratios were defined. The compressibility ratio determines the relative response of the pipe-soil system to uniform (symmetric) stresses, whilst the flexibility ratio determines the relative response to distortional (asymmetric) stresses. In practice, a large flexibility ratio indicates that the pipe has a low bending stiffness and is typical of a flexible pipe surrounded by a well-compacted surround, whereas a soil-cylinder system with a zero value for both ratios signifies a relatively rigid embedded pipe.

Expressions were presented to determine the radial stress ($\sigma_r$), circumferential (or hoop) stress ($\sigma_\theta$), shear stress ($\tau_\theta$), radial deformation ($u$) and circumferential deformation ($v$) using a radial co-ordinate system of a radial distance ($r$) from the centre of the pipe at an angle ($\theta$) from the horizontal. To assess the influence of shear stresses at the pipe-soil interface, Hoeg adopted the approach described earlier by Burns and Richards (1964). This was achieved by relating the compressibility and flexibility ratios to the extreme boundary conditions at the pipe-soil interface of no slippage and free slippage (no shear stresses) by the use of three constants. Details of these constants, the expressions for stresses and displacements stated above are discussed by both Hoeg (1968) and Poulos and Davis (1974).

Figure 2.14 shows the relationship between the predicted values of radial stress at the pipe-soil interface against the flexibility ratio (F). This figure clearly demonstrates the difference between the predicted radial stresses at the pipe crown ($\theta=90^\circ$) and pipe springline ($\theta=0^\circ$) for a rigid pipe system compared with a flexible pipe system. For a rigid pipe system, where F=0,
the radial stress at the crown is shown to greatly exceed the vertical free field stress of $P$ used in this example, whereas at the springline it approximates to the $0.35P$ assumed in this example for the lateral earth pressure. Increasing the flexibility ratio to a value above 6 is shown not to significantly alter the stress distribution pattern. At these values, applicable to a flexible pipe system, the values of radial stresses at the crown and springline positions approximate to the uniform radial stress of $0.68P$ for the free-slip condition. For the opposite interface condition of no slip (i.e. shear stresses develop at the interface) the stress at the springline is shown to be approximately twice that at the crown.

Assuming that a ductile pipe, typical of a flexible pipe system, will deform to relieve the development of shear stresses at the interface a condition of full slippage can be adopted. Conversely, a condition of no slippage is adopted for a brittle pipe typical of a rigid pipe system. Applying these conditions to the results in Figure 2.14 shows that the stress redistribution around a flexible pipe leads to a uniform stress condition. By deforming initially into the regions of least resistance at the sides of the pipe the resulting approximate elliptical pipe shape induces positive arching stress. This is shown to reduce the imposed stress at the pipe crown to a level below the applied vertical stress, whilst the lateral support is improved. Conversely, for a rigid pipe system the redistribution of soil stress leads to an increase in the vertical stress at the pipe crown. Hoeg showed that these distributions of stresses rapidly approached those of the free-field as the radial distance increased away from that of the circumference of the cylinder.

In a review of the application of elastic theory to the problem of underground structures, Pender (1980) describes a situation of a constructed tunnel, or buried pipe, subjected to an uniform surface surcharge applied at a distant boundary. This was stated as the first of three load cases applicable to these types of structures, the remaining two being discussed in the next section. The analysis presented for this first load case repeated that of Hoeg (1968), with the exception that the vertical and horizontal stresses were described in terms of uniform and distortional stress components, and the influence of the compressibility and flexibility stiffness ratios, and the boundary conditions of no and free slippage were omitted. The methods presented by Burns and Richards (1964) and Hoeg (1968) also formed the basis of the development by Gumbel (1983) of the TRL design method for buried flexible pipes. Whilst
this method is described in greater detail in Section 3.2.3, the rationale behind it was based upon the response of a buried pipe to the application of stresses both at a distant boundary, which has been described in this section, and at the pipe-soil interface. This latter loading case is described in the next section and is typically used to calculate the stress and displacement distributions around a tunnel bored through natural soil.

2.3.3 Bored Tunnels

A bored tunnel is constructed through natural soil, which is in a pre-stressed state. Excavation of the natural soil to form the tunnel void induces, above the tunnel, a reduction in the normal soil stress and an increase in the soil shear stress (Atkinson and Mair, 1981). The change of stress induced by the unloading process results in a convergent settlement pattern above the tunnel, see Section 3.8, which is arrested in the short-term by the installation of the tunnel lining. The resulting stresses imposed on the lining are modified, in an analogous manner to a buried pipe, by the relative stiffness of the tunnel and the soil. To highlight how the different construction processes of buried pipes and bored tunnels influence the determination external stresses, Einstein and Schwartz (1979) state that applying the approach described by Hoeg (1968) of ‘external loading’ would lead to calculated tunnel lining stresses that were 50-100% too conservative.

To calculate the stress and displacement distribution around a bored tunnel an alternative method was required to account for the construction process of a bored tunnel. Einstein and Schwartz (1979) referred to this type of analysis as ‘excavation unloading’. Morgan (1961) presents expressions based upon the excavation unloading of the soil which were subsequently modified by Muir Wood (1975) to apply to plane strain, and not the plane stress conditions assumed by Morgan. Both analysis simplified the true stress regime around a tunnel by neglecting the influence of initial shear stresses at the tunnel-soil interface, a situation that was later corrected by Curtis (1975).

These approaches, along with that of Hoeg (1968) were summarised by Pender (1980). As Section 2.3.2 described, the first load case related to a buried underground structure subjected to a uniform surface surcharge applied at a distant boundary. Einstein and Schwartz (1979) referred to this loading case as ‘external loading’. Pender described the (second load case)
analysis appropriate to bored tunnels by initially deriving terms for the in-situ stresses that acted at the tunnel periphery before it was bored. Construction of a tunnel causes an incremental change in the in-situ stresses which diminished with distance from the tunnel. The expressions relating to this change of stress indicate the occurrence of a convergent soil displacement pattern. This is shown to correspond with a transfer of stress shown by the positive arching mechanism, as the largest reduction of radial stress occurs at the tunnel crown and the largest increase of circumferential stresses occurs at the tunnel springline. To calculate the final state of stress after the tunnel has been bored, the equations relating to the incremental change of stress in the second load case are added to those for the in-situ soil stresses. The author stated that this final result was identical to the expressions for the first load case.

The resulting interaction between the tunnel lining and the surrounding soil medium caused by the incremental stress changes during the excavation process was described in the third load case. As the degree of interaction between the ground and the lining depends upon the relative stiffness of the lining and the ground, and because the stiffness of the lining is different in compression and tension, the reaction of the lining to symmetrical and asymmetrical loading were considered separately. The resulting interaction analysis is based upon the radial loading at the tunnel periphery, leading to expressions for radial, circumferential and shear stresses, in addition to expressions for radial and circumferential displacements. As the analysis assumed that the tunnel deformed to an elliptical shape, the radial displacement is shown to be inwards towards the tunnel at the crown and outwards away from the tunnel at the springline.

Gumbel (1981) extended the analysis presented by Pender (1980) by stating that the solutions derived for the second load case could be deduced directly from the solutions for the first load case without the need for an interaction analysis the third load case. The reasoning behind this argument was that for any soil-structure system for which the interface shear transfer conditions are known (i.e. full/no slippage), the structural response to loading at a distant boundary differs only from that when the loading is applied at the soil-structure interface in terms of relative magnitudes of uniform and distortional loading. To convert any first load case solution to the corresponding second load case solution it is necessary to apply a factor:-
\[
\frac{\sigma_{zi}}{\sigma_{zii}} = \frac{1}{2(1-\nu)} \tag{Eqn. 2.7}
\]
to all the aspects of design influenced by the uniform stress component, and a factor:-

\[
\frac{\sigma_{yi}}{\sigma_{yii}} = \frac{3 - 4\nu}{4(1-\nu)} \tag{Eqn. 2.8}
\]
to all those aspects influenced by the distortional stress component, where:

\[
\begin{align*}
\sigma_{zi} & = \text{uniform stress component for the first load case} \\
\sigma_{zii} & = \text{uniform stress component for the second load case} \\
\sigma_{yi} & = \text{distortional stress component for the first load case} \\
\sigma_{yii} & = \text{distortional stress component for the second load case} \\
\nu & = \text{Poisson's ratio of soil}
\end{align*}
\]

and the relevant aspects of design are the radial deflection, circumferential hoop thrust and ring bending moment. The practical relevance of the solutions presented for the first and second load cases was highlighted by Gumbel during the design of buried pipes in which a combination of the two load cases would be required to account for the separate effects from the backfill weight, groundwater pressure and surface loading, see Section 3.2.3. The author concluded his discussion by stating that similar considerations would also be present in the design of a tunnel.

An alternative analysis for the design of a tunnel lining was presented by Einstein and Schwartz (1979). By using the principles described later by Pender (1980) and Gumbel (1981), the external loading analysis described by Hoeg (1968) was adapted to describe an unloading excavation analysis for tunnels (see Figure 2.13). The first step in the analysis involved determining the displacements in the soil medium due to the in-situ stresses. The incremental displacements induced during the construction of a tunnel were found by calculating the displacements defined by the first load case and subtracting from these those displacements due to the in-situ stresses. By applying the boundary conditions at the ground-surface interface of no and free slippage to the incremental displacements, the contact stresses were determined. The internal lining forces induced by the contact stresses at the soil-lining interface were derived in the final step.
The proportional relationships described by Gumbel (1981) in equations 2.28 and 2.29, for the conversion of a first to second load case solutions are indicated in Figure 2.15 which was originally presented by Einstein and Schwartz (1979). Figure 2.15 shows the relationship between the normalised hoop thrust \( T/PR \) for a buried structure, where:–

\[
T = \text{predicted hoop thrust at the springline of the structure} \\
P = \text{applied vertical stress at distant boundary.} \\
R = \text{radius of the buried structure}
\]

with respect to the relative soil-structure stiffness. This relationship has been calculated using the method proposed by Hoeg (1968) for the first load case and the method proposed by the authors for the second load case. These are denoted by the symbols \( L \) and \( U \) to indicate ‘external loading’ and ‘excavation unloading’ respectively and are determined for the shear stress boundary conditions of no and free slip, denoted by the symbols NS and FS respectively. A further insight into the similarity between the first and second load cases was provided by Einstein and Schwartz (1980), during a reply to Muir Wood, by stating that their compressibility and flexibility ratios only varied in magnitude from those stated by Muir Wood (1975), and not in principle.

2.4 CONCLUDING DISCUSSION

This chapter clearly shows that stress redistribution occurs within the overlying soil above a buried structure. This redistribution has been characterised by the occurrence of either positive or negative arching. Positive arching stresses were induced when the trapdoor displaced in a downwards direction. The resulting soil stress and displacement distributions are analogous to that propagated during the excavation process of a bored tunnel. The magnitude of stress imposed on the lining after this redistribution has occurred was shown to depend primarily upon the relative stiffness of the lining and the surrounding soil. The method used to determine the stress imposed on the lining from that applied at the lining-soil interface was shown to be similar in principle to that used to calculate the imposed stress on a buried structure subjected to an applied stress at a distant boundary. These solutions were in addition shown to be greatly dictated by the assumed shear stress condition at the structure interface. For the same relative structure-soil stiffness and interface shear conditions, solutions based upon stress applied at the structure-soil interface, typical of a bored tunnel, were able to be converted to those based upon stress applied to a distant boundary, typical of a buried pipe. The second part of the
review of literature described in Chapter Three, continues this investigation by describing field and laboratory studies conducted on buried pipes and tunnels. In addition, methods used to design buried pipes are described which have been based upon the theoretical methods described in this Chapter.
Figure 2.1  Terzaghi's Trapdoor Experiments (after Terzaghi, 1936)
(a) Apparatus for investigating arching layer of sand above yielding trapdoor in
horizontal platform, (b) Pressure on platform and trapdoor before and after
slight lowering of door
Figure 2.2  Distribution of Arching Stresses above Trapdoor
(after Terzaghi, 1936)
Figure 2.3  Graph of Imposed Load against Trapdoor Displacement (after Terzaghi, 1936)

Ratio between horizontal and vertical unit pressure (K), $\sigma_h : \sigma_v$

Figure 2.4  Graph of Vertical and Horizontal Pressure with Cover
Depth at Displacements of (a) 0 mm, (b) 1 mm and (c) 7 mm (after Terzaghi, 1936)
Figure 2.5  Positive and Negative Arching Experiments  
(after McNulty, 1965)

Figure 2.6  Assumed Pressure Distribution for Shear-Plane Analysis  
(after Terzaghi, 1936)
Figure 2.7  Wedge formation above Trapdoor (after Ladanyi and Hoyaux, 1969),
(a) During Downward Trapdoor Displacement and
(b) During Upward Trapdoor Displacement
Figure 2.8  Graphical representation of K for Shear Plane Analysis (after Krynine, 1945)

Figure 2.9  Comparison of Measured and Calculated Vertical Stresses for a Yielding Trapdoor at different Cover Depths (after Ladanyi and Hoyaux, 1969)
Figure 2.10  Effective Width between Failure Planes
(after Terzaghi, 1943)

Figure 2.11  Degree of Arching versus Surface Stress for different Cover
Depths (after Getzler et al, 1968)
Figure 2.12 Comparison of Arching Experiments against Relationship of Degree of Arching against Stiffness-Geometry Factor (after Gill, 1966) (Circular (O) and Rectangular ([]) structures)

\[ A_r = 0.87 (1 - e^{-0.135R}) \]

Figure 2.13 Analysis for Buried Structures (after Hoeg, 1968)

Note: Boundaries are far away from tunnel.
Figure 2.14  Relationship of Radial Contact Stress on Buried Pipe versus Flexibility Ratio (after Hoeg, 1968)

Figure 2.15  Comparison of Thrust Coefficient at springline for 'External Loading' (L) and 'Unloading Excavation' (U) analysis (after Einstein and Schwartz, 1979)
CHAPTER THREE
3 REVIEW OF LITERATURE ON BURIED PIPES AND BORED TUNNELS

3.1 INTRODUCTION
The objective of this Chapter is to outline both field and laboratory studies describing the behaviour of both buried pipes and bored tunnels. Emphasis has been placed on studies that have dealt with the behaviour of both the buried structure and the surrounding soil, and because of the nature of the experimental work conducted by this author, on studies conducted in granular materials.

Sections 3.3 and 3.4 describe field and laboratory studies conducted using buried pipes respectively. Several of these studies discuss the long-term behaviour of a buried pipe system and this work has been summarised in Section 3.5. To provide a link with the research reviewed in Chapter 2, Section 3.2 describes the historical development of buried pipe design. Sections 2.3.3 and 2.2.3 described the elastic and plastic analysis of tunnel linings and a separate section on design is therefore not warranted herein. Field studies of bored tunnels are described in Section 3.7, followed by empirical relationships of ground movement in Section 3.8. Section 3.9 describes the laboratory studies of bored tunnels, which includes laboratory investigations into tunnel face stability. The many ideas stated in Sections 3.7, 3.8 and 3.9, are summarised in the form of a model describing ground behaviour and tunnel lining interaction in Section 3.10. Chapter three concludes with a summary of the main findings from both Chapters 2 and 3, and describes the similarities between the behaviour of buried pipes and tunnels.

3.2 HISTORICAL DEVELOPMENT OF BURIED PIPE DESIGN
3.2.1 Introduction - Marston’s Load Theory
The earliest comprehensive study on the behaviour of buried pipes was conducted at Iowa State University, Ames, U.S.A. under the guidance of Anton Marston. Supported by extensive field and laboratory results collected over a period of twenty-one years, Marston (1913 and 1930) published a theory to determine the external loads on closed rigid pipes. This theory was subsequently enhanced by employees at Iowa
State University to include all of the installation conditions stated in Table 3.1 and for use with 'flexible' as well as 'rigid' pipes.

The derivation of Marston's load theory for negative projecting pipes (see Spangler, 1960) has not been described herein, as it combines the ideas used in narrow trench and positive projecting pipe conditions. The theory relating to trench installation conditions has developed according to similar principles to those used to explain the trapdoor experiments, see Sections 2.2.2 and 2.2.3. Sections 3.2.1.1 and 3.2.1.2 outline the load theory due to the application of external soil loading for buried pipes installed in narrow trenches and positive projecting conditions respectively. Section 3.2.1.3 describes the transition condition between these two types of installation and Section 3.2.1.4 presents an adapted method to include the influence of surcharge loading of large and limited lateral extent. The application of Marston's load theory for flexible pipes was outlined by Spangler (1941 and 1948) and is described in Section 3.2.2. The historical development of the design of buried pipes is completed by a description of the Transport Research Laboratory (TRL) method in Section 3.2.3, which was developed by Gumbel (1983) on the basis of elastic stress distribution, described in Section 2.3.

3.2.1.1 Narrow Trench Pipe Condition

The formula derived for calculating loads on pipes in a narrow trench was based upon the theoretical analysis of arching described by Janssen (1895). Marston's derivation used the vertical shear plane method described in Section 2.2.3 by considering a rigid pipe installed in a vertically-sided narrow trench of width $B_d$ at a cover depth of $C$. Assuming that the backfill material in the trench would settle relative to the in-situ soil, vertical shearing stresses would be formed along the boundary between the two. Marston omitted the influence of cohesion between the backfill and the in-situ soil which is a safe assumption. The shear stress is therefore due to the frictional relief component only. By following the method described in Section 2.2.3, Marston derived an expression for the pipe load, $W_c$ as:–

$$W_c = \gamma B_d^2 \left[ 1 - \exp\left(\frac{-2K\mu'C}{B_d}\right) \right]$$

(Eqn. 3.1)
where, $\gamma$ is the unit weight of the soil, $K$ is Rankine’s active earth pressure coefficient ($K_a$) and $\mu'$ the effective coefficient of sliding friction between the fill material and the sides of the ditch ($\tan\delta$). The term contained within the squared bracket is defined as $C_d$ and can be determined directly, or from five curves, denoted by the symbols A to E corresponding to values of $K\mu'$ varying between 0.11 and 0.19, respectively on Figure 3.1.

Analysis of this theory shows it to be arguably conservative in its assumptions that all of the trench load will be borne by a rigid pipe and the adoption of Rankine’s active lateral earth pressure. Young and Trott (1984) state that the load carrying capacity of the sidefill was ignored due to the practical difficulty of ensuring good compaction along a complete pipeline. The effect of this measure, as Wetzorke (1960) illustrated, is to introduce a factor of safety into the design because a well-compacted sidefill will carry a proportion of the trench load.

The question of which value of horizontal to vertical pressure, $K$, should be adopted in an arching analysis was discussed previously in Section 2.2.3. Increasing the value of $K$ increases the magnitude of shearing stresses resulting in a reduction in the predicted vertical stress on the buried pipe. By adopting a value of $K=K_a$, Marston (1930) chose a conservative value for design whilst Terzaghi’s suggestion of $K=1$ maybe too optimistic, particularly for installations buried in loose sands or clays.

The above method assumes that frictional support will be derived between the interface of the backfill and in-situ soil. This will not occur, for example, if compaction of the backfill occurs before the removal of trench sheets. In such cases where frictional support cannot be guaranteed the load across the trench width at the level of the pipe crown represents the weight of the backfill:-

$$W_c = C \gamma B_d$$  \hspace{1cm} (Eqn. 3.2)

3.2.1.2 Positive Projecting Pipes

Spangler (1948) describes the theory used to derive expressions for calculating the external loading on a positive projecting pipe. Figure 3.2 shows a buried pipe of
diameter D buried beneath an embankment of height C. The level of the pipe crown is a distance \( pD \) above the elevation of the natural ground, where \( p \) is defined as the projection ratio. The embankment itself (Figure 3.3) was split into three soil prisms, defined as the ‘interior prism’ for the mass directly above the pipe and between the two vertical shear planes tangential to the sides of the pipe, and the two ‘exterior prisms’ adjacent to the interior prism. The direction of the shearing stresses is governed by the behaviour of the critical plane, which is defined as the horizontal plane in the fill material at the pipe crown at the beginning of construction of the embankment, prior to any settlements. A ‘projection condition’ exists when the critical plane settles more than the top of the pipe, for example with a rigid pipe. This causes the shearing stresses to act downward on the interior prism, resulting in an increase of stress on the pipe above that from the interior prism alone. The converse, known as the ‘ditch condition’ due to its similarity with the narrow trench condition described in Section 3.2.1.1 occurs when the top of the pipe settles more than the critical plane, for example with a flexible pipe. This action relieves the stress on the pipe. A ‘neutral’ condition arises when the settlement of the top of the pipe is equal to that of the critical plane.

In the case of the complete ditch and projection conditions for shallow embankments, the shearing stresses extend to the ground surface. By employing the analysis described in Section 2.2.3, an expression can be derived to calculate the load at the pipe crown \( (W_c) \) as:

\[
W_c = \gamma D^2 \left( \frac{\exp \left( \pm \frac{2K\mu C}{D} \right) - 1}{\pm 2K\mu} \right) \quad (\text{Eqn. 3.3})
\]

where the load coefficient, \( C_e \), is the term contained inside the squared bracket. In equation 3.3 a positive sign denotes negative arching occurring for the complete projection condition, whereas a negative sign denotes positive arching for the complete ditch condition. Both equations 3.1 and 3.3 were derived by applying Janssen’s arching analysis. The differences between the two equations merely relate to the assumptions that the vertical stress at the pipe crown for the positive projection condition is applied over the diameter of the pipe \( (D) \) and not the full trench width \( (B_d) \) and that the coefficient of internal friction applies to the backfill material alone \( (\mu) \) and not between the in-situ soil and the backfill material \( (\mu') \).
In the case of the incomplete ditch and projection conditions for a deep embankment, the shearing stresses do not extend to the ground surface. The distance at which the shear stresses terminate is known as the 'plane of equal settlement' (see Figure 3.3), which is defined as the horizontal plane in the embankment above which the settlement of the interior and exterior soil prisms are equal. The expressions to determine the external loads are the same as those for the complete cases, except the depth of cover (C) is replaced by \( C_e \), the distance from the top of the pipe to the plane of equal settlement, and that the weight of soil overlying the plane (i.e. over distance \( C - C_e \)) is treated as a uniform surcharge.

\( C_e \) is evaluated by equating the total settlements of the interior and exterior soil prisms below the plane of equal settlement. For this to be achieved, Marston defined the settlement pattern according to the following four variables:

- \( s_m \) - settlement of the sidefill (i.e. material adjacent to the pipe within a height \( pD \))
- \( s_g \) - settlement of the original surface of the natural ground.
- \( s_f \) - settlement of the pipe invert.
- \( d_e \) - shortening of the pipe in the vertical dimension.

This notation is illustrated in Figure 3.3, from which it can be seen that the settlement of the critical plane is \( s_m + s_g \) and the settlement of the top of the pipe is \( s_f + d_e \). The relative deformation of the interior and exterior soil prisms, defined as the settlement ratio, \( r_{sd} \), is thus:

\[
 r_{sd} = \frac{(s_m + s_g) - (s_f + d_e)}{s_m} \quad \text{(Eqn. 3.4)}
\]

Using the expressions for settlement ratio and projection ratio to equate the settlements in the exterior and interior prisms, Spangler derived an expression for \( C_e \), and thus \( C_e \), that can only be solved by trial and error. This has been summarised in Figure 3.4, which shows Spangler's computed values of \( C_e \) for different values of \( C/D \) and the multiple of the settlement and projection ratios, \( r_{sd} \rho \), for values of \( K\mu' \) of 0.13 for the trench condition and \( K\mu' \) of 0.19 for the projection condition. The
recommended values of settlement ratio \( r_J \) after Spangler's investigations on actual culverts are summarised in Table 3.2.

Figure 3.4 describes how the distinctions between trench, neutral and projection conditions are determined by the settlement ratio, which is fundamentally governed by the relative stiffness of the buried pipe and the surrounding soil. This empirical ratio therefore allows the shear plane method to be adapted to intermediate conditions between those of complete positive (trench) and negative (projection) arching conditions.

### 3.2.1.3 Effect of Trench Width

Schlick (1932) describes a series of experimental studies to investigate the influence of trench geometry upon the load imposed on rigid pipes. Results showed that with all other conditions being equal, an increase in trench width resulted in an increase in the predicted pipe load, in accordance with the narrow trench theory, until it equalled that calculated using the positive projection theory. The width at which there is a transition between the two load theories is not constant and was shown to vary between 2.5 and 5 times the pipe diameter. When the proposed trench width approaches this transition value the lesser of the two predicted loads should be adopted. It was also concluded that the equation for a narrow trench could be adopted for trenches with a sloping or stepped sides provided that the width of the trench \( B_d \) was measured at the level of the pipe crown.

### 3.2.1.4 Effect of Uniform Surcharge of Large and Limited Extent

Marston (1930) stated that the load on a pipe \( W_c \) installed in a narrow trench condition and subjected to the application of a surcharge pressure of large extent, \( U_s \) per unit area can be found by modifying equation 3.1 presented in Section 3.2.1.1 to:-

\[
W_c = \gamma B_d^2 \left[ \frac{1 - \exp\left(-2K\mu' \frac{C}{B_d}\right)}{2K\mu'} \right] + B_d U_s \exp\left(-2K\mu' \frac{C}{B_d}\right) \quad \text{(Eqn. 3.5)}
\]

As Section 2.2.3 described, increasing the depth of pipe burial diminishes the effect of the surcharge loading. For a positive projection condition, Marston stated that the effect of a surcharge loading applied above the plane of equal settlement will be the...
same as an additional layer of fill material. To determine $C_c$ from Figure 3.3 for the effect of the backfill material and the surcharge loading, the value of $C$ becomes the height of fill, $C + U_j/\gamma$. It is assumed that for a complete installation, where the height of the plane of equal settlement exceeds the cover height, that the frictional characteristics of the surcharge are the same as those for the main fill. From Figure 3.3 it is clear that the limiting applied stress condition described in equation 2.2 may arise with a flexible pipe installation in a complete trench condition as the value of load coefficient $(C_c)$ tends to a single value as the ratio of $C/D$ increases. The similarities between the trapdoor experiments and the relationships for the load coefficients $C_d$ and $C_c$ indicated for the positive arching case, are also shown for a complete projection condition by the increase of negative arching with increased depths of cover.

To determine the vertical stress on a buried pipe from a concentrated surface load Boussinesq's (1885) analysis is typically used. This analysis describes the vertical stress distribution in a semi-infinite homogeneous elastic material as a result of the application of a surface point load. Application of this analysis to determine the influence of vehicle loading on buried pipes involves a number of approximations. These include the assumption that the tyre-area is a point load, which only significantly affects the calculated vertical stress when the depth of burial is less than 1m. More importantly, the road pavement (where present) is a non-homogeneous arrangement of materials that aim to achieve a high degree of load spreading, and this tends to reduce the applied stress on a pipe thereby introducing a factor of safety into the design. The resulting stress is added to that for static loading and, as Figure 3.5 indicates, maybe a critical loading condition for a shallow pipe.

### 3.2.2 Iowa Formula for Flexible Pipes

Spangler (1941) describes a hypothesis for the design of flexible buried pipes based upon measurements of vertical and lateral stresses taken during field experiments conducted using corrugated steel pipe culverts subjected to soil embankment conditions. The basis for the hypothesis was the assumption that the buried pipe deforms to an elliptical shape upon the application of vertical loading. Spangler suggested that Marston's load theory for rigid pipes could be applied to flexible pipes.
assuming that the applied vertical load was uniformly distributed over the diameter of
the pipe. Equation 3.1 stated in Section 3.2.1.1 is correspondingly modified to:-

\[ W_c = C_d \gamma B_d \, D \]  

(Eqn. 3.6)

This is resisted by a vertical bedding reaction which was assumed to be uniformly
distributed, with a lateral extent defined by the bedding angle (\( \alpha \)). As the pipe deforms
it mobilises the lateral earth pressure of the sidefill adjacent to the pipe. Tests results
indicated that the lateral earth pressure at any point on the pipe could be found by
multiplying the recorded horizontal displacement by a constant defined as the modulus
of passive resistance, \( e_h \) (kN/m\(^2\)/m) Due to the negligible horizontal displacements at
the top and bottom 40° segments of a pipe, the derivation of the horizontal pressure
distribution was based upon a simple parabolic curve over the middle 100° segment of
the pipe. Figure 3.6 illustrates the assumed pressure distribution which was used to
derive the 'Iowa' formula for horizontal deflection, \( \Delta X \):-

\[ \Delta X = D_1 \frac{K_b \, W_c \cdot r^3}{E_p \cdot I + 0.061 \, e_h \cdot r^4} \]  

(Eqn. 3.7)

where:-

- \( D_1 \) - Deflection lag factor to calculate long-term pipe deflection
  (dimensionless)
- \( K_b \) - bedding constant (dimensionless)
- \( W_c \) - Vertical load per unit length of pipe (N/m) or kN/m
- \( r \) - Mean radius of pipe (m)
- \( E_p \) - Elastic modulus of pipe (N/m\(^2\)) or kN/m\(^2\)
- \( I \) - Second moment of area of pipe wall (m\(^4\))

Due to the difficulties of defining the modulus of passive resistance \( e_h \), Watkins and
Spangler (1958) modified \( e_h \) to \( E' \), where \( E' = e_h \cdot r \). The new parameter known as the
modulus of soil reaction was used to derive a modified version of the Iowa formula:-

\[ \Delta X = \frac{D_1 \, K_b \, W_c \cdot \frac{1}{r^3}}{E_p \cdot I \cdot \frac{1}{r^3} + 0.061 \, E'} \]  

(Eqn. 3.8)
Written in this manner, the three separate factors which affect pipe deflection can be seen as:

\[ \Delta X = \frac{\text{Load Factor}}{\text{Pipe Stiffness Factor} + \text{Soil Stiffness Factor}} \]  \hspace{1cm} (Eqn. 3.9)

Intensive research has been conducted since the publication of Spangler's original work to determine the value of \( E' \) for various sidefill materials. Howard (1977) presents the most comprehensive of these after back-calculating \( E' \) from 113 field installations covering a wide range of pipe and fill combinations, see Table 3.3. The horizontal diameter of the pipeline was measured numerous times prior to and immediately after construction of the backfill, and by assuming that the applied vertical load was equal to the vertical soil prism above the pipe a value of \( E' \) was calculated. These values are not applicable to heights of fill greater than 15m, although Howard states that back-calculated values of \( E' \) from high fills are much greater than those reported in Table 3.3. The subsequent adoption of these values in design guides has terminated the use of Marston's load theory, with the applied vertical load assumed by designers to be equal to the column of soil above the pipe, with the width equal to the pipe diameter.

The limitations of the Iowa formula have been cited by many authors, including Rogers (1985). Importantly, the pressure distribution shown in Figure 3.6 was based upon the assumption of an elliptical pipe deformation. The deviation from an elliptical pipe response is frequently observed, notably by Rogers (1988) who explains how the resulting pipe shape is influenced by the nature of the soil surround. The assumption of an elliptical response is also used for the determination of the vertical deflection, which is defined in the design guides as a performance criterion, from the calculated horizontal deflection. Whilst this method is technically less correct than for example the TRL method described in the next section, it is still widely used. This is due primarily to its simplicity, as only a single empirically derived parameter (\( E' \)) needs to be determined. Due partly to the work by Howard (1977) and experience of use, this can be assessed reasonably accurately and it is this factor which is the key to the success of any predictive buried pipe deflection method.
3.2.3 Transport Research Laboratory (TRL) Design Method

In 1977, the TRL (formerly the Transport and Road Research Laboratory, TRRL) commissioned the development of a new design method for flexible pipes to unify the previous independent design approaches required for limiting deflection (e.g. Spangler, 1941), buckling (e.g. Meyerhof, 1963) and ring compression failure (e.g. White and Layer, 1960). A comprehensive description of this method is described in Gumbel (1983).

The approach adopted by Spangler (1941) modelled the soil support provided to the buried pipe as a series of discrete springs. This required a statically indeterminate pressure distribution at the pipe-soil interface to be estimated, from which an equation to determine the horizontal pipe deflection was derived. The development of subsequent design methods by for example Bossen (1967) and Molin (1971), have only fundamentally deviated from that described by Spangler in their assumptions of this pressure distribution. By adopting the elastic continuum method described earlier by Hoeg (1968) (see Section 2.3.2), the soil is considered a part of the structural system, thereby eliminating the need to estimate this pressure distribution. As Hoeg (1968) also highlighted, this allows the load transfer condition at the pipe-soil interface to be determined.

The structural properties of the pipe-soil system are defined in terms of elastic moduli and Poisson's ratio of the pipe and soil \((E_p, \nu_p, E_s, \nu_s, \text{ respectively})\) to provide the following four parameters:

\[
E_p^* = \frac{E_p}{(1 - \nu_p^2)} \quad \text{(Eqn. 3.10)}
\]

\[
E_s^* = \frac{E_s}{(1 - \nu_s^2)} \quad \text{(Eqn. 3.11)}
\]

\[
S_r = \frac{E_p^* I}{D^3} \quad \text{(Eqn. 3.12)}
\]

\[
S_c = \frac{E_p^* A}{D} \quad \text{(Eqn. 3.13)}
\]
where \( D \) is the pipe diameter, \( A \) is the area of the pipe cross section and \( I \) is the second moment of area of the pipe wall. Adopting the approach established by Burns and Richards (1964), the pipe-soil interaction parameters are also defined for flexural and compressive stiffness by the ratios, \( Y \) and \( Z \) respectively:

\[
Y = \frac{E_s^*}{S_f} \quad \text{(Eqn. 3.14)}
\]

and

\[
Z = \frac{E_s^*}{S_c} \quad \text{(Eqn. 3.15)}
\]

The flexural stiffness ratio (\( Y \)) was used to define the range of behaviour of a pipe-soil system, see Figure 3.7. This shows that a rigid pipe system is defined by values less than 10, where over 90% of the load is carried in bending, whilst values greater than 1000 define a flexible pipe system in which 10% or less is thus carried.

The external loading on the system is represented by uniform vertical and horizontal pressures, \( p_v \) and \( p_h \), which are defined as the free-field soil total stresses at the mid-height of the pipe. The lateral pressure ratio, \( K_d \), relates the free-field horizontal pressure, \( p_{hd} \), and the vertical pressure, \( p_{vd} \), due to the dead load as:

\[
p_{hd} = K_d \ p_{vd} \quad \text{(Eqn. 3.16)}
\]

and is a function of the soil properties, method of construction, degree of compaction, pipe stiffness and proximity of the trench walls. Gumbel et al (1982) recommends provisional values of \( K_d \) which for uncompacted backfills ranged from fully active (\( K_a \)) to a value at rest (\( K_o \)). For compacted backfills, they considered that the lowest probable value corresponds to the value at rest (\( K_o \)). Values of \( K_d \) approaching unity imply more uniform loading conditions and values approaching zero more distortional conditions.

The contributions to \( p_v \) and \( p_h \) due to backfill weight, ground-water pressure and uniform and concentrated surcharges are all calculated by standard techniques, see Figure 3.8. To account for the point of application for each type of loading, two load boundary conditions were defined as either loading applied at a distant boundary, the first load case, or loading applied at the pipe-soil interface, the second load case, as the theoretical solutions describing them were stated by Burns and Richards (1964) and
Curtis (1979), respectively. The loads on the system were separated into uniform and distortional components of pressure, \( p_z \) and \( p_y \), respectively, defined as:

\[
p_z = \frac{1}{2} (p_x + p_h) \quad \text{(Eqn. 3.17)}
\]

and

\[
p_y = \frac{1}{2} (p_x - p_h) \quad \text{(Eqn. 3.18)}
\]

The significance of this division is that ring deflection is caused primarily by the distortional, or out-of-balance, component \( (p_y) \) whereas the mean hoop thrust, which governs buckling, depends on the uniform pressure component \( (p_z) \).

The first and second order response of the pipe to the uniform and distortional load components are shown in Figure 3.9. The first order response, denoted by the suffix ‘1’, is defined when either pressure component acts alone, and the second order response results when they act together. The total ring deflection \( (\delta) \), total thrust \( (N) \), bending moment \( (M) \), are products of the uniform and distortional load components and are defined as:

\[
\delta = \delta_z - \delta_y \cos 2\theta \quad \text{(Eqn. 3.19)}
\]

\[
N = N_z + N_y \cos 2\theta \quad \text{(Eqn. 3.20)}
\]

\[
M = M_y \cos 2\theta \quad \text{(Eqn. 3.21)}
\]

where the subscripts \( y \) and \( z \) indicate distortional and uniform components of response respectively.

The uniform pressure acting alone produces a uniform hoop thrust \( (N_z) \), in the pipe wall and a uniform diametral strain \( (\delta_z) \) defined as:

\[
N_z = \alpha \ p_z R \quad \text{(Eqn. 3.22)}
\]

\[
\delta_z = \frac{a \ p_z}{2 \ S_z} \quad \text{(Eqn. 3.23)}
\]

where the coefficient \( \alpha \), termed the arching factor, represents the proportion of \( p_z \) which is carried by the pipe and produces a compression in the pipe ring. In design, the uniform deflection component \( (\delta_z) \) is typically small compared to the out-of-round deflection component \( (\delta_y) \) which comprises:

\[
\delta_y = \delta_{yo} + \delta_{y1} + \delta_{y2} \quad \text{(Eqn. 3.24)}
\]
The $\delta_{yo}$ term represents the initial pipe out-of-roundness due to the backfilling of the soil to the level of the crown, whilst $\delta_y^*$ is due to the external loading and comprises the first and second order components ($\delta_{y1} + \delta_{y2}$). The first order deflection ($\delta_{y1}$) is due to the distortional component ($p_y$), acting on its own, which also results in a hoop thrust of ($N_{y1}$), where:

$$\delta_{y1} = \frac{4p_y}{108S + E_s^*}$$  \hspace{1cm} (Eqn. 3.25)

$$N_{y1} = \beta p_y R$$  \hspace{1cm} (Eqn. 3.26)

The $\delta_{y1}$ expression is similar in form to the Iowa formula, see equation 3.8, with the important difference that the numerator corresponds to twice the difference between the vertical and horizontal pressures as opposed to the vertical crown pressure. The distortional thrust coefficient ($\beta$) is determined by the shear stress conditions at the pipe-soil interface, and lie between the extremes of no and full slippage.

To simplify the theoretical model, and allow the development of design charts, Gumbel (1983) proposed that all the distortional load components should be applied at a distant boundary (first load case), whilst all the uniform load components should be applied at the pipe-soil interface (second load case). To convert a second to a first load case solution, thereby maintaining continuity, it was necessary to apply a factor to all of the uniform load components. This factor, known as the magnification factor, was defined as:

$$\lambda_z = \frac{2}{1 + \nu_s^*}$$  \hspace{1cm} (Eqn. 3.27)

where $\nu_s^*$ is the plane strain Poisson’s ratio for the soil. It should be noted that the magnification factor is, after the correction for plane strain, the reciprocal of equation 2.7, see Section 2.3.3. The magnification factor is used to determine the arching factor ($\alpha$):

$$\alpha = \lambda_z \alpha_z$$  \hspace{1cm} (Eqn. 3.28)

where $\alpha_z$ is the uniform thrust coefficient and is dependent upon the compression stiffness ratio ($Z$), varying from zero when $Z$ is infinite (e.g. rigid pipe in soft soil) to unity when $Z$ is zero. Adopting a value of $\nu_s^* = 0.3$, $\lambda_z$ equals 1.5 which offsets the frictional arching within the soil, resulting in an assumed value of $\alpha = 1$.  

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The analytical expressions for out-of-round deflection and buckling pressure were combined to obtain relationships between five non-dimensional parameters:

\[
\frac{p_y}{\alpha p_x} \quad \frac{p_y}{S_t} \quad \frac{p_y}{E_s} \quad \text{and} \quad Y
\]

Design charts which provide a complete description of the combined deflection and buckling response of any pipe-soil system were prepared for a range of values of \(p_y/\alpha p_x\) from 0.05 to 0.8. An example of a design chart for a value of \(p_y/\alpha p_x = 0.1\) is shown in Figure 3.10, onto which contours of factors of safety against buckling have been added.

After developing the method, Gumbel acknowledged that its shortcomings were a lack of experimental evidence to validate the rationale of the design method and, importantly, provide back-calculated values of \(E_s*\) and \(K_d\). In addition, the method also omits the influence of the trench wall friction on the load calculation. Laboratory tests were conducted by the then TRRL to validate the design method and have been described by Crabb and Carder (1985). Rogers et al (1995) also attempted to compare the effectiveness of this method with deflections measured during a series of experiments on buried plastic pipe. The satisfactory agreement of the method with the TRRL experiments led to its adoption and subsequent use in the development of the UK specification for highway drainage advice note HA40/89 (DoT, 1990) which specifies allowable installation conditions for pipes for highway drainage.

3.3 FIELD BURIED PIPE STUDIES

3.3.1 Trench Conditions

In the early 1970s the then TRRL began a general programme of research into the behaviour of underground pipes. They assessed that there was a lack of detailed experimental information regarding the behaviour of buried pipes under static and traffic loading, and that the Iowa formula, see Section 3.2.2, resulted in a conservative prediction of the horizontal deflection of a flexible pipe. The completion of this programme was marked by publication of separate design guides for buried rigid and flexible pipes by Young and O'Reilly (1983) and Smith (1991a and b) respectively.
based upon Marston's load theories and Gumbel (1983) respectively. A part of this programme was a field trial described by Trott and Gaunt (1972) conducted on a section of 1.83m diameter steel pipe, of wall thickness 12.7mm, buried beneath a minor road. The pipe was laid in a trench 2.75m wide and surrounded by well compacted sand.

Placement and compaction of the sidefill resulted in the recording of a negative diametral strain of 0.2% as the vertical pipe diameter elongated. This type of behaviour has been observed by many authors, including Rogers et al (1996), and is due to the pipe crown being vertically unrestrained as lateral earth pressure is exerted on the pipe during sidefill compaction/placement. Subsequent application of a surcharge stress of 38kN/m² resulted in a relative decrease in the vertical diameter of 0.2%, indicating that a well compacted surround can reduce deflections to a negligible level.

Distributions of radial soil pressure around the pipe wall shown in Figure 3.11, indicate that when the backfill was level with the pipe crown a negative pressure was recorded at the pipe invert and a maximum positive pressure was recorded at the pipe haunch. Subsequent application of a maximum surcharge of 38kN/m² from traffic loading altered this to an approximately uniform distribution. The vertical soil pressure distribution over the pipe crown (see Figure 3.12) shows that the maximum and minimum pressures were recorded above the sidefill and pipe crown positions respectively during both backfill and surcharge loading. This non-uniform vertical pressure distribution indicates the occurrence of positive arching that would be induced during the vertical deflection of the pipe. The load spreading effect of the road pavement can be observed as the predicted values of pressure at the pipe crown using the Boussinesq analysis were larger than those recorded. Comparisons between the measured pipe deflections and those predicted using the Iowa formula were subsequently shown to be poor. This was due to the contrasting loading conditions between this test, performed using a relatively shallow pipe placed within a trench and subjected to static traffic loading, compared to the soil embankment loading condition used by Spangler, resulting in a higher than recommended value of E' for the compacted sand.
Trott and Gaunt (1976) describe a further field trial in which the performance of buried pipes was measured under a major road subjected to construction and road traffic loading. Four experimental pipelines were constructed to assess the behaviour of 300mm diameter PVC-U, asbestos cement, vitrified clay and steel pipes (Figure 3.13). Each of the pipes was installed in a 0.75m wide trench surrounded by a granular soil envelope to a total cover depth of 1.2m. Pipe wall strains were measured in all except the PVC-U pipe, together with horizontal and vertical deformations for the two flexible PVC-U and steel pipes. Soil pressures were measured above all of the pipes and additionally at the sides of the PVC-U and steel pipes. Loading of the pipelines by construction traffic consisted of scapers passing over the experimental site between 70 and 100 times, whilst road traffic was simulated using three types of vehicles.

Figures 3.14 and 3.15 present the pipe deflection and soil pressure adjacent to the PVC-U pipe during installation and over a period of 31 months of traffic loading respectively. The largest change of vertical and horizontal deflection occurred during the compaction of the first layer of the backfill, whilst placement of subsequent layers caused relatively smaller deflections. By contrast, Figure 3.15 shows an approximately linear increase of vertical pressure at the pipe crown as the backfill is placed whilst low magnitudes of horizontal pressures are recorded at the pipe springline indicated little passive pressure has been developed.

The passage of the loaded scapers over the pipeline caused a large increase in vertical deflection from 1.0% after the backfilling operation to 2.8%, and a corresponding increase in the horizontal deflection from 0.8 to 2.4%. The permanent and transient deflections recorded during this load sequence (Figure 3.16) illustrate that most of the permanent deflection was caused by the first ten passages. Subsequent passages are shown to have had little effect on the permanent deflection, whilst diminishing the magnitude of the elastic response. Figure 3.14 shows that further and reduced magnitudes of loading due to the construction and trafficking of the road pavement did not alter the recorded pipe deflections from the values established during the passage of the scapers.
Figure 3.15 shows that, unlike the measured deflections, the measured pressures continued to rise throughout the passage of the scapers. Thereafter the horizontal pressure increased throughout the construction of the road pavement, whilst the vertical pressure remained approximately stable. This redistribution of pressure around the pipe would account for the stable deflections observed in Figure 3.14 after the passage of the scapers, as the pipe deformed towards a state of 'ring compression' from the earlier 'ring bending'.

The deflection measurements taken from the steel pipe mirrored those recorded in Figure 3.14, for the PVC-U pipe. Internal circumferential pipe wall strains at the invert and crown of the clay pipe showed that a maximum tensile strain of $127 \mu e$ was recorded during the passage of the scapers, the pipe relaxing soon after this loading had finished.

### 3.3.2 Embankment Conditions

Spannagel et al (1974) described a field trial to assess the behaviour of two flexible, 2.75m diameter corrugated steel pipes placed in a positive projection condition beneath a high embankment (Figure 3.17a). The pipes were placed 1.2m apart on a shaped bedding in a 7.3m wide by 2.4m deep trench with sloping sides. Well graded, granular material was placed to a height of 0.3m above the pipe crown prior to placement of the embankment fill which comprised of clayey, sandy gravel. One of the pipe was instrumented at two locations along the pipe length: at Station D at the centre of the embankment covered by 48.8m of cover, and at Station E under a sideslope with 20.7m of cover.

The instrumentation layout is shown in Figure 3.17 b and c. Changes in the pipe diameter were recorded by measuring the distances between steel spheres attached to the inside face of the pipe wall. Eight pressure cells were placed radially around the circumference of the pipe, 0.150m from the external pipe wall, to measure the soil pressure acting normal to the pipe. Electrical resistance strain gauges were attached to the internal and external pipe wall at these locations to record the longitudinal and
circumferential strain. Three pressure cells were installed 4.90m above the pipe to measure the vertical stress above the pipe crown and the sidefill. Settlement of the embankment was measured at 13 locations using sealed, fluid level settlement platforms (see Figure 3.17c).

The change of horizontal and vertical pipe diameter during and after embankment construction is shown in Figure 3.18. This shows that placement of the granular surround to the level of the pipe crown resulted in a decrease of the horizontal diameter and a corresponding increase in the vertical diameter. Placement of the embankment material above the level of the pipe crown subsequently caused a shortening of the vertical diameter, and a corresponding increase of the horizontal diameter as the pipe mobilised the passive pressure of the sidefill material. The influence of the stiffness of the sidefill material is also demonstrated as the rate of change of the pipe diameter was greatest during the initial stages of embankment construction and decreased as construction neared completion.

By contrast, a linear relationship of measured crown pressure with fill height was shown, see Figure 3.19. Through continued monitoring a further 40% increase of crown pressure was recorded 12 months after the end of construction. Based upon the settlement readings, the predicted pipe crown pressure using Marston’s load theory was shown to correlate well with that measured during the construction period, whilst the corresponding increase on completion failed to be predicted. In addition, an improved correlation between the measured uniform pressure distribution around the pipe circumference was indicated by the ring compression theory outlined by White and Layer (1960).

A field study of the performance of a 610mm diameter corrugated High Density Polyethylene (HDPE) pipe placed beneath a 29m high soil embankment was described by Adams et al. (1989). The results from this study mirrored those described above by Spannagel et al. (1974) and in addition to the laboratory study described by McVay and Papadopoulos (1986) (see Section 3.4), illustrate how a buried flexible pipe behaves when placed in a positive projection condition.
The pipe was installed in a 1.8m wide by 1.5m trench excavated into the embankment material. The trench material consisted of well compacted crushed limestone, whilst the adjacent embankment soil was a well compacted silty clayey sandy gravel. On completion of the trench backfill, an additional 2.3m thick layer of embankment soil was placed to form a total cover of 3.0m. The remaining 26m of the embankment consisted of blasted rock fill, placed and compacted in layers less than 0.9m thick.

Inductance coils were used to measure the circumferential shortening of the pipe wall in addition to measuring the pipe wall strain at the pipe springline. Pipe diameter changes were recorded using radial extensometers. Pressure cells placed within the backfill, adjacent to the pipe wall were used to measure the soil normal pressure at the pipe springline and crown positions. Further inductance coils were placed in the embankment to measure vertical compression of the earth fill at the elevation of the pipe.

The behaviour of the pipe due to the embankment construction, thereby after the placement of the sidefill to the level of the pipe crown, is shown in Figures 3.20 to 3.22. Figure 3.20 shows that during embankment construction the decrease of the vertical pipe diameter was approximately linear, whilst an increase of the horizontal diameter occurred until a fill height of 5m, whereafter it remained constant. The contrasting behaviour between the vertical and horizontal pipe deflections is shown by the recorded vertical and horizontal diametral strains at the maximum fill height of 29m, of 3.8% and -0.6% respectively. Figure 3.21 shows that the percent of circumferential shortening increased approximately linearly during embankment construction which contrasts with that shown at the springline locations. The measurements of pipe deflection and wall strain therefore indicated that the pipe deformed in bending for the first 5m of embankment construction, whereafter the pipe deformed due to ring compression.

A complicated arching pattern was observed during this field trial primarily due to the high relative stiffness of the crushed limestone, which was used as the trench material,
with respect to the adjacent buried pipe and embankment material. This is shown in Figure 3.20 by the measurements of the average vertical soil strain of the embankment and the trench materials, at a fill height of 29m, stated as a compression of 3.6 and 2.2% respectively, compared to the reduction of the vertical pipe diameter of 3.8%. It can be concluded that negative arching was induced within the trench material as the embankment material settled with respect to the trench material. Figure 3.22 shows, however, that the measured vertical stress on the pipe during embankment construction indicates that 80% positive arching was induced by the pipe. These results highlight that whilst there was an increase of stress on the trench material due to the negative arching between the embankment and trench, the relative compressibility of the pipe with respect to the trench material meant that positive arching was induced between it and the trench material.

Havell and Keeney (1988) described a ten year long field study recording the load imposed by a 3.7m high, loosely placed sandy clay backfill on a 200mm diameter vitrified clay pipeline constructed in a positive projection embankment condition above the water table. The instrumented section was not subjected to vehicular loading. Three 1.5m long tests sections were placed on 120° concrete cradles, supported by load cells placed on a concrete slab. Settlement of the embankment was measured by using compaction gauges placed 0.3, 1.5, 2.3 and 3.7m above the pipeline.

Figure 3.23 shows that the measured imposed load on the pipe increased from an initial backfill load of 19.6kN/m to 24.0kN/m one year later. Over the following four years, the load increased by 1.93kN/m/yr, decreasing to 0.7kN/m/yr for the second five years. The imposed load had not stabilised ten years after construction of the pipeline. At this stage, a good correlation was shown between Marston’s load theory and the measured loads but only the predicted load at the spigot is shown in Figure 3.23. These predictions were based upon the measured settlements to derive a settlement ratio (see equation 3.4) of unity and a saturated unit wet for the mixture of soils in the backfill.
3.4 LABORATORY BURIED PIPE STUDIES

Hoeg (1968) describes a series of laboratory based buried pipe tests which were conducted to verify the development of theoretical solutions described in Section 2.3.2. The tests were conducted within a circular rigid soil box, measuring 0.9m in diameter by 0.7m deep, using an air bag arrangement to apply a uniform surface surcharge. The 114 mm diameter test specimens consisted of equally sized rigid segments supported on load cells (hence stresses) connected to an inner supporting ring. Varying the stiffness of this support altered the pipe stiffness. Uniformly sized sand was used throughout, placed into the test rig using a ‘raining’ technique.

The distribution of measured contact stresses were recorded for a rigid pipe (denoted as Test 1), and a supporting steel cylinder with diameter:wall thickness ratio of 80 encased in a flexible foam rubber (denoted as Test 4) (see Figure 3.24). The results from Test 1 indicate that for applied vertical and horizontal stresses of 827 and 290kPa respectively, the recorded contact stresses at the crown and springline positions were 1172 and 297kPa respectively. These results indicate the occurrence of negative arching as an increase of contact stress was recorded at the pipe crown whilst the stress at the springline remained constant. Applying his own derived theoretical solutions, Hoeg showed that the closest predictions were obtained when the boundary condition of no slippage was assumed, thereby permitting the development of shear stresses.

The occurrence of positive arching was conversely recorded during Test 4 as the contact stresses indicated hoop compression and a crown pressure of only 83kPa compared to an applied vertical stress of 1034kPa. A comparison of the predicted and recorded contact stresses indicated that the prediction based on the free slippage boundary condition was similar to the recorded values, whilst those based on the no slippage condition over-predicted the stress on the sides and underpredicted the value recorded at the crown (see Figure 2.14). The use of a free slippage boundary condition in elastic theories to predict pressures around flexible pipes has also been experimentally verified by Sargrand et al (1995). Additional tests showed that under an applied surface surcharge the contact pressure at the crown of a rigid pipe was shown
to increase with increasing depths of cover. By contrast, for flexible pipes the crown pressures decreased as the depth of cover increased, see Figure 3.25.

The mathematical formulation described by Hoeg implied symmetry about the horizontal axis of the buried cylinder. The experimental work described by Luscher and Hoeg (1964) showed that similar contact stresses were recorded at the pipe crown and invert positions when the surround was placed parallel with the longitudinal pipe axis. By placing the surround in horizontal layers, thereby simulating the construction sequence of a buried pipe, Hoeg (1968) showed that stress concentrations were shown at the pipe invert, as there was a reduction of surround stiffness below the pipe springline due particularly to the difficulty of placing material beneath the pipe haunches. This limitation of the elastic theory developed by Hoeg has also been noted by Pender (1981).

Gaunt et al (1976) describe the use of a 5 x 3 x 3m deep test pit to conduct laboratory tests in which 300mm diameter PVC-U pipes were buried in a 1m wide trench. On completion of filling the test pit with well compacted sandy clay the trench was cut to form a cover depth of 0.75m. Two of the installations consisted of laying the pipe on the trench bottom and placing either well compacted or uncompacted sandy clay to the ground level. The remaining two tests used uncompacted gravel to form the bedding, sidefill and backfill zones, to a cover depth of 300mm, above which either compacted or uncompacted sandy clay was placed to the ground level.

Cyclic loading consisted of using five hydraulic actuators to apply a surface load to create a sinusoidal stress wave. Peak value loads of 5, 10, 20, 40, 60, 80 and 100kN were applied each for 100 to 200 repetitions, or until the pipe deflection had stabilised. Static loads were then applied for 6 hours by using the three central actuators acting together each applying 40, 60, 80 and 100kN. By installing pressure cells within the soil above the level of the pipe crown it was shown that the measured vertical stress at the pipe crown from the application of surface loads could be predicted accurately using Boussinesq analysis, described in Section 3.2.1.4.
Figure 3.26 shows the increase of permanent pipe deflections recorded during the cyclic and static load regimes. The effect of compacting the sidefill was clearly shown as the largest reduction in vertical diameter of 19% was recorded when the surround consisted of an uncompacted sandy clay, which contrasts with the smallest reduction of 2% when the same material was well compacted. It was concluded that the both cohesive and granular materials could be used as surround material for buried PVC-U pipes, provided that the cohesive material was thoroughly compacted.

Figure 7.27 shows the increase in vertical pipe deflection during the application of a constant static load over a period of 6 hours. A smaller increase of pipe deflection with time was recorded from both pipe tests surrounded with gravel compared to those surrounded with sandy clay. These results indicated that PVC-U pipes installed within cohesive surrounds exhibited greater relaxation than those installed in a granular surround. This is because gravel will not creep under these magnitudes of load, whilst the increase in pipe deflection in clay is primarily due to the dissipation of excess pore-water pressure generated during placement leading to consolidation settlements and increased imposed stress on the pipe.

By using both centrifuge modelling and static testing, Valsangkar and Britto (1977 and 1978) investigated the validity of the ring compression theory originally proposed by White and Layer (1960). These tests also assessed the degree of arching induced by a buried flexible pipe and the behaviour of these types of pipes under surface loading. By introducing a load reduction factor ($L_R$) to account for the redistribution of applied stress due to positive arching, the equation for the hoop stress in the pipe wall as defined by the ring compression theory was modified to:

$$\sigma_0 = \frac{L_R PD}{2t}$$

(Eqn. 3.29)

where:

- $\sigma_0$ = Hoop stress in the pipe wall (Pa)
- $t$ = Pipe wall thickness (m)
- $D$ = Diameter of pipe (m)
- $P$ = Applied vertical stress (Pa)
Each of the tests were conducted using an aluminium box, measuring 0.765 x 0.765 x 0.310m deep, and during the static tests a surface surcharge was applied using a compressed air bag arrangement. The pipe stiffness was varied by forming the 101.6mm diameter pipes from either 0.152mm thick steel or 0.354mm thick plastic. They were embedded in an uniform dense surround created by pouring the sand parallel to the longitudinal pipe axis. As Trott et al (1984) later stated, a limitation with this method of placement is that it does not simulate the construction activities which as Hoeg (1968) described previously resulted in an alteration in the behaviour of the pipe. Bending moments and hoop thrusts in the pipe wall were measured from strain gauges attached to the internal and external faces of the pipe wall at quarter distances around one-half of the pipe.

The 1977 paper describes how wooden blocks were employed to simulate the in-situ soil, thereby allowing the influence of the geometry of a trench or embankment installation condition to be assessed. Based upon tests using the steel pipe only, the results showed that the ring compression theory was valid for pipes under an embankment or a wide trench installation conditions but was not valid for narrow and battered trench conditions. During the narrow and battered trench condition tests relatively larger bending moments were induced than in the embankment condition tests as the line of hoop thrust lay outside the pipe wall. It would appear that this conclusion was due to the boundary conditions of the wooden blocks decreasing the lateral stiffness of the sidefill, a situation that in practice depends upon the properties of the in-situ soil.

The centrifuge model tests showed that the load reduction factor varied between a value of 0.46 for a narrow trench condition, as increased shear stresses were formed between the backfill and the trench walls, and 0.7, equalling the embankment condition, when the trench width exceeded four times the pipe diameter. The results described in the second paper (1978) indicated that a single load reduction factor implied from the results of the 1977 paper could not be adopted for a particular type of installation as the intensity of the surface loading, depth of cover and pipe stiffness all influenced the degree of arching.
McVay and Papadopoulos (1986) describe the use of centrifuge modelling to simulate both the construction and long-term behaviour of a large span corrugated steel culvert buried beneath an earth embankment in a positive projection condition. The authors cite a failure of a large span culvert that had been in service for over ten years. The reasons for this failure were stated as a combination of either consolidation of the earth embankment with respect to the granular surround of the culvert, thereby inducing negative arching, cyclic vehicular loading or environmental factors for example a change in temperature. The field trail described by Spannagel et al (1974), see Section 3.3.2, described an increase in soil pressure above the pipe crown of approximately 40%, 12 months after the end of construction (see Figure 3.19). To verify the laboratory results they were compared to those recorded by Selig et al (1979) for the Newtown Creek project. This case history was selected because the results indicated that there had been no influence from either vehicular loading or from environmental factors.

The geometry of the Newtown Creek project (see Figure 3.28) shows a 7.85m diameter culvert encased in a granular surround buried beneath a silty clay embankment to a total cover depth of approximately 7m. Long-term monitoring of the project continued until four months after the completion of the embankment, from which it was believed that the results had stabilised. The laboratory tests were conducted using a 254 x 305 x 254mm deep aluminium sided glass box on single walled model culverts fabricated from galvanised steel for spans varying between 131 and 157mm. Throughout each test the pore-water pressure in the cohesive backfill was measured. In addition, the vertical and lateral culvert deflections were recorded from LVDTs, and strain gauges were attached to the internal and external walls of the culvert at the crown, shoulder and springline locations. To model the frictional characteristics between the soil and the structure, smaller particle sizes were selected with similar shaped distributions to those used in the prototype at Newton Creek.

The measured vertical displacement of the culvert crown (see Figure 3.29) indicated good agreement between the results obtained from two model experiments and the
actual structure (prototype). During the placement of the backfill the culvert crown was shown to displace upwards, subsequently reversing this pattern of deflection upon placement of the fill material. Figure 3.30a shows the recorded vertical soil strains in the embankment material surrounding the model culverts at the end of construction. Adjacent to the culvert the measured vertical strains are shown to increase with depth due to overburden stress. The large vertical soil strain recorded above the culvert crown was accounted for by the minimal degree of compaction applied to the fill material during its placement.

The long-term behaviour was simulated by continuing the centrifuge tests until the excess positive pore-water pressure generated during the construction sequence dissipated and finally stabilised. The result of the consolidation process was to increase the thrust stresses at the crown and springline positions by 10 and 20% respectively. Figure 3.30b shows that the measured long-term vertical consolidation strains in the embankment material increased in magnitude with depth due to the larger generated excess positive pore-water pressures with increased overburden. It can be concluded therefore that undrained behaviour occurred in the short-term as excess positive pore-water pressures were generated in the embankment material as it was constructed. In the long-term, these pore-water pressures dissipated leading to consolidation of the cohesive embankment and subsequent negative arching between it and the structural fill. This increased the load imposed on the structural fill, and whilst positive arching occurred between it and the culvert, an increase of imposed load on the culvert resulted. The authors state that the soil strains were determined using a photographic technique, but unfortunately complete distributions of soil movements and the subsequent soil strains were not shown. The value of this information would have been to allow the complete behaviour of the culvert, structural backfill and embankment to be interpreted.

Shmulevich et al (1986) described a series of laboratory based tests examining the soil stress distribution around a 1133 mm diameter asbestos cement pipe of wall thickness 62mm (D/t=18). The tests were conducted in a 2.0 x 3.0 x 2.5m deep rigid soil box to simulate a wide trench installation, using either a uniform sand or Terra Rossa clay as
the backfill material. On completion of filling the soil box with the backfill material, a uniform surface surcharge was applied in increments of 50kPa up to a maximum value of 200kPa. Plane-stress transducers were mounted flush with the surface of the pipe to measure both the normal and tangential (shear) contact stresses at twelve evenly spaced locations around the pipe circumference at the mid-length section.

The results showed that the largest magnitude of shear stress was recorded at the pipe shoulder, contrasting with a reduced magnitude of opposite notation at the pipe haunches. A symmetrical distribution of normal stresses was recorded about the horizontal plane showing the largest magnitudes recorded at the pipe crown and invert positions, whilst negligible values were recorded at the pipe springline. Comparing the distribution of normal and shear stresses it was noted that the magnitude of shear stress at the pipe shoulder approached one half of the measured normal stress value.

Increasing the initial soil stiffness by compacting the surround material was shown to reduce the magnitude of negative arching as the normal stress at the pipe crown was reduced with a corresponding increase of normal stress at the pipe springline. The results highlighted the need to include the influence of shear stresses at the pipe soil interface. The use of Marston's load theory which omits the influence of shear stresses at the pipe-soil interface will therefore lead to an underestimation of the vertical load.

Rogers et al (1995) described a series of laboratory based experiments that examined the behaviour of buried HDPE pipes with respect to the UK DOT Highway Advice Note HA40/89 used for the design of non-pressure drainage pipes. This advice note was developed using the TRL method described in Section 3.2.3, assuming worst case loading conditions and accepting a long-term VDS limit of 5%. In addition, all non-pressure pipes are required to meet a minimum 50 year extrapolated stiffness of 1400Pa when tested in accordance with BSI (1989). This standard specifies the measurement of the vertical deflection of a pipe laterally unrestrained in a line load test arrangement during the application of a constant load over 1000 hours. The creep performance of the pipe was criticised as unrepresentative of an actual site installation.
The experiments were conducted on 100 to 300mm diameter HDPE pipes using a 1 x 1.1 x 1.0m deep steel reinforced soil box, whilst the 300 to 375mm diameter pipes were installed in a 1.5 x 1.8 x 1.5m deep steel reinforced soil box. Repeat experiments conducted using polyethylene sheets placed against the sides of the boxes indicated that the boundary effects were minimal. The complete soil installation consisted of either a loosely placed uniform gravel, or a well-graded river sand, either compacted or uncompacted in 150mm thick layers. Instrumentation consisted of strain gauges to measure the circumferential internal wall strain and three LVDTs mounted on a self-righting sledge to measure the change in horizontal and vertical pipe diameter. On completion of filling of the soil box, a cyclic and static loading sequence was applied to the soil surface using a pressurised water bag arrangement.

Figure 3.31 shows typical vertical and horizontal diametral strain measurements during the application of 1000 repetitions of a uniform surface surcharge of 70kPa. This clearly shows that during cyclic loading the rate of increase of both permanent VDS and HDS decayed exponentially and the magnitude of elastic deformation progressively decreased. This pattern of behaviour is similar to that described previously by Trott and Gaunt (1976) (see Figure 3.16), and indicates that a pressurised water bag arrangement can be used to simulate approximately the behaviour of a buried pipe subjected to traffic loading.

The results showed that a VDS greater than 5.0% was only recorded when a 300mm diameter pipe was installed with uncompacted sand, an installation that would be deemed to represent poor site practice, under high applied stresses. This contrasts with a recorded maximum VDS of 2.8% for an installation that simulated typical UK site practice using a loosely placed gravel surrounding a 300mm diameter pipe. The authors concluded that the current specification and design criteria used in the UK were conservative.

3.5 LONG-TERM BEHAVIOUR OF BURIED PIPE-SOIL SYSTEM
Under an applied stress all materials experience a reduction of stiffness with time. This reduction of stiffness with time is typically referred to as relaxation. Chua and Lytton
(1989 and 1991) proposed that provided the yield strain had not been reached, the change in elastic modulus with time for a material could be modelled by the power law relationship:

\[ E(t) = E_0 t^m \]  
(Eqn. 3.30)

where:

- \( E(t) \) = relaxation modulus (Pa)
- \( E_0 \) = Initial modulus measured at one minute following application of the load to a test piece (Pa).
- \( t \) = time (minutes)
- \( m \) = constant (dimensionless)

Typical values of the power law constant \( m \), for time in minutes, were stated by these authors for pipes manufactured from HDPE, PVC-U and concrete as 0.083, 0.031 and 0.028 respectively, and sand and clay as 0.020 and 0.100 respectively. The relationship of the relaxation modulus (equation 3.30) shows that larger values of \( m \) result in greater decreases of modulus with time.

The increase or decrease of applied stress on a buried pipe with time depends upon the difference between the relaxation rate \( m \) of the pipe material and the surrounding soil. From the values of \( m \) stated above, a granular surround to a rigid pipe will typically creep faster than a rigid pipe, increasing the soil load on the pipe over time. An example of this behaviour was illustrated by Havell and Keeney (1988) (see Figure 3.23) where the imposed load on a buried clay pipe surrounded by sandy clay increased with time. The converse of this will generally occur for a flexible pipe surrounded by a granular material, as the pipe material relaxes faster than the soil, redistributing the imposed load onto the soil over time. The pattern of soil stress redistribution induced by the application of a soil overburden stress therefore progresses in the long-term. These patterns of behaviour will not however, typically occur for a pipe buried in clay.

As the high exponent value of the power law for clay indicates \( m=0.1 \), the consolidation settlement of clay is greater than the creep rate of all the pipe materials and sand stated previously. This will result in an increase of load applied directly to the buried structure or to a granular surround when clay is used as an embankment material, as the examples by McVay and Papadopoulos (1986) and Barrett et al (1994)
show for buried culvert and tunnels respectively (see Figures 3.30 and 3.38 respectively).

To calculate the increase of pipe deflection with time for a flexible pipe, the Iowa formula was modified to include an empirically derived deflection lag factor \( D_l \), (see equation 3.8). The value generally used is 1.5, which when applied to the weight of the soil prism load, plus any additional dead surface surcharge, increases the applied overburden soil stress on the pipe. In addition, the relaxation of the pipe material has also been accounted for by the use of long-term pipe stiffness parameter, which was described by Rogers et al (1995). By using the relationship stated in equation 3.30, Chua and Lytton (1989) showed that the compressibility and flexibility factors \( C \) and \( F \) respectively in the elastic solution described by Hoeg (1968) could also be modified to predict long-term pipe-soil behaviour.

3.6 INTRODUCTION INTO THE BEHAVIOUR OF BORED TUNNELS
Section 2.3.3 described that the construction of a bored tunnel through a natural soil causes a redistribution of soil stress, notably a reduction of radial stress at the tunnel crown and an increase of tangential stress at the tunnel springline. This redistribution results in a convergent soil displacement pattern and is only arrested by the construction of the tunnel lining. The absence of a permanent or temporary support will therefore result in gross soil displacements and eventually, soil collapse. The magnitude of soil displacement, and particularly the resulting angular strain, need to be limited during the construction of a tunnel beneath an urban area. Whilst the issues of tunnel lining design, tunnel face stability and induced ground movements are treated separately in design for simplicity, it is clear that they are all 'products' of the tunnelling process and are therefore all inter-related.

Sections 3.7 and 3.9 describe field and laboratory studies of soft ground tunnelling conducted in granular material. The results from these, and those conducted in cohesive materials, have led to the development of empirical relationships which are described in Section 3.8.
3.7 FIELD STUDIES OF BORED TUNNELS

3.7.1 Short-Term Behaviour

One of the first field studies that attempted to record the complete settlement pattern around a tunnel was described by Boden and McCaul (1974). Instrumentation at two sections along the tunnel alignment included the use of magnetic ring extensometers, inclinometers, and precise levelling of surface markers to record the surface and subsurface settlement pattern, and the use of piezometers to record the changes in pore-water pressure. The 4.12m diameter tunnel was constructed at New Cross in London through dense cohesionless soils, using an experimental bentonite tunnelling machine, at a depth of 10m.

Vertical settlements recorded at four instrumented sections along the longitudinal axis of the tunnel, and above the tunnel centreline, are shown in Figure 3.32 and indicate that the settlements were in phase with the construction of the tunnel. The readings from each station show that settlement was propagated before the tunnel face had reached the station and that the subsequently induced settlement was arrested once the tail of the shield had passed the station or when the construction of the tunnel had been halted. The vertical movements recorded at borehole number B2, above the tunnel centreline, see Figure 3.33, shows that a settlement of 23mm was recorded above the tunnel crown, increasing to a value of 27mm for a region 3 to 4m above the tunnel crown, then decreasing to a surface settlement of 21mm. The recorded increase of settlement above the tunnel crown, as opposed to the expected decrease of settlement with increasing height from the tunnel crown, was due to the dilation of the dense granular material.

The lateral movements recorded adjacent to the tunnel at borehole number C1, see Figure 3.34, show that the upper soil layers tended to settle in a convergent manner towards the centre of the settlement trough, whilst the ground around the tunnel face was displaced in an expansive manner by the passage of the shield and the associated slurry pressure. A similar observation was reported by De Moor and Taylor (1991), and is due to the overpressurisation at the tunnel face.
Hansmire and Cording (1985) summarised a comprehensive study of soil deformation and tunnel behaviour of two parallel tunnels constructed beneath Lafayette Park in Washington D.C. The 3.5m diameter permanent concrete lined tunnels were constructed using a shield arrangement in dry, silty sands and gravels at a depth of 13m beneath the instrumented section, denoted as Line C. The three-dimensional settlement pattern above the advancing tunnel was recorded, from which only the settlements recorded at the transverse direction at Line C are described. Sub-surface soil displacements were measured using inclinometers and extensometers, and the surface settlement was determined by precise levelling of reference points. Instrumentation of the tunnel lining consisted of attaching strain gauges on the steel ribs of the initial lining, and measurement of the diameter changes of the lining.

A summary of the final behaviour of the soil around the first tunnel is shown in Figure 3.35. It shows that a maximum surface settlement of 6in (150mm) was recorded, whilst the vertical settlement above the tunnel crown was 13in (330mm). The attenuation of settlement from the tunnel to the surface was reduced by the dilation of the dense sand, shown by the region of expansion extending from the lining crown. The soil movements are shown to be confined to a narrow region of soil, slightly wider than the width of the tunnel. This zone of movement was bounded by shear strains of 10% in the soil above the sides of the tunnel, leading to the conclusion that the soil displacements were initiated at the tunnel crown in a manner similar to that described by Terzaghi (1936), see Section 2.2.2.

The magnitudes of the induced shear strains led the authors to employ Terzaghi's shear-plane method (see Section 2.2.3), to calculate the imposed load on the tunnels, as elastic based solutions would under-estimate the degree of arching. The resulting ground-reaction curve for both tunnels indicates that the average measured loads were comparable to the shear-plane load for a rigid tunnel in sand (see Figure 3.36). The flatness of the ground reaction curve for these tunnels between points D and D' indicates that once sufficient soil movement had occurred, the lining load remained altered. This can be concluded as the measured lining loads for both tunnels were
comparable, whilst greater soil settlement were measured during the construction of the first, compared to the second tunnel.

Leblais and Bochon (1991) describe the soil and tunnel lining behaviour during the construction of the Villejust tunnels, to the South-West of Paris in France. Twin 9.25m O.D. tunnels were excavated using a slurry shield machine through dense fine uniform sand. Measurements of ground movements were taken at four profiles, denoted as Lines A, B, C and D, in the transverse direction to the centreline of the tunnel. Extensometers and inclinometers were installed at Lines B and C, whilst the surface settlement was recorded using precise levelling at each of the profile lines. Beneath these profiles lines, instrumentation of the tunnel lining consisted of strain gauges, ground pressure cells and internal diameter measurements. The depth of cover at these profiles lines varied between 6 and 47m, and only at Section A, the tunnel was constructed beneath the water table.

From the recorded surface settlement profiles, the calculated surface volume loss at each profile was very low, between 0.22 and 0.90% for the three profiles above the water table and 1.34% for the first tunnel constructed at Line A below the water table. The low surface volume loss was attributed to the face stability provided by the slurry pressure and the observed expansion of the dense sand above the tunnel crown.

The average lining load recorded immediately after construction of the first tunnel lining was comparable to those recorded prior to construction of the second tunnel, between 450 and 850 days later. The only increase in load with time was recorded at Line A, which was constructed beneath the water table but no details are provided concerning the changes in pore-water pressure to permit an interpretation. The influence of positive arching was clearly shown by comparing the measured lining thrusts with the overburden load. At Line B, with a cover depth (C) to lining diameter (D) ratio of 5, the imposed thrust was 22% of the overburden load, increasing to 40% of the overburden load at Line D, with a C/D ratio of 2.
3.7.2 Long-Term Behaviour

Due to the fast rate at which modern tunnels are constructed, an undrained analysis is typically assumed for tunnels constructed in cohesive materials and conversely, a drained analysis for tunnels constructed in granular materials. The results obtained from the case histories by Hansmire and Cording (1985) and Leblais and Bochon (1991) appear to support the assumption of a drained analysis for tunnels constructed in granular materials because the measured settlements and lining loads quickly stabilised after construction. Long-term monitoring of tunnels constructed in cohesive materials would aid the understanding of how the changes in pore-water pressure created during the construction process leads to consolidation and influences the resulting ground movements and tunnel lining loads.

O’Reilly et al (1991) describe the results of an eleven year long programme of monitoring surface settlements for a sewer tunnel constructed in very soft clay soil at Grimsby. Superposition of both the short and long-term transverse profiles showed that the long-term profile was wider and deeper than the short-term profile (see Figure 3.37). Most importantly, it was shown that the magnitude of angular distortions of the ground did not significantly alter after 7 days, indicating that the most critical period in which damage may occur to overlying buildings will be during, and immediately after, construction of the tunnel. A comparison of the final and short-term settlement readings illustrated that there was no simple relationship between the magnitude of the initial and time-dependent settlement profiles. The use of finite element analysis, as described by the authors, appeared to represent a method of calculating the long-term settlement as they were able to model the generated pore-water pressure distribution and relative permeability of the clay and the tunnel lining. The results from empirical relationships that do not model the above factors, for example those reviewed by Selby and Attewell (1989), must be treated with caution.

Barratt et al (1994) reports the results of long-term monitoring of the loads developed in instrumented concrete rings in the northbound tunnel of the Jubilee Line, beneath Regents Park in London and in the Oxford Trunk Outfall sewer. The depth from the ground surface to the tunnel axis was approximately 20 and 15.5m, and the
surrounding soil was London and Oxford Clay respectively. The results from Regents Park showed that the rate of increase of load decreased with time corresponding with the dissipation of excess pore-water pressure generated during the construction process (see Figure 3.38a). After 19.5 years, the vertical load at tunnel axis had reached the equivalent of 60% of the overburden. Similar observations were made at Oxford where the loads on the lining had built-up more erratically than those recorded at London to between 35 and 50% of the overburden after 7.5 years, at which time the readings were terminated (see Figure 3.38b). It can be concluded that these long-term readings were as a result of the ground movements induced during the construction process, whose pattern (therefore formation of shear stresses) does not significantly alter with time (see Figure 3.37). It was also concluded that ring loads on single tunnels constructed from cast iron or concrete linings will not exceed those corresponding to an all-round pressure equal to overburden.

3.8 EMPIRICAL GROUND MOVEMENT RELATIONSHIPS

3.8.1 Introduction

Field measurements recorded during and after the construction of a tunnel provide a valuable insight into the qualitative behaviour of the tunnel lining and the surrounding ground. Peck (1969), during his state-of-the-art review on soft ground tunnelling, illustrated that the recorded surface settlement from these studies could form a database of information from which empirical predictions can be made.

3.8.2 Surface Settlement - Transverse Plane

The surface settlement (W) at a perpendicular distance from the tunnel centreline (y) is defined by the error-function curve (see Figure 3.39):

$$W = W_{max} \exp \left( \frac{-y^2}{2i_y^2} \right)$$

(Eqn. 3.31)

where:

$$i_y = \text{trough width parameter in the transverse plane (y-direction).}$$

Surface settlement measurements above tunnels constructed in the UK through clays and sands were used by O'Reilly and New (1982) to propose that $i_y$ could be defined by the linear relationship:
\[ i_y = K Z \]  
(Eqn. 3.32)

where:-

\[ Z = \text{depth from the ground surface to the tunnel axis}. \]

and for practical purposes, \( K = 0.5 \) for clays and \( K = 0.3 \) for sands (Rankin, 1988, Mair et al, 1993 and Grant and Taylor, 1996).

The value of \( W_{\text{max}} \) is calculated by predicting the area of the surface settlement trough. By using previous case histories this quantity is related to the volume of the tunnel to determine the volume of soil 'lost' during the construction process. Settlements induced during undrained conditions can subsequently be determined by relating the volume loss at the tunnel to that at the surface. The potential discrepancy between the assumed volume loss at the tunnel and that measured at the surface during drained conditions has lead to the observation that predictions based upon the error function curve do not adequately correlate with the observed settlement. Attewell et al (1986) stated that for shallow tunnels constructed in dense sands and gravels dilation may occur through the arching action induced above the tunnel crown. The resulting region of expanding soil mass above the tunnel crown reduces the volume loss observed at the surface below that assumed to originate at the tunnel crown, leading to an over-prediction of the magnitude of the surface settlement. This behaviour was shown during the case histories described by Hansmire and Cording (1985) and Leblais and Bochon (1991). The converse of this may occur for tunnels constructed though loose sands as compression effects may develop, resulting in an increase in the volume loss at the surface, with respect to that assumed at the tunnel, leading to an under-estimate of the magnitude of the surface settlement.

3.8.3 Surface Settlement - Longitudinal plane
Ranken (1988), and New and O'Reilly (1991) stated that the typical shape of the settlement profile, parallel to the tunnel axis, can be represented by a cumulative normal distribution function. The basis for calculating the longitudinal settlement profile is to initially calculate the transverse settlement profile. It is then assumed that the longitudinal trough width parameter \( (i_x) \) equals \( i_y \) and that the origin of the
Cartesian co-ordinate system, vertically above the tunnel face, coincides with 50% of the maximum settlement, $W_{\text{max}}$.

### 3.7.4 Sub-Surface Settlement

Gunn (1993) stated that it is often assumed that the shape of the subsurface settlement profiles developed during tunnel construction are characterised by a Gaussian distribution, and for tunnels in clays, based upon using a reduced depth for the tunnel. The basis for this was the linear relationship between the trough width parameter ($i$) and tunnel depth ($Z$) stated in equation 3.32. O'Reilly and New (1982) however, stated that because of the simplified assumption that the soil movement occurs along radial paths towards the tunnel axis, inaccuracies may arise around the proximity of the tunnel.

Mair et al (1993) demonstrated that from field measurements of sub-surface settlements above tunnels in London clay in the transverse plane that the trough width parameter ($K$) increased with depth, giving proportionally wider settlement profiles closer to the tunnel. A relationship was proposed to account for this variation and should be used in place of $K = 0.5$.

### 3.9 LABORATORY MODELLING OF BORED TUNNELS

Cording et al (1976) describe a laboratory based technique using concentric tubes placed within a uniform sand to represent a ground loss at the tail void. At stages during the withdrawal of the outer tube (i.e. shield), photographs were taken from which a stereo-photogrammetry technique, described by Butterfield et al (1970), was used to determine the induced sand particle displacements.

Typical results of the soil displacements around the inner tube (i.e. lining) are shown in Figure 3.40. It shows a convergent settlement pattern with maximum displacements indicated close to the lining shoulder positions. At the surface, the narrow settlement pattern was shown by the recorded values of the trough width parameter ($K$), of between 0.14 and 0.26, which compares favourably with the recommended value of 0.3 (see Section 3.8.2).
As part of a comprehensive programme of research into the behaviour of tunnels, Atkinson and Potts (1977a) describe a series of static and centrifuge laboratory model tests performed using over-consolidated Kaolin and dry Leighton Buzzard sand. Tunnel collapse was simulated by deflating a 60mm diameter thin cylindrical rubber membrane. At stages during the deflation process the displacement of lead shot embedded in the soil was recorded, in addition to the membrane pressure. The measured surface settlement troughs above the yielding tunnels in both types of soil approximated to the error function curve and only departed from this relationship once the tunnel had collapsed. Examination of the measured crown and surface settlements ($W_c$ and $W_s$) respectively showed they were related to $C/D$ ratio by the relationship:

$$\frac{W_c}{W_s} = 1.0 - \alpha \frac{C}{D}$$

(Eqn 3.33)

where $\alpha$ is a constant. The appropriate values of $\alpha$ varied from a maximum of 0.57 for dense sand to a minimum of 0.13 for over-consolidated kaolin. It was observed to be dependent upon the volumetric characteristics of the soil and the changes of stress around the tunnel. This was highlighted by comparing the volumetric and shear strain distributions calculated from measured displacements observed during tests near collapse in a dense sand without surface surcharge ($\sigma_s = 0$), in dense sand ($\sigma_s = 0$, centrifuged at 75g) and overconsolidated Kaolin ($\sigma_s = 140kPa$) shown in Figures 3.41 a, b and c respectively. High concentrations of shear strains, with magnitudes up to 20%, were recorded between the crown and the springline positions for each of the experiments. This indicates that the soil in these regions had reached a state of limiting stress. The volumetric contour distributions showed that for the tunnels in sand, these regions of shear strain were associated with regions of dilation (see Figure 3.41 a and b). The dilation effects induced during the decrease of mean normal stress, associated with the reduction of tunnel pressure, therefore had minimal influence on the volumetric distribution. By contrast, the volumetric distribution observed around the tunnel in clays showed that whilst dilation effects would similarly be induced during the decrease of tunnel pressure, this was negated by the compression resulting from the shear strains between the crown and springlines.
Atkinson and Potts (1977b) described a theoretical analysis for the stability of tunnels constructed in sand using upper and lower bound theorems for perfectly plastic materials. The upper bound, or unsafe, value to the true collapse tunnel support pressure is calculated by selecting a collapse mechanism together with an appropriate work rate calculation, in which the external loads must cause collapse. The accuracy of the upper bound solution therefore depends upon the assumed collapse mechanism. The shear strain distributions described by Atkinson and Potts (1977a), (see Figure 3.41a) were used to select the collapse mechanism shown in Figure 3.42 which in turn was used to derive:–

\[
\frac{\sigma_T}{2\gamma R} = \frac{1}{4 \cos \phi} \left( \frac{1}{\tan \phi} + \phi - \frac{\pi}{2} \right)
\]

(Eqn. 3.34)

where \( \phi \) = maximum angle of shearing resistance and can be used provided that \( C/R \geq 1 / (\sin \phi - 1) \), to ensure that the apex of the sliding wedge at B is below the ground surface. Similarities exist between this failure mechanism and the ultimate imposed load on a trapdoor suggested by Terzaghi (1943), denoted by area 'aate' in Figure 2.1. In addition, the authors note that the upper bound collapse pressure is independent of the tunnel depth and the magnitude of any surface stress. The model tests were used successfully to validate the theoretically derived upper and lower bound solutions for the stability of an unlined tunnel.

A further application of centrifuge modelling was described by Nomoto et al (1996). The laboratory arrangement (see Figure 3.43) permitted the influence of the construction process of a bored tunnel to be assessed by performing 'buried lining', 'tail void' and 'shield' type experiments. The basic experiment consisted of burying a 96mm diameter lining tube within the container, a procedure that was subsequently adapted during the 'tail void' experiment to measure, additionally, the influence of withdrawing a 100mm diameter outer shield tube. The more complex 'shield' experiment consisted of initially using a miniaturised tunnelling machine at the face of the shield tube to form an excavation, after which the tail void test was repeated. The earth pressure acting on the lining tube was created by increasing the centrifugal acceleration of the model and was measured by eight load cells attached at even distances around the circumference.
of the lining tube. The surface settlement was measured by using a movable
displacement laser.

Figure 3.44 shows the relationship between the measured earth pressure acting on the
top side of the lining and cover depth for the three types of experiments conducted.
This shows that during the buried lining experiments the measured earth pressure on
the lining was shown to be greater than that of the overburden, indicating the
occurrence of negative arching. Withdrawal of the outer tube resulted in a reduction in
the imposed earth pressure on the lining to below that of the overburden, as the
measured convergent soil settlement pattern reversed the nature of the induced arching
stresses. The inclusion of an additional source of ground loss, by excavating the in-situ
soil prior to the withdrawal of the outer tube, increased the magnitude of the maximum
recorded surface settlement from 1.9mm, for the ‘tail void’ test, to 2.8mm, for the
‘shield’ test. The observed increase of surface settlement would have resulted in a
further propagation of arching stresses shown by the reduction of the imposed earth
pressure on the lining during the ‘shield’ tests below that measured during the ‘tail
void’ tests. Results of the shield tests are seen to compare favourably with the earth
pressures predicted by using Terzaghi’s trapdoor idea at cover depth to diameter ratios
greater than 2. Figure 3.44 also importantly indicates a similar relationship of measured
earth pressure against C/D ratio for buried pipes and tunnels corresponding to the
theoretical statements made in Section 2.3.3.

A recent innovation in the field of tunnelling has been the emergence of trenchless
pipelaying techniques, for example pipejacking. Chapman (1992) described a
laboratory based study into the ground movements associated with pipejacking using a
1.5 x 1.0 x 1.5m deep plane strain glass sided test tank. The tests were conducted
using open pipejacking shields with uniform overcuts of 10 and 20mm with respect to
the 200mm diameter pipe train. The open pipejacking shield was jacked along the line
of the drive in increments of 10mm, at which stage a set of photographs was taken.
Displacements of either uniform Leighton Buzzard sand or a well graded river sand
were recorded using the stereo-photogrammetry technique along the centreline and
perpendicular to the line of drive.
The general form of the ground movements associated with the open shield pipejacking tests in the longitudinal and perpendicular planes are shown in Figure 3.45. This figure shows that in the longitudinal plane, the soil in front and above the shield was forced away from the shield, the soil displaced into the tail void and the soil immediately preceding the open shield was 'swallowed up' as the shield was jacked forwards. The results highlighted that the maximum extent of the sand movement was only three diameters from the shield and this maximum generally occurred directly in advance of the shield. The effect of increasing the C/D ratio, thereby increasing the confining stress, was to reduce both horizontal and vertical movements in front of the shield, as the sand found it easier to move into the shield. The variation of measured displacements between experiments conducted in dense and loosely placed sand, were negligible in terms of extent of displacements, but the magnitude of displacements were slightly greater for the loose compared to the dense state tests, due to the greater capacity for compression.

The ground movements in the perpendicular plane showed that the sand displaced downward and inward towards the void left behind by the shield overcut. Increasing the overcut dimension resulted in an increase in the magnitude and extent of the displacements. The effect of increased density was to cause increased dilation rates within the sand, which restricted the movements to a finite level above the pipe, as opposed to the loose state where movements extended upwards to the surface.

### 3.10 GROUND BEHAVIOUR - TUNNEL LINING MODEL

A limitation of the conventional design methods is that they treat the prediction of soil settlement, tunnel support and lining stresses separately. By adopting this approach, Wong and Kaiser (1987 and 1991) argued that the assumed mode of ground behaviour may not correspond to that encountered in the field. These authors describe the development of a model aimed at characterising the mode of ground according to the tunnel support pressure and the coefficient of earth pressure at rest ($K_o$). The model is shown in Figure 3.46 and defines two main types of ground behaviour, denoted Mode I and II.
Section 2.3.3 stated that during excavation there is a decrease in the radial stress at the tunnel crown and an increase in tangential stress at the tunnel springline, which induces the creation of arching stresses. When these arching stresses exceed the ground strength, yielding will be induced with further decrease of support pressure \((P_i)\), see Line c-d. Prior to tunnel construction through a natural soil where \(K_o < 1\), the tangential soil stress exceeds the radial soil stress at the tunnel springline, whilst the reverse condition occurs at the tunnel crown. Excavation at the tunnel face, involving a decrease in the tunnel support pressure \(p_n\), causes the stress difference between the radial and tangential stresses to increase at the springline, initiating yielding (Mode I). Where \(K_o\) approaches unity, the stress difference between the radial and tangential stresses increases at the same rate resulting in a continuous yield zone surrounding the tunnel opening (Mode II).

After yield initiation continued reduction of the tunnel pressure leads to propagation of the yield zones, defined by the area bounded by points c, d, m and n. At low values of \(K_o\) further reduction of the tunnel pressure causes the yielding of the soil to extend from the pipe springline to the tunnel shoulder leading to yielding of the soil at the roof of the tunnel. A critical \(K_o\) value \((K_{cr})\) was defined to separate Mode I developing into mode I-1 or I-2, at \(K_o < K_{cr}\), from developing into mode II, at \(K_o > K_{cr}\). Once yielding was propagated all around the tunnel, a collapse mechanism forms at the tunnel roof, in the manner similar to that shown by Atkinson and Potts (1977b) in Figure 3.42. Line m-n defines the minimum support pressure \(P_{se}\) required to prevent the collapse mechanism occurring.

The relationship between the required support pressure and the modes of tunnel behaviour shown in Figure 3.46 were related to settlement by use of a ground reaction curve (see Figure 3.47). At point 1, the support pressure equals the initial overburden pressure. Reduction of the tunnel pressure causes elastic settlements until point 2, where yielding is initiated. Continued reduction of the tunnel pressure initiates further yielding (points 2 to 4) which can be prevented by application of the minimum support pressure \((P_{se})\). As the ultimate support pressure is greater than the minimum support
pressure, the authors state that the assumption of full overburden pressure at the roof is conservative unless time-dependent process such as consolidation dominate. Where these considerations do not occur, Terzaghi’s shear plane method should be used to calculate the ultimate support pressure.

Comparing the theoretical model with the experiments described by Atkinson and Potts (1977a) conducted in dense sand, where $K_o = 1 - \sin \phi$ is less than $K_{cr}$, and over-consolidated kaolin at $K_o = 1.0$ (see Figures 3.41a and c respectively) showed the occurrence of Mode I and II behaviour respectively. These experiments also indicated that larger settlements were observed for decreasing values of $K_o$. The authors state that at higher values of $K_o$, the degree of tangential arching increases, increasing the resistance to downward movement of the soil above the crown.

3.11 CONCLUDING DISCUSSION

A review of the literature was conducted and described in Chapters Two and Three to ascertain information about stress redistribution in soil surrounding a buried structure. This stress redistribution was shown to be due to the formation of shear stresses, referred to in this study as arching stresses, which, depending upon their orientation and magnitude, altered the magnitude of the stress imposed on a buried structure compared to that of the applied overburden stress.

Experimental evidence of stress redistribution was described in Chapter Two in the form of small-scale experiments conducted using idealised buried structures. These tests were categorised according to the manner in which the arching stresses were induced. The earliest study conducted by Terzaghi (1936) used an artificial mechanism, the trapdoor, to create the arching stresses within the overlying soil by displacing the trapdoor into, or away from, the overlying soil. The results showed that displacing the trapdoor away from the overlying soil caused a reduction of stress imposed on the trapdoor compared to that initially applied by the soil overburden. Conversely, displacing the trapdoor into the overlying soil was shown to result in an increase of imposed stress on the trapdoor to a magnitude greater than that of the initial soil overburden stress. This redistribution of stress above the trapdoor was explained in
terms of the formation of arching stresses extending upwards from the trapdoor into the overlying soil. Continued displacement of the trapdoor resulted in the soil yielding and the imposed stress on the trapdoor reaching an ultimate state. Importantly, the tests highlighted the importance of the soil cover depth. The imposed stress on a trapdoor that displaced away from the overlying soil decreased as the depth of soil cover increased, reaching a limiting value for depths in excess of two or three times the door width. It can be concluded therefore that the imposed stress on a trapdoor is independent of the state of stress in the soil at a distance of two or three times the door width above the pipe. In practice, this corresponded to the influence of a surface surcharge stress diminishing with soil cover depth, until at large depths of cover the imposed stress on a trapdoor equalled that of a small soil load.

From these results, Terzaghi (1943) proposed the theoretical 'shear-plane' method to predict the vertical stress imposed on a displacing trapdoor. The approach was based upon the development of failure planes extending vertically from the sides of the trapdoor to the surface of the overlying soil. Depending upon the direction of these shear planes and thus that of the trapdoor, the vertical stress imposed upon it could be predicted for positive and negative arching cases. Marston (1930) also adopted this approach to develop a theory to predict the external loads applied to buried pipes in narrow trench and embankment conditions. For the trench condition, the shear plane method was used to determine the frictional relief provided by the backfill material with respect to the natural soil. For the embankment condition, the shear plane method was used to determine the stress distribution induced by vertical shear stresses extending from the pipe springings to the ground surface. This approach was adopted to determine the imposed stress on the contrasting installations of a very flexible and a very rigid pipe, with respect to the surround. These two installations were referred to by Marston as complete trench and projection conditions respectively. An empirical constant defined as the settlement ratio was subsequently used to determine the imposed stress on a buried pipe with a pipe-soil stiffness between these two extremes of behaviour. The resulting analysis highlighted the importance of the relative stiffness between the pipe and the surround in the design of buried pipes, a factor that was not emphasised by the trapdoor experiments, due to their nature.
Marston's load theories form the basis of the UK specification for determining the external loads applied to rigid pipelines used for drainage applications (Young and O'Reilly, 1983). The need to determine the stress redistribution of the soil overlying a rigid pipe is because the worst load case will occur after redistribution. The shear plane method has also been applied in the field of tunnelling. The propagation of arching stresses by a yielding trapdoor is observed to be analogous to the volume loss around a bored tunnel. The results of the trapdoor experiments to determine the ultimate imposed stress on a trapdoor were shown to mirror the theoretical solutions proposed by Atkinson and Potts (1977a and b) and Wong and Kaiser (1991) to determine the minimum support pressure required to ensure that the soil does not collapse at the tunnel face. The applied overburden stress at the tunnel-soil interface is subsequently modified in an analogous manner to a buried pipe by the relative lining-soil stiffness.

For a flexible pipe installation, however, the maximum imposed stress occurs prior to pipe deflection as this causes the redistribution. The field trials described by Spannagel et al (1974) and Adams et al (1989) for flexible pipes buried beneath soil embankments showed that as the construction proceeded the resulting increase of overburden stress resulted in a change in the manner of pipe deformation. During the initial stages of embankment construction the application of vertical stress was resisted by the pipe deforming by distortional bending. The resulting redistribution of overburden stress was observed by an increase in the stiffness of the sidefill and a reduction below the level of the applied overburden stress of the imposed stress on the pipe. Continued embankment construction led to the pipe deforming by hoop compression, as the redistribution led to a state of uniform all-round earth pressure on the pipe wall. Similar alternations in the mode of pipe deformation had also been measured for flexible pipes placed in narrow trench conditions (Trott and Gaunt, 1972 and 1976). The mode of pipe deformation and therefore flexible pipe behaviour is determined by the relative pipe-soil stiffness. Importantly, the behaviour is governed by the initial stiffness of the sidefill, and additionally for trench installations the stiffness of the natural soil.
The most important performance limit for a flexible pipe is vertical pipe deflection. Due to the stress redistribution of the soil overlying a flexible pipe a non-linear increase of pipe deflection is observed for increasing magnitudes of applied overburden stress. Predictive methods that assume a linear increase of pipe deflection with applied stress and an elliptical pipe deformation will correspondingly over-predict the deflection of a pipe at high magnitudes of applied stress using a back-calculated value of soil modulus from an installation subjected to a lower magnitude of applied stress. This variation of soil modulus with increasing magnitudes of applied stress was highlighted by Howard (1977).

The review of the literature has shown that to predict the behaviour of a buried pipe it is necessary to understand the behaviour of the pipe, the soil and the interaction between them. It highlighted that there is lack of detailed information regarding the behaviour of both the buried pipe and the surrounding soil. The behaviour of a buried pipe and the surrounding soil can be measured by their displacements. The displacement measurements can then be used to determine shear and volumetric soil strain distributions around a buried pipe. This information indicates the magnitude and distribution of shear planes that form the basis of predicting the imposed stress on buried pipes. In addition, the review of literature highlights the importance of strain gauge readings on the pipe wall to determine the nature of the support provided to the pipe by the surround material and the degree of imposed stress on the buried pipe. Combining the measurements of imposed stress on a buried pipe with the shear soil strain distribution will provide a clearer understanding of the mechanism of buried pipe behaviour. These measurement techniques can also be used to provide a clearer understanding of the mechanisms of bored tunnel behaviour. A comparison can then be made between the behaviour of buried pipes and bored tunnels. A thorough investigation into the stress redistribution of the soil surrounding buried pipes and bored tunnels is thus proposed.
Figure 3.1 Determination of Narrow Trench Load Coefficient $C_d$
(after Young and Trott, 1984)

A - $C_d$ for $K_u$ and $K_u' = 0.19$, for granular materials without cohesion
B - $C_d$ for $K_u$ and $K_u' = 0.165$ max. for sand and gravel
C - $C_d$ for $K_u$ and $K_u' = 0.150$ max. for saturated top soil
D - $C_d$ for $K_u$ and $K_u' = 0.130$ ordinary max. for soil
E - $C_d$ for $K_u$ and $K_u' = 0.110$ max. for saturated clay
Figure 3.2 Complete Positive Projection Condition (after Young and Trott, 1984)

Figure 3.3 Notation for Settlement Ratio (\(r_{sd}\)) (after Bulson, 1985)
To extrapolate values of $C_e$ for higher values of $C/D$ use the following equations:

<table>
<thead>
<tr>
<th>Condition</th>
<th>$r_{mp}$</th>
<th>Equation of curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incomplete trench</td>
<td>-0.1</td>
<td>$C_e = 0.82(C/D) + 0.05$</td>
</tr>
<tr>
<td>$K\mu = 0.13$</td>
<td>-0.3</td>
<td>$C_e = 0.69(C/D) + 0.11$</td>
</tr>
<tr>
<td></td>
<td>-0.5</td>
<td>$C_e = 0.61(C/D) + 0.20$</td>
</tr>
<tr>
<td></td>
<td>-0.7</td>
<td>$C_e = 0.55(C/D) + 0.25$</td>
</tr>
<tr>
<td></td>
<td>-1.0</td>
<td>$C_e = 0.47(C/D) + 0.40$</td>
</tr>
<tr>
<td></td>
<td>-2.0</td>
<td>$C_e = 0.30(C/D) + 0.91$</td>
</tr>
<tr>
<td>Incomplete projection</td>
<td>0</td>
<td>$C_e = C/D$</td>
</tr>
<tr>
<td>$K\mu = 0.13$</td>
<td>+0.1</td>
<td>$C_e = 1.23(C/D) - 0.02$</td>
</tr>
<tr>
<td></td>
<td>+0.3</td>
<td>$C_e = 1.39(C/D) - 0.05$</td>
</tr>
<tr>
<td></td>
<td>+0.5</td>
<td>$C_e = 1.50(C/D) - 0.07$</td>
</tr>
<tr>
<td></td>
<td>+0.7</td>
<td>$C_e = 1.59(C/D) - 0.09$</td>
</tr>
<tr>
<td></td>
<td>+1.0</td>
<td>$C_e = 1.69(C/D) - 0.12$</td>
</tr>
<tr>
<td></td>
<td>+2.0</td>
<td>$C_e = 1.93(C/D) - 0.17$</td>
</tr>
<tr>
<td></td>
<td>+3.0</td>
<td>$C_e = 2.08(C/D) - 0.20$</td>
</tr>
<tr>
<td></td>
<td>+4.0</td>
<td>$C_e = 2.19(C/D) - 0.21$</td>
</tr>
<tr>
<td></td>
<td>+5.0</td>
<td>$C_e = 2.28(C/D) - 0.22$</td>
</tr>
<tr>
<td>Complete trench</td>
<td></td>
<td>Use the formula and $K\mu = 0.13$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$C_e = C_a = \frac{1 - \exp(-2K\mu C / D)}{2K\mu}$</td>
</tr>
<tr>
<td>Complete projection</td>
<td></td>
<td>Use the formula and $K\mu = 0.19$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$C_e = \frac{\exp(2K\mu C / D) - 1}{2K\mu}$</td>
</tr>
</tbody>
</table>

Figure 3.4 Determination of Positive Projection Fill Coefficient, $C_e$ (after Young and Trott, 1984)
Figure 3.5  Variation of Pipe Deflection with Cover Depth subjected to Surface Loading (after Gehrels, 1982)

A parabolic distribution of horizontal earth pressure is assumed

\[ e_s = \text{modulus of passive resistance of soil} \]

\[ \alpha = \text{bedding angle} \]

Figure 3.6  The Distribution of Pressures around a Flexible Pipe under Earth Loads (after Spangler, 1941)
Figure 3.7 Classification of System Behaviour for different Pipe Materials (after Young and Trott, 1984)
### Calculation of Loads on a Buried Flexible Pipe

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill load</td>
<td>$Y$ : mean unit weight of soil</td>
</tr>
<tr>
<td>For $h/D &gt; 2.5$</td>
<td>$P_{vd} = Y(h + D/2)$</td>
</tr>
<tr>
<td>$P_{hd} = K_d P_{vd}$</td>
<td></td>
</tr>
<tr>
<td>For $1 &lt; h/D \leq 2.5$</td>
<td>Allowance is made for the weight of soil displaced by the pipe:</td>
</tr>
<tr>
<td></td>
<td>$P_{vd} = Y(h + D/4)$</td>
</tr>
<tr>
<td></td>
<td>$P_{hd} = K_d Y(h + D/2)$</td>
</tr>
<tr>
<td>Concentrated surcharge</td>
<td>Use standard elastic stress distributions (Poulos and Davies 1974) to</td>
</tr>
<tr>
<td></td>
<td>determine vertical and horizontal pressures due to surcharge.</td>
</tr>
<tr>
<td>For $h/D &gt; 2.5$</td>
<td>$P_{vt} \cdot P_{ht}$ evaluated at pipe axis</td>
</tr>
<tr>
<td>For $1 &lt; h/D \leq 2.5$</td>
<td>$P_{vd}$ calculated as mean vertical pressure on plane of width $D$ at pipe</td>
</tr>
<tr>
<td></td>
<td>crown.</td>
</tr>
<tr>
<td></td>
<td>$P_{ht}$ evaluated at pipe axis</td>
</tr>
<tr>
<td>Uniformly distributed surcharge</td>
<td>$P_s$</td>
</tr>
<tr>
<td>For surcharge pressure $P_s$</td>
<td>$P_s = P_s$</td>
</tr>
<tr>
<td>$P_{hs} = K_d P_{vs}$</td>
<td>Traffic wheel loads</td>
</tr>
<tr>
<td></td>
<td>Calculated as combinations of concentrated surcharges.</td>
</tr>
<tr>
<td></td>
<td>Values of $P_{vt}$ due to standard UK traffic have been tabulated elsewhere.</td>
</tr>
<tr>
<td>Underwater installation</td>
<td>$Y_w$ : unit weight of water</td>
</tr>
<tr>
<td></td>
<td>$Y_s$ : submerged unit weight of soil ($Y_s = Y - Y_w$)</td>
</tr>
<tr>
<td></td>
<td>Soil effective stress components</td>
</tr>
<tr>
<td></td>
<td>$P_{vd}' = Y'(h + D/2)$</td>
</tr>
<tr>
<td></td>
<td>$P_{hd}' = K_d P_{vd}'$</td>
</tr>
<tr>
<td>Hydrostatic pressure</td>
<td>contributing directly to $P_z$:</td>
</tr>
<tr>
<td></td>
<td>$P_w = Y_w(h_r + h + D/2)$</td>
</tr>
<tr>
<td>Internal vacuum</td>
<td>Treat vacuum pressure $P_q$</td>
</tr>
<tr>
<td></td>
<td>(reduction in pressure below atmosphere) as external fluid pressure</td>
</tr>
<tr>
<td></td>
<td>contributing directly to $P_z$.</td>
</tr>
</tbody>
</table>

Figure 3.8  Calculation of Loads on a Buried Flexible Pipe (after Gumbel and Wilson, 1981)
External load on system

Pipe hoop thrust (compressive positive)

Pipe ring deflection (per unit radius)

**Figure 3.9** Uniform and Distortional Load Components and Related Effects (after Gumbel et al, 1982)
DEFINITIONS

- $E_s^*$ Elastic modulus of soil (plane strain)
- $S_{sp}$ Flexural stiffness of pipe ring ($= E_p^* l/D^3$)
- $P_y, P_z$ Distortional and uniform pressure components
- $\alpha$ Arching factor: for preliminary design assume $\alpha_f = 1$
- $\delta_y$ Increment of deflection due to external loading
- $\mu_l$ Reduction factor on buckling pressure due to initial out-of-roundness $\delta_{y0}$ and long-term deflection increment $\delta_{y3}$

Figure 3.10 Annotated Deflection - Buckling Chart (after Gumbel et al, 1982)
Figure 3.11  Radial Soil Pressure around a large diameter Steel Pipeline at Kirtling after Backfill and Surcharge Loading (after Trott and Gaunt, 1972)
1) Backfill to road base level  
2) 19 kN/m$^2$ surcharge  
3) 38 kN/m$^2$ surcharge

---

**Figure 3.12** Vertical Soil Pressure over the Pipe Crown of a large diameter Steel Pipeline at Kirtling after Backfill and Surcharge loading (after Trott and Gaunt, 1972)
Figure 3.13  Layout of Experimental Pipes beneath the Marlow-Bisham Bypass (after Trott and Gaunt, 1976)
**Figure 3.14** Deformation of the PVC-U Pipe laid beneath the Nearside Lane (after Trott and Gaunt, 1976)
Figure 3.15  Vertical and Horizontal Soil Pressures around the PVC-U pipe laid beneath the Nearside Lane (after Trott and Gaunt, 1976)
Figure 3.16  Permanent and Transient Deflections of the PVC-U Pipe caused by the passage of Scapers (after Trott and Gaunt, 1976)
Figure 3.17 Installation and Instrumentation details for a Steel Culvert buried beneath a 49m high Embankment (after Spannagel et al, 1974)
Figure 3.18  Diametral Strain changes of a Steel Culvert during and after construction of a 49m high Embankment (after Spannagel et al, 1974)

Figure 3.19  Measured and Predicted Crown Pressure on a Steel Culvert during and after construction of a 49m high Embankment (after Spannagel et al, 1974)
Figure 3.20  Variation of HDPE Pipe Diametral Strain and Vertical Compressive Strain of Trench and Free-Field Soil during construction of a 30.5m high Embankment (after Adams et al, 1989)

Figure 3.21  Variation of HDPE Pipe wall strain during construction of a 30.5m high Embankment (after Adams et al, 1989)
Figure 3.22  Variation of Earth Pressure on HDPE Pipe during construction of a 30.5m high Embankment (after Adams et al, 1989)

Figure 3.23  Loads on a Clay Pipeline over 10 year period (after Havell and Keeney, 1988)
Test Series No 1 - Rigid pipe
Cover/Diameter ratio = 1
Surface stress = 827 kPa

Test Series No 4 - Flexible pipe
Cover/Diameter ratio = 2
Surface stress = 1034 kPa

Figure 3.24 Distribution of Contact Stress around buried Model Rigid and Flexible Pipes (after Hoeg, 1968)

Figure 3.25 Relationship of Crown Stress for a Flexible Pipe (Test No 4) against Cover Depth under an Applied Uniform Surface Stress of 1034 kPa (after Hoeg, 1968)
Figure 3.26 Effect of Installation Method on PVC-U Pipe Deformation (after Gaunt et al, 1976)
Figure 3.27  Increase in Pipe Deformation with time under Static Loading (after Gaunt et al, 1976)
Figure 3.28  Geometry of Newtown Creek Culvert (after Selig et al., 1979)

Figure 3.29  Vertical Displacement at Culvert Crown during construction of a 4.6m high Sidefill and 7m high Embankment (after McVay and Papadopoulos, 1986)
Figure 3.30  Vertical Strains in Soil Embankment measured by Centrifuge Modelling
(after McVay and Papadopoulos, 1986)
Each complete cycle takes 2 minutes. Readings are not taken for each cycle.

Figure 3.31  Diametral Strain of HDPE Pipe due to application of 70kPa cyclic loading (after Rogers et al, 1995)
Figure 3.32  Vertical Ground Movements recorded along Tunnel Centreline constructed at New Cross, London (after Boden and McCaul, 1974)
Figure 3.33 Vertical Ground Movements recorded above a Tunnel constructed at New Cross, London (after Boden and McCaul, 1974)
Lateral Ground Movements recorded above a Tunnel constructed at New Cross, London (after Boden and McCaul, 1974)
Figure 3.35 Summary of Ground Behaviour around the First Tunnel constructed at Lafayette Park, Washington D.C. (after Hansmire, 1975)
Figure 3.36  Ground Reaction curve for both Tunnels constructed at Lafayette Park, Washington D.C.  
(after Hansmire and Cording, 1985)
7 Days
Air Off
Final

Array 'A'

Distance to \( C \) in metres

20
18
16
14
12
10
8
6
4
2
0

Array 'B'

Distance to \( C \) in metres

10
8
6
4
2
0
2
4
6
8
10
12

Array 'C'

Figure 3.37 Development of Long-Term Surface Settlement, in the Transverse Plane, above a tunnel constructed at Grimsby (after O'Reilly et al, 1991)

(b) Transverse settlements
Figure 3.38 Development of Load Imposed on Tunnels constructed at
(a) Regents Park, London and (b) Oxford Sewer
(after Barratt et al, 1994)
Figure 3.39  Error Function curve
Figure 3.40  Subsurface Total Displacement Vector plots for Model Tests in Sand (after Cording et al, 1976)
Figure 3.41  Contours of Volume and Shear Strains around Model Tunnels near collapse (a) Dense sand, $\sigma_4=0$, (b) Dense sand $\sigma_4=0$, centrifuged at 75g and (c) Overconsolidated Kaolin, $\sigma_4=140\text{kN/m}$ (after Atkinson and Potts, 1977a)

Figure 3.42 Upper Bound Collapse Mechanism (Atkinson and Potts, 1977b)
Figure 3.43 Laboratory Arrangement of Centrifuge Shield Model (after Nomoto et al, 1996)

Figure 3.44 Earth Pressure on top side of Buried Pipe, Tail Void and Shield Model Tests (after Nomoto et al, 1996)
Figure 3.45  Ground Movements associated with Open Pipejacking Model Tests (after Chapman, 1992)
Figure 3.46 Modes of Tunnel Behaviour for various $P/P_o$ ratios against Stress Ratio $K_o$ (after Wong and Kaiser, 1987)

Figure 3.47 Idealised Ground Reaction curve for a Point at Tunnel (after Wong and Kaiser, 1987)
### Table 3.1 Installation Conditions described by Marston's Load Theory
*(after Marston, 1930)*

<table>
<thead>
<tr>
<th>Installation Condition</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipes in Narrow Trenches</td>
<td>Pipes installed in narrow trenches excavated into the undisturbed soil</td>
</tr>
<tr>
<td>Positive Projecting Pipes</td>
<td>Pipes installed in a shallow bedding with the top of the pipe projecting above the natural ground surface, then covered with an embankment</td>
</tr>
<tr>
<td>Negative Projecting Pipes</td>
<td>Pipes installed in shallow trenches with the top of the pipe below the natural ground surface, then covered with an embankment</td>
</tr>
</tbody>
</table>

### Table 3.2 Recommended values of settlement ratio ($r_{sd}$) for the design of rigid pipes placed within a positive projection condition
*(after Young and Trott, 1984)*

<table>
<thead>
<tr>
<th>Situation</th>
<th>Foundation Soil Condition ($d_c = 0$)</th>
<th>Value of $r_{sd}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wide Trench</td>
<td>Any</td>
<td>+0.7 to +1.0</td>
</tr>
<tr>
<td>Embankment</td>
<td>Rock or unyielding soil</td>
<td>+1.0</td>
</tr>
<tr>
<td></td>
<td>Ordinary soils</td>
<td>+0.5 to +0.8</td>
</tr>
<tr>
<td></td>
<td>Yielding soils</td>
<td>0 to +0.5</td>
</tr>
<tr>
<td>Pipeline in very soft ground</td>
<td>Pipeline supported by piles</td>
<td>Use of complete projection</td>
</tr>
<tr>
<td>Material Description</td>
<td>British Soil classification group symbols</td>
<td>Modulus of Soil Reaction (E') (MN/m²)</td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>--------------------------------------------</td>
<td>----------------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Un-compacted</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a)</td>
</tr>
<tr>
<td>Aggregates to BS 882 10mm, 14mm, 20mm sizes 14 - 5mm graded 20 - 5mm graded</td>
<td>-</td>
<td>7.0</td>
</tr>
<tr>
<td>Aggregates to BS 1047 Lightweight aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregates to BS 3797 Sintered pulverised-fuel ash See Note (c)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel - single sized</td>
<td>GPu (d)</td>
<td></td>
</tr>
<tr>
<td>Gravel - graded</td>
<td>GW (e)</td>
<td></td>
</tr>
<tr>
<td>Sand and coarse grained soil with less than 12% fines</td>
<td>GP (f)</td>
<td>1.0</td>
</tr>
<tr>
<td>Coarse grained soil with more than 12% fines</td>
<td>GM (i)</td>
<td>-</td>
</tr>
<tr>
<td>Fine grained soil with medium to no plasticity and containing more than 25% coarse grained particles (LL &lt;50% (r))</td>
<td>CL (l)</td>
<td>-</td>
</tr>
<tr>
<td>Fine grained soil with medium to no plasticity and containing less than 25% coarse grained particles (LL &gt;50% (s))</td>
<td>CL (l)</td>
<td>-</td>
</tr>
</tbody>
</table>

Shaded area represents recommended values to use.

Notes
(a) For any situation where bedding and sidefill trench material must be placed and compacted within temporary trench supports, the value chosen for \(E'\) should be that associated with the uncompacted material.
(b) BS 1377 Test 12, "Determination of the Dry Density/Moisture Content Relationship (2.5Kg rammer method)", is used to determine the Standard Proctor Density.
(c) Imported bedding and sidefill material, conforming to BS 882, BS 1047 or BS 3797 will compact to a density close to maximum when just dumped in a trench and subjected to the normal workmanship practice generally associated with pipe laying.
(d) Poorly graded uniform gravel.
(e) Well graded gravel.
(f) Poorly graded gravel.
(g) Well graded sand.
(h) Poorly graded sand.
(i) Very silty gravel.
(j) Very clayey gravel.
(k) Very silty sand.
(l) Clays with low plasticity.
(m) Silts with low plasticity.
(n) Clays with intermediate plasticity.
(o) Silts with intermediate plasticity.
(p) Mixtures of ML and CL.
(q) Mixtures of MI and CI.
(r) Clays and silts, with a liquid limit less than 50% and an appreciable fraction passing the 75mm BS test sieve, with more than 25% coarse grained particles.
(s) As note (r) with less than 25% coarse grained particles.
(t) <40% relative density.
(u) 40 - 70% relative density.
(v) >70% relative density.

Table 3.3 Modulus of Soil Reaction \(E'\) for use with Iowa formula (after Howard, 1977)
4. RESEARCH PHILOSOPHY

4.1 INTRODUCTION

Chapters 2 and 3 described previous research studies that have been conducted to investigate the main aspects of the behaviour of buried pipes and bored tunnels. This review of the literature highlighted in particular, that few studies have been undertaken to record information regarding both the behaviour a buried pipe and the surrounding soil. Basic measurements from field and laboratory studies involved recording the behaviour of the pipe itself by measuring the changes of the pipe dimensions and attaching strain gauges to the pipe wall. Internal circumferential pipe wall strains provide a quantitative description of the effects of the installation procedure upon the behaviour of a buried pipe, whilst the measurement of both external and internal circumferential pipe wall strains yield the data required for calculations of pipe wall thrusts and bending moments. Measurement of the behaviour of the surrounding soil involved the use of earth pressure cells and fluid level settlement platforms, which provided a more comprehensive insight (albeit at discrete points) into buried pipe-soil interaction. It was concluded, however, that in general there was a lack of information regarding the measurement of both the behaviour of buried pipes and the surrounding soil, particularly from laboratory-based experiments.

By contrast, studies describing the complete behaviour of bored tunnels and the surrounding soil appeared to be more numerous. Sections 3.7 and 3.9 described a number of field and laboratory studies that were conducted primarily to investigate the ground movements around bored tunnels, but also investigated the areas of tunnel face stability and tunnel lining behaviour. Nevertheless there does still appear to be generally a lack of detailed information regarding the area of tunnel-soil interaction, particularly for the behaviour of tunnels in granular materials.

The literature review showed that, in addition to the traditional methods used to investigate the behaviour of buried pipes, photographic techniques (already used for tunnels) could be employed to investigate the additional behaviour of the soil surrounding buried pipes. The combined use of the photographic technique and pipe
wall strain measurement could also be used to determine tunnel lining stresses and ground movements around tunnels constructed through granular soils.

This chapter initially outlines in Section 4.2 the different research methods available and explains why a laboratory physical modelling technique was adopted for this study. Section 4.3 describes the basic factors which influence the behaviour of buried pipes and tunnels, in the context of which the effects of the boundary conditions (described in Section 4.4) can be judged. Experimental considerations are outlined in Section 4.5, and Section 4.6 concludes this Chapter by discussing the philosophy behind the test programme.

4.2 METHODS OF INVESTIGATION
The approach adopted by a research investigation can be classified into broadly three groups; field monitoring, laboratory modelling and theoretical modelling. This section describes the advantages and disadvantages of each of these three approaches, from which it was decided that laboratory modelling would be adopted.

The field trials described in Sections 3.3 and 3.7, for buried pipe installations and bored tunnels respectively, showed that measurements could be obtained to describe the behaviour of a buried structure and the surrounding soil. Measurements from these type of installations are potentially free from any significant approximations and assumptions that may be incorporated into a laboratory or theoretical model. To obtain a comprehensive description of the behaviour of a buried structure it is necessary to record the internal dimensions of the structure, attach strain gauges onto the wall of the structure, and also embed pressure cells into the walls of the structure.

Instrumentation of the surrounding soil would typically require, for a buried pipe, pressure cells to be installed during the backfill operation, piezometers to record the changes in the pore water pressure (in the case of cohesive materials) and subsurface settlement monitoring to record the complete ground movement pattern. The degree and detail of the information gained from a field trial would depend upon the number of instrumented sections used and, for each of these sections, the number and variety of the instruments installed. Irrespective of the chosen method of investigation, all
measuring techniques must be as precise and reliable as possible to ensure that the results are meaningful. Satisfying these criteria is more demanding for field, as opposed to laboratory, conditions because the control over soil conditions and environmental factors is reduced. For a field trial therefore the instrumentation has to be insulated from the effects of water and temperature, and the absence of soil rotation between experiments that occurs during laboratory testing restricts access to the instrumentation to enable them to be serviced. It is apparent that a detailed field trial is expensive, requires considerable resources and is, most importantly, unique.

An intermediate approach between a field trial and small scale laboratory testing is to reproduce the field conditions in a laboratory thus enabling a higher degree of control over the factors which influence the behaviour of the instrumented subjects. This approach was adopted by the researchers at the Iowa State University, for example Spangler (1948), and at Ohio State University, for example Sargrand et al (1995), who investigated the behaviour of buried pipes by using test rigs subjected to field conditions. These studies highlighted the problems associated with this method, including the need for a very large test rig. The scale of each experiment increased the time required to conduct each experiment, thereby limiting the number of experiments that were performed and ultimately, the scope of the research project.

Laboratory modelling techniques can be classified according to the degree with which they simulate the behaviour of the real structure. The following description of previous bored tunnel studies highlights the variety and complexity of potential modelling techniques. The trapdoor experiments conducted by Terzaghi (1936) represent the most simplistic representation of the ground behaviour around tunnels in granular soils. A more complex two-dimensional representation of tunnel construction was studied in the 1970s by researchers at Cambridge University, who used a deflating air bag to initiate tunnel collapse in static and centrifuge models. At the same time, Cording et al (1976) proposed an alternative analysis by using concentric tubes to create a tail void loss, a procedure that was subsequently repeated in the centrifuge by Nomoto et al (1996). Nomoto and his colleagues demonstrated that the complete process of tunnel construction could be simulated by using a miniaturised tunnelling machine, a practice
also adopted by Kim et al (1996). However Bolton et al (1996) stated that, whilst the simulation of a travelling tunnel head achieved the correct soil strain and stress behaviour, it was a difficult and expensive modelling technique that would not further existing knowledge unless the equipment was highly reliable and great control was exercised.

These contrasting approaches highlight the advantages and disadvantages of different modelling techniques. Simulating the complete behaviour of a real structure is expensive and complicated, whereas simplifying the behaviour of a real structure is cheaper and easier to develop whilst still yielding extremely valuable information. With all modelling techniques scaling problems need to be carefully considered before the results from the experiments can be used to predict the behaviour of a real structure.

The third possible approach is to develop a theoretical model to predict the redistribution of soil stresses around buried pipes and tunnels. The success of a theoretical model is dependent upon how it simulates the behaviour of the buried structure and the soil, the construction process and its comparison to experimental data. Due to the complicated nature of the behaviour of buried pipes and, in particular the excavation process involved with bored tunnels, a three-dimensional finite element analysis would represent the best approach (Sakajo et al, 1996). Results from such an analysis would need to be validated by, initially, laboratory model data, and ultimately field trial results. Development of a theoretical modelling technique would therefore be slow, but once validated for a certain set of conditions, could be used to provide an infinite number of permutations of varying site conditions.

Following from the review of literature it was decided that the investigation would be based upon laboratory modelling using a 1.5 x 1.0 x 1.5m deep test rig, (see Figure 4.1) developed previously by Chapman (1992). The dimensions of this test rig were based upon the requirement to develop a laboratory modelling technique which could simulate trenchless pipelaying construction processes at an intermediate scale between a field trial and small-scale testing. This approach was also adopted during this research project for the tunnel tests as the diameters of tunnel tubes corresponded
closely to those used by Chapman and were between 194 and 245mm. For the buried pipe tests however, the outside diameter of the pipes were between 120 and 250mm and were typical of sections of pipe used in practice. In addition, the behaviour of the selected pipe and tubes, and the surrounding soil could be viewed between the maximum spacing of 400mm between the steel stanchions of the test rig.

For the experiments reported herein, the simplistic nature of the tail void approach was adopted to represent the behaviour of a bored tunnel. This approach permitted the controlled promotion of a uniformly distributed volume loss at the tunnel lining during which the ground movements and lining response could be recorded. The behaviour of the tail void tunnel tests and the buried pipe tests were recorded at the interface between the structure or surrounding soil and the glass wall of the laboratory test rig. To allow a comparison of the behaviour of these two types of buried structures, the structures were split along their cross-section, i.e. perpendicular to the longitudinal axis of the structures. By adopting a similar scale of outer (shield) and inner (lining) tubes to those used previously by Chapman, the ground movements recorded in the perpendicular plane for the tunnelling situation recorded herein could be compared with those from the pipejacking tests after a withdrawal of the outer tube by 10mm, thereby allowing an assessment of the influence of the face loss to be made.

4.3 FACTORS INFLUENCING THE BEHAVIOUR OF BURIED PIPES AND BORED TUNNELS

The factors influencing the behaviour of buried pipes and bored tunnels have been isolated and are discussed in Sections 4.3.1 and 4.3.2 respectively.

4.3.1 Buried Pipes

Figure 4.2 summarises the main factors that influence the behaviour of a buried pipe. To ensure the satisfactory performance of highway drainage pipelines in the UK, the Specification for Highway Works, Highway Construction Details (DoT, 1991a and b) and the advice document HA40/89 (DoT, 1990) provide recommendations concerning a range of acceptable surround materials, fill materials, trench widths, bedding thickness and cover depths.
4.3.1.1 Pipe Type - Geometry and Nature

The influence of the stiffness of the pipe is shown by the relative critical performance parameters for rigid and flexible pipes which are excessive loading and excessive deformation respectively. The structural behaviour of a rigid or flexible pipe must be considered in relation with the stiffness of the soil surround. The stiffness of a pipe is governed by the mechanical and thermal properties of the material from which it is manufactured and its geometry. These factors combined influence the behaviour of the pipe during the installation procedure, the ease with which soil can be placed beneath the pipe haunches and its structural behaviour whilst being subjected to externally applied stresses.

No consideration will be given to the behaviour of the joints between the sections of buried pipe as the performance of these structural elements will only be critical if the existing controls on the performance of buried pipes are relaxed. In addition, there will be no consideration to the behaviour of structural wall pipes, the simpler situation of single-walled pipes being most important to determine flexible pipe behaviour.

4.3.1.2 Surface Loading

Trott and Gaunt (1976) showed that construction traffic loading was the worst potential loading case for road drainage pipes. The reasons for this are the reduced cover depth (compared with that of the finished road) and the potential application of vehicle loading which approaches or exceeds the legal axle load limit. The design charts stated in the advice note HA40/89 (DoT, 1990) do not consider the influence of the higher loads that may result from construction traffic loading and recommends that the influence of this loading condition can be diminished by temporary bridging, increased cover depth or the adoption of a stronger pipe and bedding combination design. Due to the limited data on the influence of traffic loading on flexible pipes, it is assumed that the influence of traffic loading is calculated for flexible pipes in the same manner as that for rigid pipes using the classifications of ‘main’, ‘light’ and ‘field’ road loading. Continued application of a load of the same magnitude is seen for a flexible pipe to result in an increase in vertical pipe deflection according to an asymptotic
relationship with the number of applications. The results from Trott and Gaunt (1976) showed that the permanent deflection of the two types of flexible pipes tested had reached an ultimate value after 10 passes of the scapers.

The influence of surface loads of limited extent diminishes with depth, whereas surface loads of large extent are assumed to be transmitted to the pipe unaltered. For a buried pipe constructed beneath a high embankment, the worst loading condition will arise from the self-weight of the embankment material. Deformation of a flexible pipe surrounded by a granular material under a constantly applied static loading will tend to a certain value, the curve of deformation against time being asymptotic.

4.3.1.3 Cover Depth to Pipe Diameter Ratio (C/D)
From the discussion regarding the influence of surface loading it is apparent that the cover depth (C) to pipe diameter (D) ratio influences the magnitude of the surface loading on a buried pipe. For a very flexible buried pipe constructed beneath a positive projection embankment, Figure 3.4 illustrates that the load on the pipe reaches a limiting value, whereas for an extremely rigid pipe the load on the pipe increases rapidly with cover depth. For a narrow trench installation condition, due to influence of trench wall friction, the load on a pipe reaches a limiting condition (see Figure 3.1), which is similar to the relationship for a very flexible pipe installation beneath a soil embankment. The relationship between soil loading and traffic loading was summarised in Figure 3.5 for a flexible pipe in terms of the measured VDS. At shallow depths of cover, traffic loading is shown to be the dominant factor, with soil load becoming the dominant factor at higher depths of cover.

4.3.1.4 Bedding
The bedding beneath a buried pipe is used to provide uniform support to the pipe and is particularly important for weak in-situ soils. Uniform support to the pipe is required to ensure that there is limited longitudinal settlement, the line and level of the pipeline is maintained and that no point loads are formed. Increasing the bedding angle beneath a buried pipe increases the degree of support, which for a flexible pipe decreases the
resulting deflection for a given load and for a rigid pipe increases the load carrying capacity.

For a rigid pipe, Young and Trott (1984) state that a concrete bedding may be adopted when the highest possible support is required, for example for a shallow-buried pipe installed under a heavily trafficked roadway. Due to its ease and speed of construction a granular bedding is the most widely used form of bedding for rigid pipes in the UK. In addition, during construction it would be expected that the self-weight of the rigid pipe would cause it to settle into the bedding, reducing the imposed stress on the pipe. For flexible pipes, a bedding of granular material of at least 100mm thick is specified for road drainage applications, irrespective of the pipe diameter (DoT, 1991b). Use of cohesive material for the bedding and sidefill material for PVC-U pipe installations was shown by Gaunt et al (1976) to result in a relatively large long-term increase of pipe deflection as the pore water pressure generated during construction dissipated, resulting in consolidation settlements.

4.3.1.5 Installation Condition - Geometry and Mechanical Properties

To permit access adjacent to the pipe, the minimum specified trench width for a flexible drainage pipe is the pipe diameter plus 300mm (DoT, 1991b). The effect of increasing the width of the trench is to reduce the beneficial actions of trench wall friction and the passive resistance of the in-situ soil (which is usually greater than that of the compacted sidefill). The transition trench width, between a narrow trench condition and an embankment installation condition, at which the in-situ soil has no influence on the behaviour of the pipe is usually quoted as being five times the diameter of the pipe (e.g. Schlick, 1932). The influence of sloping, rather than vertical, trench walls is to increase the load on the pipe as the effect of trench wall friction is reduced.

The stiffness of the sidefill material is the most important factor in determining the short and long-term behaviour of a buried flexible pipe. The relative pipe-soil stiffness governs the degree of imposed stresses on the buried pipe from the external loading conditions, through the actions of arching and degree of passive lateral soil resistance generated. To develop the beneficial actions of positive arching and passive lateral soil
resistance to a flexible pipe, and for a rigid pipe to reduce the potential degree of
negative arching, both the vertical and horizontal soil stiffness are important. In
practice, the specifications for UK highway drainage pipelines state that the surround
should extend to 300mm above the pipe crown and that the bedding, haunch and
surround shall be brought up equally on both sides of the pipe and carefully compacted
in layers not exceeding 150mm thickness (DoT, 1991a). The installation of the sidefill
material can also largely determine the long-term behaviour of a flexible pipe as the
compaction of the sidefill material may result in a negative VDS, a condition which is
only reversed upon application of sufficient overburden and/or surface stresses.

The role of the backfill material is primarily to protect the installation. Structurally, the
horizontal and vertical stiffness of the backfill influences the lateral and vertical
distribution (hence magnitude) of the induced positive or negative arching stresses.
This is highlighted by the 'induced trench method' in which compressible material is
placed directly above the crown of the buried pipe to induce positive arching as the
compressible material deforms relative to the adjacent, external soil prisms (Valestad,

4.3.1.6 Water Table
During the lifetime of a buried pipeline it will be subjected to external water loading.
This will normally take the form of groundwater, and will be controlled by water table
fluctuations. The effect of a water table above the pipe crown on the pipe stresses will
be two-fold. The vertical load (hence stress) applied to the pipe will be reduced as the
buoyant unit weight is applicable for the distance between the elevation of the pipe
crown and the water table. For a flexible pipe this will reduce the tendency for
distortional bending. The second effect is that the a hydrostatic water pressure will act
on the pipe which will cause it to undergo a uniform hoop compression. For a flexible
pipe this will cause a reduction in the diameter of the pipe around the complete
circumference. In situations where the hydrostatic pressure is high and the pipe is
relatively flexible, buckling may result.
The effect of pore water pressure changes on buried pipe behaviour was highlighted by the laboratory study reported by McVay and Papadopoulos (1986) during the construction of a clay embankment above and beside a flexible pipe in a granular surround. Construction of the embankment led to the generation of positive pore water pressures, which would have increased with depth as the total stresses on a particular soil layer. After completion of the embankment, the dissipation of this excess pore water pressure was aided by the drainage path provided by the granular surround, and resulted in consolidation settlement. The induced negative arching between the embankment and the granular surround resulted in an increase of load on the buried pipe, as evidenced by the recorded thrust stresses at the crown and the springline of the culvert increasing by 10 and 20% respectively. Consolidation of the in-situ soil adjacent to trench installations may also occur for trenches constructed in very soft clays as a result of changes in pore water. The influence of consolidation settlement upon buried pipe installations must be considered with respect to the long-term design criteria.

4.3.2 Bored Tunnels

4.3.2.1 Soil Type and Density
The type of soil through which a tunnel is bored influences the magnitude and extent of the induced soil displacements, the imposed stresses on the tunnel lining and the stability of the tunnel face. The experiments conducted by Chapman (1992) showed that in areas where the soil was compressed, the extent and magnitudes of displacements in a dense sand were greater than those in a loose sand due to their relative compressibilities. In areas where the soil experienced stress relief however, the reverse was observed as the increased dilation rate of dense sand limited the extent of the observed displacements. It is generally assumed that tunnels constructed in non-cohesive materials typically behave in a drained manner resulting in only short-term displacements, due to the relatively rapid rate at which modern tunnels are constructed.

4.3.2.2 Cover Depth to Tunnel Diameter Ratio (C/D)
It was stated in Section 2.2 that the cover depth to tunnel diameter (C/D) ratio greatly influenced the degree of arching induced in the overlying soil and would influence the
behaviour of tunnels constructed particularly through granular soils. This mechanism will influence both the nature of the observed ground movements and the stresses imposed on the tunnel lining. Increasing the cover depth increases the confining stress, which as Atkinson and Potts (1977a) showed, magnified the effect of the dilation of the soil.

4.3.2.3 Overcut Ratio (o/r)

The distance between the outside diameters of the shield and lining is commonly referred to as the overcut (o). The ratio between the overcut dimension (o) and the lining radius (r) around the circumference of the lining influences the volume and distribution of ground loss at the lining. This consequently influences the magnitude and distribution of the ground movements induced during the construction of the tunnel. Increasing the overcut ratio increases the magnitude and extent of the induced soil movements, propagating greater arching stresses.

4.3.2.4 Pipe Alignment

The induced volume loss can also be influenced by the alignment of the excavation. Deviation of the alignment of the tunnel machine increases the excavated area and, because the tunnel lining has a fixed area, results in greater volume loss and ultimately increased magnitudes of induced ground movements.

4.3.2.5 Water Table

The conclusions stated regarding the influence of water above a buried pipe are equally applicable to the case of a bored tunnel, with the important additional consideration that the presence of water greatly influences the stability of the tunnel face.

4.4 BOUNDARY CONDITIONS

Reproducing a laboratory arrangement which would be able to investigate each of the factors described in Sections 4.3.1 and 4.3.2 would be complicated, time-consuming and potentially unnecessary. The approach adopted for these experiments was to concentrate only on those factors which were perceived to be important. The following section describes boundary conditions that were applicable to both the buried pipe and
bored tunnel experiments and then describes specific considerations for the buried pipe and bored tunnel experiments respectively.

The aim of the laboratory work was to conduct controlled experiments which necessitated some degree of simplification. The soil type chosen was a uniform Leighton Buzzard sand, which has been used by numerous researchers including Chapman (1992). The properties of this sand are well defined, and because of the size and uniformity of the sand particles (between 1-2mm, see Figure 5.20) they are visible on photographs, thereby allowing the developed digitised method to be used without the need for markers to be placed in the sand. The uniform nature of the sand results in a small density range between dense and loosely compacted states, thus reducing the variability of the resulting displacements and increasing the consistency of the soil density between each experiment. Use of Leighton Buzzard sand during the tunnel tests also facilitated comparison with the results of Chapman (1992).

A dry sand was used throughout the experimental programme to eliminate the need to measure the pore water pressure around the buried structure. For reasons of practicality, during each buried pipe experiment the bedding, sidefill, backfill and in-situ soil materials were all represented by Leighton Buzzard sand. The resulting buried pipe installation simulated a positive projection embankment condition.

The buried pipe experiments were conducted using PVC-U pipes having external diameters of 250 and 168mm and wall thicknesses of 2.7 and 3.5mm respectively, and a 120mm diameter vitrified clay pipe with a wall thickness of 10mm. The use of PVC-U and vitrified clay pipes allowed the contrasting behaviour of flexible and rigid buried pipe installations to be observed. The very flexible PVC-U pipe with a diameter of 250mm was selected to maximise pipe deflection with respect to the displacement of the soil surround thereby inducing significant positive arching stresses within the overlying soil. A vitrified clay pipe was used to simulate rigid pipe behaviour as the mechanical and thermal properties were known and reliable. Selecting a pipe of 120mm diameter allowed the smooth barrel end required to create a seal between the pipe and glass wall of the test tank to be formed using a rotating diamond cutter. In addition,
McNulty (1965) showed from his trapdoor experiments that greater relative displacements between the trapdoor and the surrounding soil were required to induce the same degree of negative compared to positive arching. The use of a smaller diameter clay pipe compared to the 250mm diameter PVC-U pipe aided the promotion of negative arching as it allowed a larger ratio of height of fill to pipe diameter to be subjected by use of the same magnitude of surface surcharge. The 168mm diameter PVC-U was selected to provide an intermediate pipe stiffness between the 250mm diameter PVC-U and vitrified clay pipes.

Use of rigid and flexible pipes additionally allowed observations to be made regarding the contrasting frictional behaviour at the pipe-soil interface. The use of pipes with varying external diameters, whilst using the same sand surround throughout the experimental programme, allowed variation of the aspect ratio between the pipe diameter and the mean size of the sand particles to be studied. Adopting a mean diameter of 1.5mm for a Leighton Buzzard sand particle, the resulting aspect ratios for the pipe-sand combinations used in this experimental programme were 80, 112 and 167. An aspect ratio of between 15 and 90 is typical for UK drainage applications.

For comparison with Chapman (1992), tunnel tubes with outside diameters of 194, 219 and 245mm were used. Use of the 194mm diameter tube as the inner (lining) tube resulted in overcut ratios of 0.13 and 0.26. These dimensions compare favourably with those of Chapman who used a 200mm diameter lining tube, resulting in overcut ratios of 0.1 and 0.2. Whilst these overcut ratios are considerably greater than those used in actual tunnel projects, the principle behind the simulation of tail void loss is nevertheless applicable to simulating bored tunnel behaviour.

From the literature review it was concluded that the behaviour of buried structures was influenced by the soil cover depth to structure diameter ratio (C/D). The influence of this factor was therefore assessed rather than during the buried pipe tests adopting the minimum burial depth of between 1.2 and 0.6m for rigid and flexible pipe highway drainage under road and field loadings (DoT, 1990 and 1991b). To simulate increasing depths of burial, the water bag arrangement developed by Chapman (1993) was used.
to apply minimum and maximum static uniform surface surcharges of 50 and 150kPa respectively. This arrangement was also used to simulate traffic loading by repeat application of a peak cyclic uniform surface surcharge of 70kPa.

4.5 EXPERIMENTAL CONSIDERATIONS

The main experimental consideration was to assess the influence of the sides of the test rig on the behaviour of the buried structures. Atkinson and Potts (1977a) illustrated that it is possible to measure the plane strain behaviour of an unlined tunnel by recording the displacements of lead shots placed in the centre of the test rig, thereby reducing the boundary conditions of the test rig. Upon removal of the glass face of the test rig, the unlined tunnel was placed vertically within the test rig to allow the surround material and the lead shots to be placed in vertical layers. For these tests it was not possible to use the X-ray technique adopted to record the position of the lead shots for safety reasons and the use of markers was avoided as they locally stiffen the soil medium.

An alternative approach is to record the behaviour of the buried structure and the surrounding soil at the glass wall of a test rig. This approach allows the location of the cross-section of the buried structure and the surrounding sand particles to be recorded photographically, and hence far more comprehensively, and displacements to be determined by the developed digitising method. Measurement of the position of sand particles, rather than markers, also allows the controlled placement of sand in horizontal layers, thereby simulating construction activities. The only disadvantage with this approach is that the friction between the buried structure, or the soil, and the glass may influence the observed behaviour. This problem is partially negated by coating the ends of the buried structures with polytetrafluoroethylene (PTFE), which has a very low coefficient of friction, thereby allowing the structure to deform freely against the glass face. As Cording et al (1976) stated, glass-sand friction will only influence the observed behaviour at very high stress levels, a condition that was not applicable during the tunnel experiments. Due to high static surface stresses applied during the buried pipe experiments it was deemed necessary to investigate this potential problem. This was achieved by conducting an experiment throughout which
the strain response of the buried pipe was recorded at the glass-sand interface and centrally within the test rig and by a series of shear box experiments to calculate the angle of friction between glass and sand. The aim of these additional studies was to deduce the influence of the boundary conditions of the test rig so that the behaviour of the true structure may be determined. Whilst this consideration is important, the influence of boundary conditions is reduced when the results of tests subjected to the same boundary conditions are compared on a relative, rather than an absolute basis.

4.6 PHILOSOPHY BEHIND PROGRAMME

The primary aim of the experimental programme was to investigate the behaviour of buried pipes. Investigating the behaviour of the 'tail void' simulated tunnel tests was a secondary aim. To develop laboratory experimental techniques to achieve these aims it was necessary to simplify the behaviour of the real structures and the factors which influence them. After consideration was given to the limitations of the developed experimental techniques, it was intended that the experiments were conducted in a manner as realistic as possible to enable the results to be meaningful and allow them to be related to theoretical, experimental and field studies. To achieve the aims of the project it was necessary to develop the following: a new measurement technique to record displacements from photographs; the apparatus for the tunnel experiments; the measurement device for recording strain gauges; and the installation procedure for the buried pipes. This development work was a major part of the project. Prior to commencing with an experimental programme using the buried pipe equipment (see Section 5.8), numerous preliminary tests were undertaken to refine and develop the adopted equipment and procedure. The results from the two tunnel tests presented herein are deemed to be pilot studies and consequently further development is required to utilise fully the equipment.
Figure 4.1  Construction details of the Test Tank (after Chapman, 1992)
1. Surface load
2. Ratio of cover depth to pipe diameter
3. Ratio of bedding thickness to pipe diameter
4. Ratio of bench width to pipe diameter
5. Shape of trench
6. Type of bedding and sidefill
7. Method of laying bedding and sidefill
8. Type and method of laying backfill
9. Type and state of in situ soil
10. Groundwater level
11. Quality of pipe material
12. Pipe class (or S.D.R.)
13. Pipe abnormalities
14. Longitudinal bending effects
15. Temperature
16. Time
17. Settlement of pipe and fill

Figure 4.2 Factors influencing Buried Pipe Behaviour (after Rogers, 1985)
5. EXPERIMENTAL METHOD

5.1 INTRODUCTION
Chapter Four outlined the factors which influenced the philosophy and design of the experimental equipment used for the investigation of the behaviour of buried pipes and bored tunnels. The aim of this section is to describe the methods used to perform these tests. Sections 5.2 and 5.3 respectively outline the equipment and the data acquisition systems that were developed to provide controlled conditions for these tests. The decisions that were taken during the development of these systems is discussed.

Throughout each experiment, a specifically developed digitising method (see Section 5.3.1) was used to record the total displacement fields of both the buried structures and the surrounding soil. In addition, a strain gauge data acquisition system was also specifically developed to monitor the structure wall strains. A Leighton Buzzard sand was used throughout the experimental programme and is classified in Section 5.4. Section 5.5 describes the method used to place the sand into the test tank at uniform densities. The test procedure for the buried pipe experiments was developed as a result of numerous preliminary experiments and is described in Section 5.6. The test programme for these experiments is presented in Section 5.8. Sections 5.7 and 5.9 describe the test procedure and programme adopted for the two full tunnel tests respectively.

5.2 LABORATORY TEST EQUIPMENT
5.2.1 Laboratory Test Tank
Both the buried pipe and tunnel tests were conducted using a laboratory test tank originally designed by Chapman (1992). The general arrangement of the test tank and its construction are shown in Figure 5.1 and Figure 4.1 respectively. Only a brief description of the construction of the test tank is presented herein (for greater detail refer to Chapman, 1992). The internal dimensions of the tank are 1.5m long, 1.5m high and 1.0m wide. The tank was designed to apply a maximum vertical stress of 200kPa to the soil contained within it, which approximates to 12.5m of overburden, with a $K_o$ value of unity. The structural frame of the tank was designed to give a maximum
viewing panel of 400 by 500mm, using 30mm thick glass and a factor of safety of 2 on the assumed applied stress. The steelwork used for the supporting frame was designed on the basis of deflection rather than load capacity.

To reduce the effects of tank wall friction on the behaviour of the buried structures, the buried structures were viewed through the 1.5m (rather than the 1.0m) wide glass side. Except for a single buried pipe test, the buried structures were placed in the centre of the 400mm wide, central viewing panel. The height of the viewing panel was determined by the position of adjustable horizontal beams, which provided hoop strength to the tank and supported the glass.

5.2.2 Tank Lid and Water Bag Loading Arrangement
To simulate burial depths in excess of that permissible into the tank, a water bag arrangement was developed (Chapman, 1992). Once the tank was full and the tank lid positioned, it was possible to vary the water pressure in the bag to apply an uniform surcharge stress of between 0 and 200kPa to the surface of the sand. The lid and water bag are illustrated in Figures 5.2 and 5.3 respectively. The rubber sheet underneath the steel lid was secured by sandwiching it between a 50mm wide peripheral steel strip and the steel lid. The plan dimensions of the rubber were 50mm smaller than those of the internal dimensions of the tank. A calibrated Budenberg pressure valve was used to control the water pressure in the bag.

5.2.3 Application of Static and Cyclic Uniform Surface Loading
The water bag was only used during the buried pipe tests to apply a uniform static surface stress of between 50 and 150kPa for periods of 30 minutes, simulating approximately 2.7 to 8m of additional soil overburden respectively. To prevent excessive sand displacement, and hence rupture of rubber, at the corners additional support was provided by 300mm long by 115 (adjacent to the glass) or 145mm (adjacent to the steel) wide corner plates made from 50mm thick plywood (Figure 5.4). The plates provided 90mm of support to the water bag of which 50mm was beneath the steel clamping strip. The sides of these plates were tapered to prevent shearing of the water bag.
Only a static surface stress was applied during the initial tests since the water feed into the bag was controlled by a manually operated pressure valve. The results of preliminary tests to simulate cyclic loading by filling and emptying the water bag resulted in a minimum cycle period of approximately 11 minutes. This was considered both impractical and unrealistically slow in comparison with vehicle loading in practice. Modifications were consequently made to the water bag arrangement, including the fabrication of an inlet valve to replace the previously used manually operated pressure valve. A sinusoidal stress wave was used to represent the vertical stress wave applied to a buried pipe by traffic loading. This wave was generated by a Dartec signal generator which controlled the output of an air compressor valve. The modifications produced a cycle with a minimum period of 100 seconds, which represented the shortest time required for complete inflation and deflation of the bag via a water filled steel container.

5.2.4 Buried Pipe Installation

Section 4.4 described that the buried pipe experiments were conducted using two PVC-U and a vitrified clay pipe. Details of the cross-sectional dimensions, mechanical and thermal properties of these three pipe specimens are stated in Table 5.1. Each pipe specimen was cut to a length 995mm, 20mm shorter than the internal width of the tank (Figure 5.5). To permit easier installation a 30mm thick foam rubber plug was attached using a cynacrylitie based glue to the rear of each pipe specimen to fill this resulting void. The foam plug acted as a ‘spring’, which was compressed during pipe installation and held the front of the pipe against the glass. Although only providing a small nominal axial force, the plug ensured that no sand ingress occurred during the test. Polytetrafluoroethylene (PTFE) coated adhesive tape was attached to the front and rear ends of each pipe specimen to reduce the glass and steel/pipe friction respectively to a minimum.

It was important to establish square cut ends for each pipe specimen. The two PVC-U pipe specimens were initially cut to length using a hand saw, placed onto a flat steel
bench and any irregularities were removed by filing. The vitrified clay pipe was cut using a single pass of a circular diamond cutter.

5.2.5 Tunnel Simulation
To avoid scaling difficulties between the ground movements recorded with this apparatus and those recorded by Chapman (1992), the dimensions were chosen to correspond as closely as possible. The outside diameter of the inner (lining) and the two outer (shield) tubes were 194, 219 and 245mm respectively (close to those sought, see Section 4.4) and corresponding wall thicknesses of 5, 5 and 6mm. Details of the cross-sectional dimensions, mechanical and thermal properties for the inner (lining) tube are stated in Table 5.1. The final arrangements of these three tubes are shown in Figure 5.6.

Figure 5.7 shows a cross-section along the centreline of the tunnel apparatus. The length of the two outer tubes was 1350mm. The outer tube was supported outside the test tank by a single ‘Y’ shaped support, the height of which could be adjusted for use with both outer tubes (Figure 5.8). This support configuration allowed a maximum withdrawal distance of the outer tube of 590mm. The 1270mm long inner tube was restrained by a circular steel plate at its rear whilst the outer tube was withdrawn.

5.2.6 Tail Void Simulation
To simulate approximately the effects of a volume loss at the tail void of a tunnel shield, two concentrically placed tubes were abutted against the glass at the start of each test. This method allowed the total displacement field around the inner tube to be recorded as the outer tube was withdrawn. The initial investigation represented the simple case of a uniform annulus. Future work can be conducted to simulate tail voids of different shapes by adapting the equipment to allow the elevation of the inner tube to altered.

To establish a uniformly sized annulus throughout the withdrawal process, brass stud supports were attached along the length of the inside of the outer tube onto which the inner tube was placed. The dimensions of the studs were 18 and 22mm wide, and 8 and
19mm high for the 219 and 245mm diameter outer tubes respectively and were placed at 120° intervals from the invert. Each brass stud was attached to the inside wall of the outer tube by 3mm diameter flush mounted screws at 170mm intervals along its length. To avoid damaging the externally mounted strain gauge at the invert of the inner tube during installation or withdrawal of the outer tube it was offset circumferentially by 20mm. To ensure correct orientation and avoid rotation during installation and withdrawal, two 12mm wide by 5mm thick by 380mm long mild steel rails were attached to the invert of the inner tube (see Figure 5.6).

5.2.7 Modifications to the Test Tank - Inlet Details

To allow the tunnel experiments to be conducted it was necessary to form an access hole in the rear wall of the test tank. This initially involved cutting a 300 wide by 380mm high rectangular hole into the rear wall of the test tank. A 275mm diameter hole was cut into a 390 wide by 500 high by 10mm thick steel plate which was connected to the outside of the test tank by 12mm diameter bolts welded to the test tank. During the buried pipe tests a 5mm thick, 305 mm diameter steel plate was fixed over the 275mm diameter hole using 8mm diameter screws welded to the plate.

Sand leakage was prevented through the annulus between the 275mm diameter hole and the exterior of the outer tubes by using two white rubber seals sandwiched between the a steel ring and the test tank (see background of Figure 5.8). PTFE coated adhesive tape was attached across the inside wall of the rubber seals to minimise the friction between the outer tube and the rubber seals.

The annulus between the inner and outer tube was sealed to maintain a two-dimensional tail void loss of known volume. Foam rubber was attached on the inside wall of the 219mm outer tube to fill the 8mm wide annulus, the foam being tapered at the rear and coated with PTFE tape to facilitate installation of the inner tube. For the 245mm outer tube, the 19mm wide annulus was filled by using a 8mm thick rubber ring attached to the outer tube by a 6mm thick steel ring (see Figure 5.6).
5.2.8 Shield Withdrawal Equipment

After installation of the outer and inner tubes, the inner tube restraint and outer tube withdrawal systems were assembled (Figures 5.8, 5.9 and 5.10). Using an analogy of a steel pile embedded in a dense sand, the load required to withdraw the outer tube was predicted to be 19kN. A design load of 50kN was subsequently specified for the withdrawal equipment.

Two 1.1 m long 25kN capacity cables, separated by 140 mm, were attached to the outer tube by a single connecting ‘mask’ plate (see Figure 5.11) formed from a 260 by 300mm wide, 10mm thick steel sheet. The ‘mask’ plate was connected to the rear of the outer tube by use of 9mm diameter bolts screwed into 30mm long by 21mm diameter mild steel tubes welded to the outside wall of the outer tubes (see background of Figure 5.8). Two 30mm long by 45 external diameter tubes were welded to the rear of the ‘mask’ plate to allow the installation of the two 25kN load capacity bolts (see Figure 5.11) connected to the two 25kN cables. A 20 by 80mm rectangular hole was cut into the ‘mask’ plate to allow passage of the strain gauge wires to the datalogger.

The other end of the two 25kN cables were bolted to a 100 by 260 wide by 10mm thick connecting plate supported by a stand (see foreground of Figure 5.9). Onto the reverse of the connecting plate was attached a central 50kN capacity bolt for the connection of the pulling device. Initial trials were conducted using a 31kN capacity hand winch, which was deemed unsafe due to the transient variation of the stress application causing high peak loads. The withdrawal mechanism adopted consisted of using a wall mounted crane to apply a constant strain to the outer tube. The design of the reaction frame was initially based upon the use of the hand winch and the layout of 39kN load capacity laboratory floor bolts. Subsequent adoption of the crane as the pulling device required a 50kN cable and modifications to the reaction frame to include a 406mm diameter pulley wheel (see foreground of Figure 5.10) to translate the vertical to horizontal force. The weld arrangements for the pulley wheel and the connecting bolts for the attachment of the ‘mask’ plate to the outer tubes, were designed for a far greater force than 50kN.
5.2.9 Tunnel Lining Restraint System

The inner tube needed restraining against the glass while the outer tube was withdrawn. The layout of the restraint system for the inner tube is illustrated in Figure 5.7. To measure the effects of the soil load on the inner tube (after a stage of withdrawal of the outer tube), the axial loading applied by the restraint system to the inner tube would need to be removed. On this basis the connection to the rear of the inner tube was provided by a 10 mm thick, 194 mm diameter steel 'restraint' plate (see Figure 5.11), reducing to a thickness of 5mm for a 5mm wide annulus around the external circumference. The design of a free connection between the inner tube and the restraint plate was to allow simpler assembly and dismantling of the system.

The design of the restraint frame was governed by the arrangement of the two 25 kN capacity cables and the dimensions of the bolts used to attach the cables to the connecting plate. Due to the potentially large restraint forces, 100 by 100mm rolled hollow sections were used which resulted in a reduction in the maximum potential withdrawal distance to 840mm (see Figure 5.8). A restraint bar was used to connect the restraint plate to the restraint frame (see Figure 5.11). This bar comprises three sections, a 820mm long central section of 35mm diameter steel tube of 5mm thickness which at one end has a fixed 90mm length of 20mm diameter threaded bar and at the other has a detachable 410mm length of 20mm diameter threaded bar. The detachable length of threaded bar, which was passed through the restraint frame, permitted the assembly of the restraint frame.

5.2.10 Filling and Emptying the Test Tank

The procedure for filling the test tank was the same as that used previously by Chapman (1992), involving a Floveyor auger to transport the sand from a skip into the test tank. The auger uses plastic discs placed adjacent to the hopper of the auger to generate an air draught which carries the sand particles from the hopper into the test tank. Whilst the technique involves minimal contact between the rotating plastic discs and the sand, dust is generated. The dust was removed by a dust extractor placed adjacent to the auger. A polythene tent arrangement was placed on top of the test tank.
to restrain the dust generated within the test tank and was only removed once the dust within the test tank had settled. The general arrangement of the auger and the polythene tent are shown in Figure 5.12. This technique allowed large quantities of sand to be moved, which was dispersed within the tank using a sand raining device (see Figure 5.13). The sand disperser was designed by Chapman (1992) to distribute the sand as evenly as possible from a single point of entry from the auger to obtain a uniform density. Some mounding occurred however, directly beneath the base holes of the pyramid and marginal deficiency of sand was noted around the periphery. The method used to attain the different densities of sand using this equipment is described in Section 5.5.

The sand was manually excavated from the test tank as access problems within the test tank prohibited the use of the auger. To minimise the detrimental effects of the dust generated by this operation, the excavation was conducted in the evening when the laboratory was empty and the generated dust was removed by the extractor.

5.3 INSTRUMENTATION

The approach adopted for the experiments was determined by the need to record photographically the behaviour of the buried structure and the surrounding soil. The development of a digitising method to determine the soil and structure displacements is described in Section 5.3.1. The use of strain gauges attached to the buried structures to record their response to the applied loading is described in Section 5.3.2.

5.3.1 Measurement of Soil-Structure Displacements

The glass panel allowed the position of individual sand particles and the cross-section of the buried structure to be recorded photographically (see Figure 5.14). Stereophotogrammetry and digitising techniques were available to determine the displacements of the individual sand particles and the cross-section of the buried structure from the recorded photographs. A third method in which the negatives of photographs were scanned to represent a digital image was also available and would have required the use of markers placed adjacent to the glass within the sand to
determine soil displacements. Each method was assessed according to its accuracy, speed, development time and cost.

5.3.1.1 Stereo-Photogrammetry
The stereo-photogrammetry technique was used by Chapman (1992) and is described in further detail by Butterfield et al (1970). The technique involves taking photographs from the same camera position, before and after the movements have occurred. Once developed, sequential photographs are placed alongside each other beneath a stereo-viewer. Whilst viewing both photographs, the operator adjusts their position until a three-dimensional image is formed. From this image, the two-dimensional plan location of an object can be measured and the third dimension represents the relative displacement. Cording et al (1976) and Chapman (1992) estimated that the overall precision of this technique was ±0.15 and ±0.10 mm respectively.

5.3.1.2 Digitising Method
The principle behind the digitising method is that the location of an object on a photograph, attached to a digitising board, can be recorded by the use of a cursor placed above the object relative to a co-ordinate system installed within the digitising board. The location of each selected object is stored in a computer file through a controlling software package, for example ‘Prosurveyor’. As Section 5.3.1.4 describes in further detail, a transformation process is required to convert the measured locations of each object, based upon the co-ordinate system within the digitising board, to the real location. The precision of the measured co-ordinate of an object from the digitising board is ±0.01mm. The precision achievable with the transformation process was typically ±0.1mm resulting in a theoretical accuracy of the same order as the stereo-photogrammetry technique. In practice, both techniques require the manual identification and location of an object. An advantage of the stereo-photogrammetry technique over both the digitising and scanning methods is that there is a direct connection between successive photographs.

An investigation was undertaken to determine the expected precision of the digitising method by recording the location of eight sand particles eight times from a photograph.
taken during a buried pipe test. Variability in the sample readings would therefore arise from the measuring technique itself. A sample of readings has a standard deviation, $S_x$, defined as:

$$S_x = \sqrt{\frac{\sum(x_i - \bar{x})^2}{N-n}}$$  \hspace{1cm} (Eqn. 5.1)

where

- $N$ - Number of independent readings
- $n$ - Number of restrictions
- $x_i$ - Recorded value
- $\bar{x}$ - Mean of actual observations

Provided that the number of readings is large ($N>30$), then the errors in the sample can be described by a normal distribution curve. After transforming the measured co-ordinates to their absolute values, the standard deviation for the $x$ and $y$ co-ordinates were ±0.12 and ±0.14mm respectively based upon values of $N=64$ and $n=8$.

To determine how reliable the sample mean ($\bar{x}$) is with respect to the true mean it is necessary to determine the standard deviation of the mean of the sample, defined as the standard error of the sample mean ($S_{\bar{x}}$) given as:

$$S_{\bar{x}} = \frac{S_x}{\sqrt{N}}$$  \hspace{1cm} (Eqn. 5.2)

A hypothesis can be stipulated that with 95% confidence, the true mean ($\mu$) lies within the limits defined as:

$$(\bar{x} - 1.96 S_{\bar{x}}) < \mu < (\bar{x} + 1.96 S_{\bar{x}})$$  \hspace{1cm} (Eqn 5.3)

In addition, it is necessary to determine the difference between two true means at different steady states ($\mu_1 - \mu_2$). To state whether this difference is due to an external source or to the spread of the sample data, it is necessary to apply the same 95% confidence limits as before to the difference of two sample means ($\bar{x}_1 - \bar{x}_2$) given as:

$$(\mu_1 - \mu_2) = (\bar{x}_1 - \bar{x}_2) \pm 1.96 \sqrt{\frac{S_{x1}^2}{N_1} + \frac{S_{x2}^2}{N_2}}$$  \hspace{1cm} (Eqn 5.4)

where $S_{x1}$ and $S_{x2}$ are the standard deviations of the two samples respectively.
\(N_1\) and \(N_2\) are the number of readings for the two samples respectively.

The precision of a co-ordinate based on a single reading (i.e. \(N=1\)) is calculated from equations 5.2 and 5.3 with a 95% confidence level as \(\pm 0.27\)mm and the corresponding precision of a displacement based on a single reading is therefore \(\pm 0.39\)mm using equation 5.4.

### 5.3.1.3 Scanning

The scanning method represents a completely computerised process of identifying the location of objects recorded photographically. This approach has been used successfully in the field of geotechnical engineering research by, amongst others, Grant and Taylor (1996). The computerised nature of this technique would be expected to result in an increase in accuracy and speed of analysis when compared to the two techniques described previously. The following description of the technique follows that used by the author during a preliminary investigation. Photographs were taken in the manner described in Sections 5.3.1.4 and 5.3.1.5 during a buried pipe test with a 35mm Single Lens Reflect (SLR) camera and Kodak 100ASA film. The negative of the film was then converted into a digital image, represented by a series of pixels, and stored on a CD-ROM. After transferring the image files onto the hard disk of a personal computer, a digital image software package 'Visilog' was used to manipulate the digital images. Each pixel of a digital image was represented by 255 shades of grey ranging from one, representing black, to 255, representing white. The successful application of this method relies upon a contrast of brightness between the objects to be measured and the remainder of the image. In the context of this project it was desirable to record the position of individual sand particles and specific locations on the cross-section of the buried structures as images representing consecutive stages of a test. A trial was conducted to examine whether individual particles of Leighton Buzzard sand could be used as natural markers. Two images were selected representing consecutive stages of a test recorded from the same camera with similar lighting arrangements. The same criteria of brightness was selected for these images, for example 170, so that only very bright and nearly white pixels were selected. The resulting image highlighted the desired white targets points with respect to a black
background. The resulting digital images showed that although the sand particles did act as natural markers, the use of the same brightness criteria for each image did not result in the complete selection of identical sand particles. The slight variation in both the positioning of the externally mounted lights and the fluctuation in the brightness of the ambient light within the laboratory during the duration of the test led to an inability to reproduce photographs with exactly the same brightness, resulting in the selection of different individual sand particles. The lack of continuity between consecutive images resulted in the conclusion that man-made markers, e.g. glass beads, must be placed into the soil medium.

It was an aim of this research project to avoid the use of artificial markers in the soil medium since they would potentially influence the soil displacements. Although many researchers have successfully implemented such a technique, their installation procedure typically involved a test tank that could be rotated through 90° so that the glass panel (through which the markers were viewed) was temporarily removable on completion of the filling operation to allow the markers to be accurately placed. This was not achievable with the laboratory arrangement used for the current work. Markers would therefore need to be positioned at the glass-sand interface during the filling process. Such a procedure was adopted during the tunnel test T2 but it was deemed an unnecessary compromise of the installation procedure during the buried pipe tests in which it was intended to simulate accurately the construction process. In addition, markers would have resulted in a coarse measurement of the displacement patterns. For example preliminary buried pipe tests, analysed using the digitising method, had indicated a small zone of soil shear emanating from, and at, the pipe-soil interface. It was considered that the use of markers would not have indicated this.

In reviewing the three methods, it would appear that none of them totally fulfils the needs of this project. The scanning method is potentially the most precise method of measurement as it does not involve manually locating the position of the objects. It would after a period of development be the quickest method, but in view of the drawbacks stated above was rejected. The standard deviation of the transformed co-ordinates using the digitising method compared favourably with the precision
achievable using the stereo-photogrammetry technique (±0.15 mm), but applying confidence intervals of 95% to a single reading increased the achievable precision of the digitising method to twice this value. Chapman (1992) noted that a major limitation of the stereo-photogrammetry technique was that it was a very laborious technique because the position of each sand particle had to be recorded manually. Although the digitising method also requires the position of individual sand particles and the cross-section of the buried structure to be located manually, the co-ordinates are stored automatically on a computer file from the digitising board. In view of the number, size and stages of the tests that were planned it was deemed that this was a very significant advantage over the stereo-photogrammetry technique. The digitising method was duly selected, but because the technique was not well established within the department it had to be customised for use within this project.

5.3.1.4 Procedure used for the Digitising Method
Prior to filling the test tank with sand, a reference grid comprising of 2mm crosses was drawn on the inside face of the glass panel at 100mm horizontal and vertical intervals. The reference points were covered by reflective tape to protect the grid markings during pipe installation and to allow illumination throughout the test. Reference points were established around the circumference of the pipe wall by attaching, on top of reflective tape, PTFE coated adhesive tape in approximately 5mm wide sections with 1mm gaps. This gap was sufficiently small to ensure that no sand leakage occurred between the pipe and the glass but was sufficiently large to create a reference point at the boundary between the two types of tape and the internal structure (i.e. pipe or tunnel) wall.

The developed photographs were sorted according to test number, stage of testing and recorded region of the subject area. For a particular test and region, the photographs were attached to the digitising board in sequential order. The position of the reference grid on the inside of the glass panel was located on each photograph using a cursor according to an arbitrary co-ordinate system created by a software package within the digitising board. In sequential order, the co-ordinates of the reference grid, structure (i.e. pipe or tunnel) wall and individual sand grains were located and stored in a
computer file. This process was carried out to record, at least twice, each reference point around the circumference of the inside wall of the structure (i.e. pipe or tunnel) and approximately 40 sand particles per 100mm square section on the photograph. Although the size of a sand grain is reduced on a photograph, individual grains can be easily located since the multi-coloured nature of the sand allows certain grains to appear as discrete targets. A consistent approach was adopted during the measurement of the sand grains which entailed, for example, recording the top left corner of each grain.

A computer program was written to sort the co-ordinates of each of the measured objects into separate stages of the experiment. For each photograph, a transformation process described by Albertz and Kreiling (1980) was incorporated into a further computer program to convert the measured, arbitrary grid, to the known 100 by 100mm reference grid. The transformation process involved, for each of the x, y co-ordinates, a translation, rotation and scale factor. From a knowledge of the measured and actual reference grid co-ordinates for each photograph, six parameters were calculated by the computer program relating to the three parts of the transformation process and the two planes. This process was subsequently used to transform each of the measured structure (i.e. pipe or tunnel) wall and sand grain positions to the known grid and thus find their real co-ordinates. Relative displacements between each stage of testing could then be calculated as the location of each point on the structure (i.e. pipe or tunnel) wall or an individual sand grain was recorded in sequential order.

5.3.1.5 Errors Inherent in the Digitising Method
An inherent problem occurs when attempting to record small displacements since errors arising represent a significant proportion of the recorded value. Chapman (1992) identified four sources of error associated with the stereo-photogrammetry technique: alignment of the cameras, production of the photograph, measurement from the photograph and reduced structure and sand displacement due to friction of the sand against the glass. Each of these sources of error are applicable to this digitising method and are discussed hereafter.
The position of the camera is critical to the success of measuring the structure (i.e. pipe or tunnel) wall and sand displacements. The camera arrangement used by Chapman (1992) was adopted for this project (see Figure 5.1). Each of the three 35mm SLR cameras used to photograph the respective subject areas was attached to a horizontal steel plate which was in turn attached to a rigid boom mechanism connected to the test tank. The cameras were numbered to ensure that the same camera was used for a particular subject area throughout a test. The procedure used to ensure that the camera was exactly perpendicular to the glass involved adjusting the boom mechanisms to ensure that the plan location of each of the cameras was the same using reference marks established on the laboratory floor sighted through the holes in the plates. In addition, the boom mechanisms were checked with a spirit level and the distance from the glass was measured. Each boom mechanism was arranged so that when the camera was attached to them, they typically focused upon a central reference grid point. Handling of the cameras during a test was kept to a minimum to ensure that the cameras were in a fixed position between photographs. This was aided by the use of remote shutter releases and motor winds. The cameras were only touched during a test to adjust the focus and exposure times for each of the three photographs recorded at each stage of a test. However, the duration and number of stages involved for many of the tests meant that the 36 exposure film had to be replaced periodically and the camera had to be removed for safe storage overnight.

Optical distortion can occur if the object being photographed is in different parts of the field of view. This problem is reduced by taking measurements from the centre of the photograph and using good quality photographic equipment, which for these experiments was manufactured by Canon and Minolta. Film distortion, whether inherently caused during film manufacture, or due to alignment or during printing was reduced by the use of good quality film processed using high quality automatic equipment. The effects of distortion on the measured displacements were observed during the early part of the experimental programme, resulting in modifications to the camera set-up and data analysis. The boom mechanisms used for the preliminary investigations and the first four experiments listed in Table 5.9, were the same as those used by Chapman (1992). To view the complete width of the 400mm wide subject...
area, the position of each of the booms, and thus the cameras, had to be adjusted for each stage of the tests. Problems arising from the constant adjustment of the camera position became evident in a few of the displacement vector plots. For this camera arrangement additional sources of error were minimised however, particularly with respect to camera distortion, by providing overlapping fields of view and recording only measurements from the centre of the photograph.

For the remaining tests, the boom mechanisms were modified to view the complete 400mm wide subject area and thus were not adjusted during a test. Due to the size of the cameras, it was not practically possible to provide also overlapping fields of view. A compromise was established in determining the most effective camera distance, between the size of the individual sand particles observed on the resulting photographs and minimising the effects of optical distortion. Subsequent digitising of the resulting photographs revealed continuity problems between the displacements recorded by different cameras when the transformation process was based upon the use of the four extreme corner reference grid points. The reasons for this were due to the effects of optical distortion because the reference points, on which the transformation process was based, had become closer to the edge of the photographs when compared with photographs taken from the previous camera position. To reduce the effects of optical distortion, the complete transformation process for each camera was based upon eight transformation processes, using the eight 100 by 100mm spaced reference grids on each photograph. This procedure was used for each of these nine tests, except the soil filling stages of Test 250WPD1^ (the set of photographs originally indicating this problem).

Whilst difficulties have been shown during the use of this measuring technique with the original camera arrangement and original analysis technique for the second camera position, the results from these tests have been included. This is because the errors associated with these displacements typically involved continuity problems at the edges of the photographs for stages of a test where the observed magnitude of displacement approximated to the error inherent for the measurement technique. For these initial stages of a test when the sand is being placed into the test tank, only the measurement
of the pipe displacements is considered in the discussion of the results. The central positioning of the pipe within the subject areas explains why continuity is illustrated between photographs.

Additional inaccuracies in the measured displacements may result from refraction of light through glass. This error was calculated using the derivation outlined by Chapman (1992), see Appendix 1. A range of possible errors was calculated using a spreadsheet for typical and maximum measured displacements, indicating a maximum error of 0.2 mm for 30 mm thick glass.

Opportunities to assess the accuracy of the complete photographic technique (i.e. the position of the camera between photographs, the development process and the measurement technique) were available throughout the experimental programme. For example, no movement would be expected for the buried pipe and the surrounding soil overnight when subjected to negligible loading. Before and after photographs were analysed independently as part of the programme and any differences could be attributed to errors. Similar opportunities were also available during the initial stages of the tunnel withdrawal, where the expected sand displacements were limited to a small zone above the tunnel. These tests showed that the manual nature of the measurement technique was the dominant influence upon the inaccuracies. The sand particle displacements were typically based upon a single observation. Accordingly by adopting the precision of a single displacement as ±0.39 mm (see Section 5.3.1.2), a minimum measured displacement of ±0.5 mm was used for plotting the contours of displacement.

For each of the buried PVC-U pipe tests the displacement of the pipe crown, invert and springline were used to determine the overall pipe displacements and diametral strains. Clay pipe distortion could not be measured and so only overall displacements were found. Table 5.2 shows the minimum number of measurements made at each of these positions. Using a standard deviation of ±0.13 mm, the procedure described in Section 5.3.1.2 was repeated to produce a (smaller) precision value for the measured
displacements, which was in turn used to determine the precision of diametral strain for each buried pipe experiment (see Table 5.2).

5.3.2 Strain Gauge Measurement
To record changes in the response of the buried structures too subtle to be discerned using the displacement technique, strain gauges were attached to the internal and external wall of the pipe and inner tube specimens. Circumferential strain gauges were used to indicate the shape of the structure during external loading. In addition, axial strain gauges allowed an interpretation of whether plane strain or plane stress conditions applied. Valsangkar and Britto (1977 and 1978) demonstrated that these measurement could be coupled with those of externally mounted circumferential strain gauges to determine the hoop stress ($\sigma_h$), hoop thrust ($T$), and bending moment ($M$), in the structure wall from the following equations:-

$$
\sigma_h = \frac{(\varepsilon_0 + \varepsilon_1) E}{2} \quad \text{(Eqn. 5.5)}
$$

$$
T = \sigma_h \cdot t \quad \text{(Eqn. 5.6)}
$$

$$
M = \frac{(\varepsilon_0 - \varepsilon_1) E t^2}{12} \quad \text{(Eqn. 5.7)}
$$

To convert to plane strain conditions equations 5.1 to 5.3 are multiplied by:

$$
\frac{1}{(1 - \nu^2)} \quad \text{(Eqn. 5.8)}
$$

where

- $\varepsilon_0$ = Circumferential external wall strain ($\mu$)
- $\varepsilon_1$ = Circumferential internal wall strain ($\mu$)
- $t$ = structure wall thickness (m)
- $E$ = Elastic modulus of structure (kPa)
- $\nu$ = Poisson's ratio of structure

The derivation of these equations was given by Potts (1977) and is repeated in Appendix 2.

A TA880 datalogger manufactured by C.I.L. Electronics was used in conjunction with an eight-channel electronic switch, manufactured within the Department, to measure the sixteen strain gauges. The circumferential strain gauges were mounted on the test
specimens 15mm from the end of the specimen adjacent to the glass. As uniaxial strain gauges were used, the axial strain gauges were mounted on the test specimen adjacent to, and behind, the circumferential strain gauges. Ideally, the gauges would have been placed at mid-distance along the specimens to reduce any end effects. For all these tests, however, except for Test 250WPL1*, it was necessary to compare the displacements and the test specimen response in the same plane.

The sixteen strain gauges were attached to the test specimens in four arrangements denoted as A, B, C and D and shown in Figure 5.15 and Table 5.3. These arrangements are listed in chronological order and indicate how the location of strain gauges was reviewed on the basis of the experimental results. The initial arrangement (A) aimed to record the hoop response of the external wall of the pipe. Arrangement B aimed to record the hoop response of the internal and external walls of the pipe, in addition, to measuring the axial strain response to assess whether plane strain or plane stress conditions applied. Arrangement C provided a second measurement of the wall thrust at the structure springline, whilst retaining measurements of axial strains at the springings, as this measurement became an increasingly important aspect of the project. Arrangement D was used for Test 250WPL1* to measure the effects of friction at the glass interface by comparing the responses at both the glass interface and mid-way along the pipe. From these arrangements it can be seen that there was no measurement of wall thrust at the springline for the initial arrangement (A), a single measurement on the RHS springline (90°) for second arrangement (B), on both springline positions during the third arrangement (C), and at both springline positions at the front and mid-length sections for the final arrangement (D).

Uniaxial foil type electrical resistance strain gauges were used throughout the tests attached using Loctite ‘757’ cynacrylate glue for the PVC-U specimens, ‘aralidite’ two-part epoxy resin for the vitrified clay specimen and a ‘P2’ two-part polyester based glue for the steel specimens. The use of a dry sand prevented the need for waterproofing. The thermal coefficients of expansion of the strain gauges were matched closely to those of the material on which the gauges were attached, with the
exception of the vitrified clay pipe, where a difference of thermal coefficient of expansion of $3.7 \times 10^{-6} \, ^\circ C^{-1}$ existed.

The measured strain, for each gauge, was calculated by the datalogger on the basis of the gauge factor stated by the strain gauge manufacturer, the input voltage across the wheatstone bridge arrangement of 1 volt and the use of single 'active' strain gauge configuration for the wheatstone bridge arrangement. These values were written to a computer file for storage. The input voltage was set at 1 volt throughout the experimental program. The voltage was supplied using a Farnell power unit, except during experiments 168WPL3.5, 120WPL2.5 and T2 where a second TA880 datalogger was used. A wheatstone bridge circuit was constructed within the department based upon a single active and dummy gauge arrangement for the measurement of eight channels. Dummy gauges, consisting of eight strain gauges attached to a piece of the test specimen, were placed adjacent to the wheatstone bridge circuit.

The externally mounted strain gauges were protected by a thin sheet of perspex placed over the gauges and fastened down by insulating tape. This sheet was articulated by a series of slots so that it followed the contours of the strain gauges. Connecting wires to the externally mounted strain gauges were attached to the outside of the test specimens by insulating tape and for the pipe specimens, fed through a hole in the foam rubber plug (see Figure 5.5). The wires connected to the internally mounted strain gauges were attached to the inside wall of the test specimen. At the rear of the test specimens these connecting wires were attached to connecting plugs. Prior to installation of the test specimens, these plugs were connected to two separate lengths of cable which were fabricated to pass through holes in the 10mm thick, 305mm diameter steel plate (see Section 5.2.7), thereby forming a connection between the strain gauges and the datalogger.

5.3.2.1 Calibration of Strain Gauge System

Calibration of the strain gauge system involved subjecting the test specimen to a line load test under parallel plate loading. Strain gauge measurements were taken
throughout each test, for PVC-U pipe specimens they were compared with values predicted from the vertical deflection of the specimen, and for the rigid pipes with values based upon the applied line load (see Appendix 3).

The cross-sectional dimensions, mechanical and thermal properties of the four test specimens used in the model buried pipe and tunnel tests were presented in Table 5.1. In addition to these specimens, line load tests were also conducted on lengths of PVC-U and vitrified clay pipe shorter than that of the bearing plate of the testing machine. The ratio of the pipe radius to wall thickness (R/t) of each of the flexible pipe specimens (i.e. excluding the vitrified clay pipes) was in excess of eight and thus in the predictions of strains in Tables 5.4 and 5.5, thin-wall theory was assumed (according to thin-wall theory it is assumed that the distribution of strain across the pipe wall is linear so that the neutral and centroid axes are coincident). This approach is therefore not strictly appropriate for the two vitrified clay pipes, with R/t ratios of six. Using formulae presented by Young (1989), the distance of the neutral axis from the centroid axis was calculated to be approximately 0.2mm for the clay pipes on the basis of a solid rectangular section of thickness d at a radius r. Assuming a linear distribution of strain across the pipe wall, the difference between predicted strains based upon thin-wall theory and the calculated location of the neutral axis was ±3με.

Tests were conducted using a Dennison '6025' universal testing machine to establish for each gauge a linear relationship between either applied load, or deflection, and measured strain. The strain gauge readings were shown to be correctly orientated, positioned and bonded to the specimens. The measured strains in Tables 5.4 and 5.5 are the average of at least two test series conducted up to six months apart. As the results were comparable, they indicate that there were no measurable creep effects of the gauges. In view of the successful calibration of the test specimens, the main aim of discussing the results is to assess whether the attachment of the strain gauges to the specimens resulted in a localised stiffening effect (an effect also investigated by conducting tensile tests, see Section 5.3.2.2).
Deflection based parallel plate tests - PVC-U pipes

A Linear Variable Differential Transformer (LVDT) in the cross-head of the machine enabled the vertical displacement of the bearing plate to be recorded to ±0.1mm. A small seating load of approximately 0.1kN was applied to the pipe and datum readings of strain and vertical deflection were taken. A vertical deflection was then applied at a rate of 12.5mm/min in increments of VDS of 1% up to a maximum of 5%. At each increment, the vertical deflection was maintained for 1 minute to allow a minimum of thirty measurements of the strain gauges to be taken, after which the pipe was unloaded at the same rate.

In order to calibrate the two, 995mm long PVC-U test specimens it was necessary to distribute the point load from the bearing plate of the test machine uniformly along the full length of the pipe crown. This was achieved by clamping a 1.1m long, 50mm wide mild steel channel centrally to the bearing plate.

The relationship between measured internal circumferential wall strain and vertical deflection is shown in Figures 5.16 and 5.17 for the 250mm diameter pipe from strain gauge measurements taken at both close to one end and midway along the pipe respectively, and in Figure 5.18 for the 168mm diameter pipe. Table 5.4 shows the measured and predicted circumferential strains at the springline of the 250 and 168mm diameter PVC-U pipe specimens, based on vertical deflections of 10 and 5mm respectively, using measurements at a VDS of 1% as a datum.

Figures 5.16 and 5.17 show that the measured values of springline strain at the front section for the 250mm diameter pipe were comparable to those at the mid-length, which was also shown by the back-calculated values of elastic modulus (Table 5.4). These figures, in addition to Figure 5.18, also show that the measured strains at the springline of both the 995 mm long sections were all consistently lower than the predicted values. This is highlighted in Table 5.4 by the back-calculated values of elastic modulus, which are all correspondingly greater than the assumed value for PVC-U of 2.8GPa. The over-prediction of elastic modulus corresponds to the consistent recording of much larger strains in the vertical plane (0° and 180°)
compared to those in the horizontal plane (90° and 270°). This behaviour is explained by the stress concentration that would arise from the use of a 50mm wide channel bearing plate on the pipe crown, which is more prominent for the 168mm diameter pipe due to the larger applied loads.

To investigate whether the difference between the measured and predicted strains was a feature of the testing procedure, a 200mm length of 250mm diameter, which could be placed completely beneath the 300mm diameter circular bearing plate of the testing machine, was tested. Similar strain gauges were used but were attached using an alternative cynacrylate glue (termed Glue 2) to the Loctite '757' cynacrylate glue used for the test specimens (termed Glue 1). The results (Table 5.4) show negligible differences between the two glues and the difference between the measured and assumed values of elastic modulus was approximately 3%. It can be concluded that the measurements agree with those based upon thin wall theory, indicating that the attachment of strain gauges did not stiffen the pipe.

Load based parallel plate tests - Vitrified Clay and Mild Steel

Calibration of the strain gauges attached to the 995mm long, 120mm diameter and 300mm long, 180mm diameter vitrified clay pipes and the 1270mm long, 194mm diameter mild steel tube was similar to the deflection based tests except that a predetermined load rate (0.4kN/m/s) was adopted according to BSI (1991). Similarly to the deflection based tests, a specimen shorter than the bearing plate was also tested. The strain gauges were attached to the 300mm long vitrified clay specimen using the polyester glue (termed Glue 4) used for the mild steel tube, in addition to the epoxy resin glue (termed Glue 3) used for the 995mm long vitrified clay pipe. A three-edge bearing test was conducted, using a 1010mm long, 150mm by 100mm by 10mm mild steel rolled hollow section to distribute uniformly the applied load along the pipe crown.

The measured and predicted circumferential strains at the springline of the vitrified clay and mild steel pipes were based upon an applied load of 20.7kN/m for the 300 mm long, 180mm diameter vitrified clay specimen and 8kN/m for the other two specimens.
The applied load of 20.7kN/m represents 26% of the yield stress of the 180mm diameter vitrified clay pipe, whilst the applied load of 8kN/m represents 12 and 14% of the yield stresses (42 and 250MN/m²) for the 120 and 194mm diameter vitrified clay and steel pipes respectively. The results show excellent correlation between the predicted and measured circumferential strains for all of the three specimens based on a minimum of two test series conducted up to six months apart. Comparison of these results indicated that no creep problems were experienced with the readings and a return to the initial datum on removal of the applied load was always observed.

5.3.2.2 Tensile Tests of PVC-U Specimens

Perry (1985) reports that both experimental and analytical studies have shown that the stiffness of a strain gauge can produce a significant reinforcing effect if the gauge is installed on a material with an elastic modulus, of 7GPa or less. As this is analogous to a composite beam situation, the reinforcing effect is dependent upon the elastic modulus and thickness of the member onto which the gauge is attached, and the length of the strain gauge. Perry states that in tests using gauges on plastics, errors ranging between -10 and -30% had been reported. The reinforcement by the gauge can be characterised as either local or global, or a combination of both. The reinforcement is defined as local when the cross-section of the member at the gauge location is great enough that the contribution of the gauge to bearing the applied bending moments or in-plane loads is negligible.

Twelve, 225mm by 30mm wide rectangular-shaped tensile test specimens were fabricated from an offcut of the 250mm diameter PVC-U pipe used during the buried pipe model tests. They were each tested in accordance to the methods stated in BSI (1976) to determine the tensile strength, elongation and elastic modulus of the plastic. The dimensions of the specimens was determined to allow the same single, 14mm long by 5mm wide uniaxial electrical strain gauges and crynacrylite glue as those used for the buried pipe model test specimens to be attached in both the transverse and longitudinal directions. To assess the effect of increased stiffness from both the glue and the strain gauges, two strain gauges were attached to each of four samples, four of
the samples were coated with glue only (over the same representative area) and four were used as control samples. Each of the samples were placed within a Lloyd's '2000R' tensile test machine at a distance of 120 to 140mm between the edges of the two grips and tested to a rate of grip separation of 1mm/minute. A 10kN load cell was used based upon a peak load of 4.45kN calculated for the 2.7mm thick specimens from the tensile strength of PVC-U of 55MPa. An extensometer was clipped centrally onto each specimen to measure the extension over a 50mm length. Measurements of load and extension of the extensometer were recorded in the form of a graphically produced relationship. During the strain gauge samples tests, the load was recorded at time intervals correlating with those of the strain gauge readings.

The measured elastic moduli are shown in Table 5.6. Figure 5.19 shows typical results of axial stress and axial strain for the three types of specimen used. The tangent elastic modulus was determined from typical axial strain of 0.3%, or an extension of 0.15 mm, corresponding to an axial stress of about 9MPa, or 729N. Some inaccuracy of determining the gradient occurred due to the slippage of the extensometer which led to the typical 'S'-shaped curve shown in Figure 5.19.

The four control specimens had an average elastic modulus of 2.9GPa, which compares favourably with the assumed value of 2.8GPa. With glue only, the modulus increased by 0.5GPa above that of the control samples. Based upon the extensometer readings, the attachment of the strain gauges resulted in a similar increase in modulus to that observed with glue only. The results from the strain gauges, which it can be assumed experienced no comparable slippage, produced an average value of 3.7GPa. It can be concluded that the attachment of the strain gauges resulted in a localised stiffening effect of approximately 15 and 28% based upon extensometer and strain gauge measurements respectively. Although further tests would be required to investigate the influence of the size of the tensile specimen and hence enable the localised nature of this effect to be assessed, the results of the line load tests indicate that it is not sufficient to influence the overall behaviour of the pipe or the measurements from the strain gauges.
5.3.2.3 Precision of Strain Gauge System

During the calibration and test tank experiments, measurements of strain gauges were recorded a minimum of thirty times at a steady state of applied stress. Table 5.7 shows the calculated standard deviations and standard errors for these tests on the basis of a minimum of ten sets of thirty readings from random strain gauges at random stages during a test at a steady state of applied stress and a single restriction (i.e. \( N=30 \) and \( n=1 \)) using equations 5.1 to 5.3. The variability between the values of standard deviation from these sets of samples was shown to be negligible for each of the tests. A maximum standard deviation of \( 140 \mu \varepsilon \) was recorded during four tests in which a particular analogue Farnell power supply was used. Use of a different analogue power unit reduced the value to \( 28 \mu \varepsilon \) and use of a digital supply via a second TA880 datalogger reduced it further. The reliability of the power unit is therefore an important factor in the precision attainable.

The precision of the measured strains due to a change in applied stress are determined from these standard deviations using equation 5.4. Table 6.4 shows the range to be \( \pm 71 \mu \varepsilon \) when using the Farnell power supply and a maximum of \( \pm 14 \mu \varepsilon \) for the remaining tests. Using equations 5.5 and 5.6, these values of difference in means were then used to determine the precision of the hoop thrust. A maximum variability in hoop thrust was shown to be \( \pm 0.4 \text{kN/m} \) for the tests with a standard deviation of \( \pm 140 \mu \varepsilon \) and a maximum of \( \pm 0.08 \text{kN/m} \) for the remaining plastic pipe tests. Due to the higher elastic modulus of the vitrified clay pipe, the precision of the strain measurements of \( \pm 71 \mu \varepsilon \) resulted in a precision of thrust of \( \pm 35 \text{kN/m} \) rendering the measured values of hoop thrust insufficiently significant.

5.4 SOIL CLASSIFICATION

A, Leighton Buzzard, single-sized sand was used throughout the test programme. The particle size distribution for this sand (Figure 5.20) shows it to be uniformly graded sand with \( d_{10} \), \( d_{30} \) and \( d_{60} \) values of 1.18, 1.37 and 1.65 mm respectively. Table 5.8 presents the soil parameters gained from other standard soil tests (BSI 1980b, 1990). The effective internal angles of shearing resistance (\( \phi' \)) were obtained by conducting tests using a 300 by 300mm shear box over the stress ranges used in the tank.
experiments. The effective dilation angle (\(\phi\)) was calculated using the formula proposed by Bolton (1986):

\[
\phi' = \phi'_{\text{critical}} + 0.8\phi
\]  
(Eqn. 5.9)

which relates the angle of shearing for the sand at its peak (\(\phi'\)) and critical (\(\phi'_{\text{critical}}\)) states.

5.5 SAND DENSITIES

The literature review highlighted the sensitivity of the behaviour of the buried structures to the soil surround. It was necessary, therefore, to obtain consistent soil densities both within and between tests. Two broad states of soil density were used in these tests, termed 'loose' and 'dense'. To assess the influence of soil stiffness on the behaviour of buried pipes the thickness and compactive effort applied to the sand layers forming the sidefill and backfill were varied (Table 5.11).

Both the loose and dense states were achieved using a sand raining device described in Section 5.2.10, which was raised throughout the filling operation to maintain an approximately minimum constant drop height of 100mm. Each horizontal layer of sand was carefully levelled using a rake operated from outside the test tank. The dense state was achieved by tamping each layer of sand after it was placed. In general, 250mm thick layers of sand were compacted using a 10kg steel tamper, having a base size 200 by 200mm, dropped from a height of 250 to 400mm. A similar sized 1.5kg wooden tamper was used to compact the sand adjacent to the glass after a single pass of compaction had been applied using the steel tamper to avoid breakage. Four passes were applied to each layer in a consistent pattern. During Test 250WPD1, the sidefill zone was placed in 125mm thick layers and compacted with four passes of the 10kg steel tamper. To reduce the observed negative vertical diametral strain (VDS) during Test 250WPD1, a repeat test was conducted (Test 250WPD1^) in which the sidefill and backfill were placed in 50mm layers and compacted with four passes using the 1.5kg wood tamper.

Separate tests were conducted without the presence of a buried structure or surface stress to determine the initial in-situ density of the sand placed into the test tank in
accordance with the dense and loose states. Three metal containers, with a minimum volume of 2651cm$^3$, were placed into the test tank prior to the filling operation in which the sand was placed in 250mm thick layers. On completion of the filling operation the metal containers were carefully excavated by hand. The density of the dense and loose states were measured at 1644 and 1522kg/m$^3$ respectively, representing 92 and 85% respectively of the maximum density of 1801kg/m$^3$.

5.6 BURIED PIPE TEST PROCEDURE

The procedure used for the buried pipe tests was separated into three steps: pipe installation, placement of the sidefill and backfill soil material and the application of surface stress.

5.6.1 Pipe Installation

To achieve the appropriate cover depth to pipe diameter (C/D) ratio for each test, a horizontal line was drawn onto the inside face of the glass to indicate the proposed bedding level of the pipe specimen. Leighton Buzzard sand was placed into the test tank to form the bedding in 250mm thick layers and compacted from within the tank to achieve a dense state, as described in Section 5.5. The pipe was placed onto the compacted bedding such that the filled end was adjacent to the glass (Figure 5.21). This was achieved by initially compressing the foam rubber at the rear of the pipe against the steel wall opposite the 1.5m long glass face. Some local disruption to the sand bedding layer did however occur during this operation adjacent to the pipe. Consequently after placing the pipe, the sand layer was again levelled and compacted from outside the test tank (Figure 5.22). A set of photographs was then taken to define the state of the pipe and the bedding.

5.6.2 Placement of Sidefill and Backfill

The aim of the first four buried pipe tests was to simulate both good and poor site practice. Good site practice was defined as placing the soil surround carefully adjacent to the pipe to ensure good contact. This installation condition is described as Well Placed (WP) and was achieved as follows:

Step 1 - Placement of the pipe on a compacted sand bedding.
Step 2 - Sand hand-fed remotely onto the sand bedding beneath the pipe haunches. This material was placed to form an approximate bedding angle of 90°, guided by two 45° inclined lines drawn, after the pipe had been placed, on the outside face of the glass from the centre of the pipe.

Step 3 - Sand placed uniformly to the level of the pipe springline, using the dispersal unit.

Step 4 - Sand placed uniformly to the level of the pipe crown, using the dispersal unit.

Step 5 - Sand placed uniformly to the top of the test rig, using the dispersal unit.

Poor site practice was simulated by not providing support beneath the pipe haunches. This resulted in a small bedding angle, typically 16°, and was achieved by following only Steps 1, 4 and 5 outlined above (i.e. omitting Steps 2 and 3). During Step 4, the sand was placed either uniformly to either side of the pipe to create a Poorly Placed (PP) sand surround, or in an Asymmetric manner (PPA). In this latter case, the sand was allowed to fall to the right-hand side (RHS) of the pipe first, before filling at the other side (hence the asymmetry), rather than bringing up the levels on both sides simultaneously.

It should be noted that the density state of the sidefill and backfill material was the same throughout a particular test. A set of photographs was taken on completion of each stage in the filling operation. For safety reasons, the filling operation was restricted to week days within the day-time hours of the University. This meant that the filling operation was often performed over a number of days. When this occurred, the filling operation was halted at a convenient stage to allow a set of photographs to be taken and the cameras to be removed. To observe the overnight behaviour of the buried pipe system, at the start of the next day the cameras were returned to their original positions and a similar set of photographs was taken before the filling operation re-commenced.

5.6.3 Uniform Surface Stress

Once the tank was filled, a uniform surface stress was applied in all tests using the water bag arrangement. The static surface stress was applied according to the schedule
given in Table 5.12, the stress remaining constant at each static increment for thirty minutes to allow for thirty measurements of each strain gauge. Over this period relaxation of the pipe will occur. Random sampling of strain gauge measurements during the application of static stress (see Section 5.3.2.3) indicated that there was no significant change in pipe wall strain over this period, as the variation between measurements was of the same magnitude as the variation of the measuring technique itself (Table 5.7). A set of photographs was taken after the surface stress had been applied for fifteen minutes in each case.

After the 70kPa static surface stress had been applied for 30 minutes, the surface stress was reduced to zero to allow the unload behaviour of the pipe and the surrounding soil to be recorded over a period of 30 minutes. A Dartec signal generator was then used to apply a 70kPa cyclic surface stress with a period of 100 seconds. Throughout the application of cyclic surface stress using the signal generator the behaviour of the pipe was recorded by strain gauge measurements. The cycle time was however, too short to allow a set of photographs to be taken to record the load - unload behaviour of the pipe and the surround soil. A compromise situation was developed whereby the cycles of stress not recorded photographically were applied using the signal generator. For the stages during the cyclic regime where the load-unload behaviour of the buried pipe and the surrounding soil was recorded photographically, the 70kPa static surface stress was applied manually for a duration of typically between 15 and 30 minutes. This duration permitted a minimum of thirty measurements of each strain gauge and a set of photographs to be recorded. The surface stress was thereafter reduced to zero to allow the unload behaviour of the pipe and the surrounding soil to be recorded. The surface stress was removed for a period of between 15 and 30 minutes to allow thirty measurements of each strain gauge and a set of photographs to be recorded. This procedure was adopted to record the load - unload behaviour of the pipe and the surrounding soil after the application of 100, 500 and 1000 cycles.

Once the maximum surface stress of 150kPa had been applied for 30 minutes, the surface stress was reduced to zero to allow the unload behaviour of the pipe and the surrounding soil to be recorded over a period of 30 minutes.
5.7 TUNNEL TEST PROCEDURE

The sand was placed into the test tank in 250mm thick layers and compacted to achieve a dense state up to the bedding level of the outer tube to be used. The elevation of the bedding layer was adjusted so that the outer tube would be positioned centrally within the 275mm diameter hole in the back of the tank. The annular rubber seal was then loosely attached to the test tank and the elevation of the 'Y' shaped external support frame was adjusted according to the diameter of the outer tube used. The outer tube was placed onto the 'Y' support, passed through the annular seal and placed onto the horizontal compacted sand bedding adjacent to the glass. The outer tube was rotated so that a line of the supporting brass buttons was at the invert of the outer tube. Using the two rails attached to the invert of the inner tube as a guide, the inner tube was positioned inside the outer tube and also placed adjacent to the glass. The inner tube restraint and outer tube withdrawal systems were then assembled.

Before any displacement of the outer tube had occurred, a set of photographs and thirty measurements of each of the strain gauges were taken. The outer tube was withdrawn incrementally. On completion of each stage of withdrawal, the connecting plate was restrained and the protruding length of the outer tube was measured. A set of photographs and thirty measurements of each of the strain gauges were then taken. The withdrawal increments were increased progressively throughout the withdrawal, with approximate initial increments of 4mm and later maximum increments of 30mm.

5.8 PROGRAMME FOR BURIED PIPE TESTS

An extensive programme of preliminary tests was conducted before the main programme of buried pipe tests was conducted to refine the experimental and digitising procedures. Analysis of the photographs taken during these tests highlighted the likely lateral zone of influence of the buried pipe on the surrounding soil, thus defining the requirement that the test specimen be placed centrally within the 400mm wide subject area.
Details of the buried pipe tests and the test specimens are given in Tables 5.9 and 5.1 respectively. The stages during the buried pipe tests are described in Tables 5.11 and 5.12, and are denoted by a 'X'. Table 5.11 details the installation of the pipe (Step 1) and placement of the soil surround (Steps 2 to 5), whilst Table 5.12 details the application of uniform surface stress (Steps 6 to 18). It should be noted that during the test programme the placement of the soil surround and the application of surface stress was varied, thus steps between 2 and 18 may have been omitted. The first four experiments of the main test programme (Tests 250PPL!, 250WPL!, 250WPDI and 250PPLA1) aimed to investigate the influence of the installation procedure on pipe-soil interaction and to maximise the resulting pipe and soil displacements (see Rogers et al., 1996). This resulted in the use of a 250 mm diameter PVC-U pipe with a SDR of 93 (compared to typical SDR values of 41), at a minimal C/D ratio of unity.

The results of Test 250WPDI showed that the pipe was raised from the bedding during the compaction of the sidefill. The subsequent occurrence of a prominent void beneath the pipe invert meant that these results could not be compared to those of Test 250WPL! to investigate the influence of the density of the surround. The aim of Test 250WPDI\(^{1}\) was to repeat Test 250WPDI whilst ensuring that the pipe invert remained in contact with the bedding during the placement of the dense state sidefill. This aim was achieved by placing a 4kg, 1.0m long steel bar at the invert of the pipe and placing the sidefill and backfill in 50mm thick layers, compacted with a 1.5kg wood tamper. The results of Test 250WPDI\(^{1}\) showed, however, that a very large increase in the vertical diameter of the pipe was recorded, which was deemed not to be an easily reproducible factor. For this reason, the remaining tests in the programme were conducted using a well placed loose surround with the aim of investigating the influences of pipe stiffness, C/D ratio and cyclic loading.

Two tests were conducted using the vitrified clay pipe at C/D ratios of 1 and 2.5, Tests 120WPL! and 120WPL2.5 respectively, the higher value based on the observations of Terzaghi (1936). The influence of C/D ratio was also assessed during Test 250WPL2, which repeated Test 250WPL1 at an increased C/D ratio of 2. Repeatability of the complete test procedures were assessed in Test 250WPL1*, a repeat of Test
250WPL1, which also allowed an investigation into the effects of friction at the glass interface (see Section 6.5). The final two tests (168WPL2.5 and 168WPL3.5) were performed with a 168mm diameter PVC-U pipe which had an intermediate pipe stiffness between the 250mm diameter PVC-U pipe and the 120mm diameter vitrified clay pipe. Test 168WPL2.5 was a repeat of Test 120WPL2.5, as both pipes were placed to a C/D ratio of 2.5 and incorporated cyclic loading. To assess further the influence of the C/D ratio, Test 168WPL3.5 was undertaken to repeat Test 168WPL2.5 but at an increased C/D ratio of 3.5.

5.9 PROGRAMME FOR TUNNEL TESTS

Only a short series of preliminary tests (to prove the experimental techniques and the validity of the crane withdrawal system) and two full tunnel tests were carried out. The two full tests are described in Tables 5.10 and 5.13. Throughout both of these two tests the restraint force to the inner tube was maintained. This ensured that the displacement of the sand and the inner tube were measured throughout the tests, whilst experience was gained using the equipment.
Figure 5.1   General arrangement of Laboratory Test Tank and typical arrangement of camera booms used to record sand and buried structure displacements
Figure 5.2  Top view of Test Lid

Figure 5.3  Underside view of Test Lid showing Water Bag arrangement
Figure 5.4  Wooden supports placed beneath and at the four corners of the Test Tank lid to prevent rupture of Water Bag arrangement
Figure 5.5  Buried Pipes used during the laboratory tests

Figure 5.6  Tunnel Tubes used during the laboratory tests
Figure 5.7  Longitudinal section through Test Tank and Assembled Tail Void Tunnel Equipment (Dimensions in mm)
Figure 5.8 General arrangement of equipment at the rear of Test Tank used to Restrain the Inner Tube and Withdraw the Outer Tube, showing the 'Mask' plate, Inner Tube Restraint bar and 'Y' shaped Outer Tube supporting frame.
Figure 5.9  General arrangement of Connecting plate for steel cables and Stand

Figure 5.10  General arrangement of Pulley wheel
Figure 5.11 General arrangement of 'Mask' plate used to connect the Outer Tube to the two steel cables, through which the Inner Tube Restraint bar passes, connected to a circular Restraint plate.
Figure 5.12  General arrangement of equipment used to during filling of the Test Tank, showing the Floveyor auger
Figure 5.13  General arrangement of sand disperser used to obtain uniform placement of sand within Test Tank
Figure 5.14  Typical photographs taken during the placement of the Sidefill material and subsequent application of a Surface Stress during a Buried Pipe Test
Notes: — Circumferential strain gauges • Longitudinal strain gauges

Figure 5.15 Location of strain gauges on Test specimens
Figure 5.16 - Internal Circumferential Pipe Wall Strain (Front section) for 250mm diameter PVC-U pipe in Parallel Plate Test
Figure 5.17 - Internal Circumferential Pipe Wall Strain (Central Section) for 250mm diameter PVC-U pipe in Parallel Plate Test
Figure 5.18 - Internal Circumferential Pipe Wall Strain for 168mm diameter PVC-U pipe in Parallel Plate Test
Figure 5.19 - Comparison of PVC-U tensile tests, Clean Sample (No 2), Glue-Only Sample (No 1) and Strain Gauge Sample (No 3)
Figure 5.20   Particle size distribution for the Leighton Buzzard sand
Figure 5.21  Placement of the Buried Pipe Specimen adjacent to Glass face of Test Tank

Figure 5.22  Final Installation of Buried Pipe Test Specimen within Test Tank (denoted as Step 1)
Table 5.1  Details of dimensions, mechanical and thermal properties of the test specimens used in the model buried pipe and tunnel tests.

<table>
<thead>
<tr>
<th>Test Specimen Material</th>
<th>PVC-U</th>
<th>PVC-U</th>
<th>Vitrified Clay</th>
<th>Mild Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Diameter (mm)</td>
<td>250</td>
<td>168</td>
<td>120</td>
<td>194</td>
</tr>
<tr>
<td>Wall thickness (mm)</td>
<td>2.7</td>
<td>3.5</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>SDR ratio</td>
<td>93</td>
<td>48</td>
<td>12</td>
<td>39</td>
</tr>
<tr>
<td>Radius/Wall thickness</td>
<td>46</td>
<td>24</td>
<td>6</td>
<td>19</td>
</tr>
<tr>
<td>ratio</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic Modulus (GPa)</td>
<td>2.8</td>
<td>2.8</td>
<td>70</td>
<td>190- 210</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.38</td>
<td>0.38</td>
<td>0.30</td>
<td>0.27 - 0.30</td>
</tr>
<tr>
<td>Coefficient of thermal</td>
<td>60 - 70</td>
<td>60 - 70</td>
<td>7.3</td>
<td>11.4</td>
</tr>
<tr>
<td>expansion (1x10⁻⁶)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Sources**

The mechanical and thermal properties of the unplasterised polyvinylchloride (PVC-U) and vitrified clay pipes were obtained directly from the relevant manufacturer. Additional reference for the values stated above for the PVC-U and mild steel pipes were obtained from Brandrup and Immergut (1988) and Gere and Timoshenko (1991).
### Table 5.2  Precision of pipe displacements and diametral strains

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Number of measured points</th>
<th>Standard Error of pipe displacements ±(mm)</th>
<th>Precision of pipe displacements 95%±(mm)</th>
<th>Precision of VDS &amp; HDS 95%±(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250PPLA1</td>
<td>4</td>
<td>0.092</td>
<td>0.180</td>
<td>0.102</td>
</tr>
<tr>
<td>250WPDI</td>
<td>4</td>
<td>0.092</td>
<td>0.180</td>
<td>0.102</td>
</tr>
<tr>
<td>250WPL1</td>
<td>4</td>
<td>0.092</td>
<td>0.180</td>
<td>0.102</td>
</tr>
<tr>
<td>250PPL1</td>
<td>4</td>
<td>0.092</td>
<td>0.180</td>
<td>0.102</td>
</tr>
<tr>
<td>120WPL1</td>
<td>Total = 91</td>
<td>0.049</td>
<td>0.096</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14 used</td>
<td></td>
<td></td>
</tr>
<tr>
<td>250WPDL1</td>
<td>6</td>
<td>0.075</td>
<td>0.147</td>
<td>0.083</td>
</tr>
<tr>
<td>250WPL2</td>
<td>6</td>
<td>0.075</td>
<td>0.147</td>
<td>0.083</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>6</td>
<td>0.075</td>
<td>0.147</td>
<td>0.083</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>6</td>
<td>0.075</td>
<td>0.147</td>
<td>0.124</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>6</td>
<td>0.075</td>
<td>0.147</td>
<td>0.124</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>Total = 147</td>
<td>0.019</td>
<td>0.037</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>96 used</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:

- For details of Test Codes see Table 5.9.
### Table 5.3 Arrangements of strain gauges for buried pipe and tunnel tests

<table>
<thead>
<tr>
<th>Arrangement of Strain Gauges</th>
<th>Test Code</th>
<th>Number of thrust measurements at springline (90° and 270°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>250PPLA1</td>
<td>None</td>
</tr>
<tr>
<td>B</td>
<td>250WPD1</td>
<td>90° at Front Section</td>
</tr>
<tr>
<td></td>
<td>250WPL1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>250PPL1</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>120WPL1</td>
<td>90° and 270° at Front Section</td>
</tr>
<tr>
<td></td>
<td>250WPD1^</td>
<td></td>
</tr>
<tr>
<td></td>
<td>250WPL2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>168WPL2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>168WPL3.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>120WPL2.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>T1 and T2</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>250WPL1*</td>
<td>90° and 270° at Front</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90° and 270° at Mid-Length</td>
</tr>
</tbody>
</table>
Table 5.4  Measured and predicted strain measurements for the 250 and 168 mm diameter PVC-U pipes at 10 and 5 mm vertical deflection respectively.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Gauge Location</th>
<th>Springline Strain (με)</th>
<th>Circumferential Strain (με)</th>
<th>Elastic Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>External Wall</td>
<td>Internal Wall</td>
<td></td>
</tr>
<tr>
<td>250mm dia., 995mm long</td>
<td>Front</td>
<td>1021</td>
<td>-968</td>
<td>3.04</td>
</tr>
<tr>
<td></td>
<td>Mid-length</td>
<td>966</td>
<td>-995</td>
<td>3.08</td>
</tr>
<tr>
<td>168mm dia., 995mm long</td>
<td>Front</td>
<td>1212</td>
<td>-1341</td>
<td>3.50</td>
</tr>
<tr>
<td></td>
<td>Mid-length (Glue 1)</td>
<td>983</td>
<td>-1043</td>
<td>2.98</td>
</tr>
<tr>
<td>250mm dia., 200mm long</td>
<td>Mid-length (Glue 2)</td>
<td>975</td>
<td>-998</td>
<td>3.06</td>
</tr>
<tr>
<td>Predicted - 250mm dia. pipe</td>
<td></td>
<td>1066</td>
<td>-1087</td>
<td>2.8</td>
</tr>
<tr>
<td>Predicted - 168mm dia. pipe</td>
<td></td>
<td>1561</td>
<td>-1623</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Notes:
- Predicted values of strain based upon measured vertical deflection, an elastic modulus of PVC-U of 2.8GPa (see Table 5.1 and Sections 5.3.2.1 and 5.3.2.2) and the equations given in Appendix A.3.
- The back-calculated values of elastic modulus calculated from the measured strains at the springline are based upon the predicted values of stress.
Table 5.5 Measured and predicted strain measurements for the 120 and 180mm diameter vitrified clay pipes, and the 195mm diameter mild steel pipe.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Gauge Location</th>
<th>Total Load (kN/m)</th>
<th>Springline Strain</th>
<th>Circumferential Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>120mm dia, Clay</td>
<td>Front</td>
<td>63</td>
<td>-65</td>
<td></td>
</tr>
<tr>
<td>180mm dia, Clay</td>
<td>Mid-length (Glue 3)</td>
<td>133</td>
<td>-167</td>
<td></td>
</tr>
<tr>
<td>195mm dia, Steel</td>
<td>Front</td>
<td>165</td>
<td>-179</td>
<td></td>
</tr>
<tr>
<td>Predicted - 120mm dia. Clay</td>
<td>59</td>
<td>-70</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Predicted - 180mm dia. Clay</td>
<td>132</td>
<td>-154</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Predicted - 195mm dia. Steel</td>
<td>170</td>
<td>-178</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- Predicted values of strain based upon measured applied load, elastic modulus for vitrified clay and mild steel of 70 and 200 GPa (see Table 5.1) respectively and the equations given in Appendix A.3.
### Table 5.6 Elastic modulus (GPa) for PVC-U specimens during tensile tests

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>CLEAN SAMPLE Extensometer Reading</th>
<th>GLUE SAMPLE Extensometer Reading</th>
<th>STRAIN GAUGE SAMPLE Extensometer Reading</th>
<th>Strain Gauge Reading</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.0</td>
<td>3.6</td>
<td>Faulty specimen</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2.7</td>
<td>3.4</td>
<td>3.5</td>
<td>3.9</td>
</tr>
<tr>
<td>3</td>
<td>3.1</td>
<td>3.0</td>
<td>3.5</td>
<td>3.8</td>
</tr>
<tr>
<td>4</td>
<td>2.8</td>
<td>3.4</td>
<td>2.9</td>
<td>3.5</td>
</tr>
<tr>
<td>Average</td>
<td>2.9</td>
<td>3.4</td>
<td>3.3</td>
<td>3.7</td>
</tr>
</tbody>
</table>

### Table 5.7 Precision of strain gauge measurements for model buried pipe tests

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Standard deviation (με)</th>
<th>Confidence interval 95% ±(με)</th>
<th>Difference in means, 95% ±(με)</th>
<th>Hoop thrust ±kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>250PPLA1</td>
<td>28</td>
<td>10</td>
<td>14</td>
<td>N/A</td>
</tr>
<tr>
<td>250WPD1</td>
<td>140</td>
<td>50</td>
<td>71</td>
<td>0.4</td>
</tr>
<tr>
<td>250WPL1</td>
<td>140</td>
<td>50</td>
<td>71</td>
<td>0.4</td>
</tr>
<tr>
<td>250PPL1</td>
<td>140</td>
<td>50</td>
<td>71</td>
<td>0.4</td>
</tr>
<tr>
<td>120WPL1</td>
<td>140</td>
<td>50</td>
<td>71</td>
<td>35</td>
</tr>
<tr>
<td>250WPD1^</td>
<td>28</td>
<td>10</td>
<td>14</td>
<td>0.08</td>
</tr>
<tr>
<td>250WPL2</td>
<td>14</td>
<td>5</td>
<td>7</td>
<td>0.04</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>14</td>
<td>5</td>
<td>7</td>
<td>0.04</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>14</td>
<td>5</td>
<td>7</td>
<td>0.05</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>14</td>
<td>5</td>
<td>7</td>
<td>0.05</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>3</td>
<td>1</td>
<td>2</td>
<td>0.7</td>
</tr>
</tbody>
</table>

**Notes:**

- For details of Test Codes see Table 5.9.
Table 5.8  Details of soil parameters for Leighton Buzzard sand

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Leighton Buzzard Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content</td>
<td>0.14%</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.63</td>
</tr>
<tr>
<td>Maximum Density</td>
<td>1801 kg/m³</td>
</tr>
<tr>
<td>Minimum Density</td>
<td>1493 kg/m³</td>
</tr>
<tr>
<td>Maximum void ratio</td>
<td>0.79</td>
</tr>
<tr>
<td>Minimum void ratio</td>
<td>0.49</td>
</tr>
<tr>
<td>Coefficient of uniformity</td>
<td>1.36</td>
</tr>
<tr>
<td>Coefficient of curvature</td>
<td>1.04</td>
</tr>
<tr>
<td>Angle of shear resistance ($\phi'$)</td>
<td></td>
</tr>
<tr>
<td>- Dense state</td>
<td>46°</td>
</tr>
<tr>
<td>- Loose State</td>
<td>36°</td>
</tr>
<tr>
<td>- Critical state</td>
<td>33°</td>
</tr>
<tr>
<td>Dilation angle ($\phi$)</td>
<td></td>
</tr>
<tr>
<td>- Dense state</td>
<td>14°</td>
</tr>
<tr>
<td>- Loose state</td>
<td>4°</td>
</tr>
</tbody>
</table>
General notes for Tables 5.9 to 5.13

D  Dense state sand (in Test Code)
L  Loose state sand (in Test Code)
C  Cover depth
D  Outside Diameter of pipe
R  Radius of pipe
t  Overcut distance
S.D.R.  Standard Dimension Ratio
X  Step or stage of test performed

Installation Condition of Surround:
WP  Well-Placed
PP  Poorly-Placed
PPA  Poorly-Placed, Asymmetrically placed sidefill

^ indicates that the experiment was a repeat of Test Code 250WPD1 but with a
4 kg steel bar placed at the pipe invert to maintain the elevation of the pipe
invert during sidefill compaction.

* indicates that the experiment was a repeat of Test Code 250WPL1 to
compare the repeatability of the test procedure and the influence of the glass.

Buried pipe test code is defined as:
Pipe diameter (D), Installation of Sidefill and Backfill, density state of sidefill
and backfill, cover depth to pipe diameter ratio (C/D)

All the experiments were conducted using Leighton Buzzard Sand.
The bedding layers for each experiment were placed in 250mm layers to a
dense state.
Table 5.9  Details of the Buried Pipe test series

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Pipe Material</th>
<th>Pipe Diameter</th>
<th>Installation of Surround &amp; Bedding</th>
<th>Surround/Bedding Density (D or L)</th>
<th>C/D Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>250PPLA1</td>
<td>PVC-U</td>
<td>250</td>
<td>PPA</td>
<td>L</td>
<td>1</td>
</tr>
<tr>
<td>250WPD1</td>
<td>PVC-U</td>
<td>250</td>
<td>WP</td>
<td>D</td>
<td>1</td>
</tr>
<tr>
<td>250WPL1</td>
<td>PVC-U</td>
<td>250</td>
<td>WP</td>
<td>L</td>
<td>1</td>
</tr>
<tr>
<td>250PPL1</td>
<td>PVC-U</td>
<td>250</td>
<td>PP</td>
<td>L</td>
<td>1</td>
</tr>
<tr>
<td>120WPL1</td>
<td>CLAY</td>
<td>120</td>
<td>WP</td>
<td>L</td>
<td>1</td>
</tr>
<tr>
<td>250WPD1^</td>
<td>PVC-U</td>
<td>250</td>
<td>WP</td>
<td>D</td>
<td>1</td>
</tr>
<tr>
<td>250WPL2</td>
<td>PVC-U</td>
<td>250</td>
<td>WP</td>
<td>L</td>
<td>2</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>PVC-U</td>
<td>250</td>
<td>WP</td>
<td>L</td>
<td>1</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>PVC-U</td>
<td>168</td>
<td>WP</td>
<td>L</td>
<td>2.5</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>PVC-U</td>
<td>168</td>
<td>WP</td>
<td>L</td>
<td>3.5</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>CLAY</td>
<td>120</td>
<td>WP</td>
<td>L</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 5.10  Details of the full Tunnel test series

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Density State (D or L)</th>
<th>C/D Ratio</th>
<th>t/R Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>D</td>
<td>4.5</td>
<td>0.26</td>
</tr>
<tr>
<td>T2</td>
<td>D</td>
<td>2.5</td>
<td>0.26</td>
</tr>
</tbody>
</table>
Table 5.11 Details of pipe installation and placement of soil surround

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Step 1</th>
<th>Step 2</th>
<th>Step 3</th>
<th>Step 4</th>
<th>Step 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>250PPLA1</td>
<td>X</td>
<td>N/A</td>
<td>N/A</td>
<td>X-1-250</td>
<td>X-1-250</td>
</tr>
<tr>
<td>250WPD1</td>
<td>X</td>
<td>X</td>
<td>X-1-125</td>
<td>X-1-125</td>
<td>X-1-250</td>
</tr>
<tr>
<td>250WPL1</td>
<td>X</td>
<td>X</td>
<td>X-1-125</td>
<td>X-1-125</td>
<td>X-1-250</td>
</tr>
<tr>
<td>250PPL1</td>
<td>X</td>
<td>N/A</td>
<td>N/A</td>
<td>X-1-250</td>
<td>X-1-250</td>
</tr>
<tr>
<td>120WPL1</td>
<td>X</td>
<td>X</td>
<td>X-1-60</td>
<td>X-1-60</td>
<td>X-1-120</td>
</tr>
<tr>
<td>250WPD1^</td>
<td>X</td>
<td>X</td>
<td>X-5-50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>250WPL2</td>
<td>X</td>
<td>X</td>
<td>X-1-125</td>
<td>X-1-125</td>
<td>X-2-250</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>X</td>
<td>X</td>
<td>X-1-125</td>
<td>X-1-125</td>
<td>X-2-125</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>X</td>
<td>X</td>
<td>X-1-84</td>
<td>X-1-84</td>
<td>X-5-84</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>X</td>
<td>X</td>
<td>X-1-84</td>
<td>X-1-84</td>
<td>X-7-84</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>X</td>
<td>X</td>
<td>X-1-60</td>
<td>X-1-60</td>
<td>X-5-60</td>
</tr>
</tbody>
</table>

Additional notes for Table 5.11

Basic description of stages of the pipe installation and placement of soil surround is:

- Step 1: Pipe placed on compacted, horizontal bedding
- Step 2: Sand hand fed beneath pipe haunches
- Step 3: Sand placed uniformly to pipe springline
- Step 4: Sand placed uniformly to pipe crown
- Step 5: Sand placed to the top of the test tank

Notation used for the placement of the soil surround, Steps 2 to 5 is:

- Step was performed, number of sand layers, thickness of sand layer
Table 5.12  Details of uniform surface stress regime during buried pipe tests

<table>
<thead>
<tr>
<th>Step Number</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
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</thead>
<tbody>
<tr>
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<tr>
<td></td>
<td>50s</td>
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<td>70c</td>
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<td>Load</td>
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<td>Load</td>
<td>Load</td>
<td>Load</td>
<td>Load</td>
<td>Load</td>
<td>Load</td>
<td></td>
</tr>
<tr>
<td>250PPLA1</td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>250WPD1</td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>250WPL1</td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
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<td></td>
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<td>250PPL1</td>
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<td>120WPL1</td>
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<td></td>
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<tr>
<td>250WPD1^</td>
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<td>X</td>
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<td>X</td>
<td></td>
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<td></td>
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<td>X</td>
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<td></td>
</tr>
<tr>
<td>250WPL2</td>
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<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>250WPL1*</td>
<td>X</td>
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</tbody>
</table>
Additional notes for Table 5.12

Notation used for the uniform surface surcharge loading is defined as:

Magnitude of surcharge (kPa), Nature of loading (s or c) and for cyclic loading only, the number of cycles

- s  static loading. Magnitude of loading constant for 30 minutes.
- c  sinusoidally applied cyclic loading.

Magnitude of loading represents the peak value of applied stress of 70kPa and a minimum applied stress of 0 kPa

Table 5.13  Details of the distance of withdrawal of 245 mm diameter outer tube

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Distance of withdrawal of outer tube / mm</th>
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<tbody>
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<td>0</td>
</tr>
<tr>
<td>T1 (C/D=4.5)</td>
<td>X</td>
</tr>
<tr>
<td>T2 (C/D=2.5)</td>
<td>X</td>
</tr>
</tbody>
</table>


6. EXPERIMENTAL RESULTS

The aim of Chapters 6 and 7 is to present and then discuss respectively the results of the buried pipe and tunnel tests. At stages during these tests measurements were taken from strain gauges attached to the buried structures and displacements of the buried structure and surrounding soil were recorded. The use of the same measuring techniques during both types of tests has led to the adoption of similar methods of presenting the results. The results from the buried pipe and tunnel tests are presented in Sections 6.1 and 6.2 respectively.

6.1 BURIED PIPE TEST RESULTS

This Section aims to present the results from all the buried pipe tests by initially presenting the strain gauge measurements, followed by the displacement measurements. The strain gauge measurements are described in Section 6.1.1. Measurements of strains in the longitudinal plane are examined with respect to those in the circumferential direction (see Section 6.1.2) to assess whether plane stress or plane strain conditions apply in the calculation of imposed load at the pipe springline (Section 6.1.4). An investigation was undertaken to assess the influence of wall friction between the sides of the test tank and the Leighton Buzzard sand by conducting a repeat of the buried pipe test 250WPL1, which is referred to as Test 250WPL1*. The results from this test are described in Section 6.1.3 and are used to determine the magnitude of applied stress to the buried pipe, thus the degree of loading (Section 6.1.4).

The total displacement field around a buried pipe is presented in the form of both vector and contour plots (Section 6.1.5). The contour plots are presented to show the magnitude and distribution of horizontal and vertical displacements. The vertical displacement contour plots have additionally been used to produce settlement profiles (Section 6.1.6). From the displacement data, values of pipe diametral strain and soil strain distribution plots have been calculated and are described in Sections 6.1.7 and 6.1.8 respectively.
6.1.1 Circumferential pipe wall strain

The measured pipe wall strains are presented for each test in Appendix 4, using Step 1 as the datum reading and tensile strains as positive. To determine the response of the buried pipe under external loading, measurements of strain are required prior to and during the application of external loading. The shape of the deformed pipe, thus indicating how the pipe derived its structural support, was defined by for example Rogers (1985) by the use of internal circumferential pipe wall strain measurements. (An aid to interpretation is given by Table 6.4, which describes the shape due to the static application of 150kPa at the surface, although in some cases this followed a series of cyclically applied surface stresses). As Section 7.1 describes in greater detail, the strain gauge measurements from the two vitrified clay pipe tests (Tests 120WPL1 and 120WPL2.5) proved to be unreliable. For these reasons, the circumferential strain measurements have been shown for each buried PVC-U pipe test (only) on completion of the backfill (Step 5) and during the application of a uniform static surface stress of 150kPa (Step 17) respectively, using Step 1 as the datum reading, in Tables 6.1 and 6.2. The internal pipe wall measurements are presented, except for Test 250PPLA1 where only external pipe wall measurements were recorded.

During the buried pipe test programme, the pipes were installed according to Well Placed (WP) and Poorly Placed (PP) conditions. The main distinction between these two types of installation conditions was the nature of the placement of sand up to the level of the pipe crown. After placing the pipe on the bedding (Step 1), during the WP tests sand was hand fed beneath the pipe haunch locations (Step 2) and placed evenly using the sand disperser on either side of the pipe to the level of the pipe springline and crown (Steps 3 and 4 respectively). This contrasts to the direct placement of the sidefill material using the disperser to the level of the pipe crown (Step 4) in the PP tests, thereby omitting Steps 2 and 3, which results in a small bedding angle, typically 16°, and the formation of large voids beneath both pipe haunch positions.

Typical examples of internal pipe wall circumferential strain distributions were recorded during Tests 250WPD1 and 250WPL1, which were installed according to WP conditions with dense and loose sidefill and backfill material respectively, and
250PPL1, which was installed with a loose sidefill and backfill in a PP surround (Figure 6.1). The figure highlights the difference in the response of the same pipe placed in different soil surrounds during the application of a maximum surface stress of 150kPa (Step 17) using the completion of the backfill material (Step 5) as the datum reading. The response recorded during Test 250WPDI is characterised by small (typically less than 500με) compressive strains around the complete perimeter of the pipe. Placing the WP surround in a loose rather than dense state (Test 250WPL1) caused increased compressive strains around the springlines but otherwise a similar strain profile indicating deformation predominantly according to uniform hoop compression, which was typical of all of the pipes in a well placed surround. The distortional, or ring bending component, of pipe deformation is shown to be less during Test 250WPDL compared to Test 250WPL as the lateral displacement of the pipe is resisted by the stiffer surround.

To assess the inferred shape of the pipe deformation, the circumferential strain distribution was compared with the five types of distribution described by Rogers et al (1996), which are reproduced in Figure 6.2. The recording of a symmetrical strain response about the vertical axis during Test 250WPDL, with greater compressive strains at the springline locations (90° and 270°) compared to those at the crown and invert positions (0° and 180° respectively), indicates the occurrence of a very stable pipe (little deformation) undergoing marginal elliptical deformation. By comparison, Test 250WPL shows a broadly similar pattern with greater deformation, as would be expected with a looser material. However, on the LHS of the pipe (locations 180° to 360°) the greatest compressive strain was recorded at the pipe haunch location (225°), indicating the pipe deformed to an inverted 'heart' shape pattern and consequently that the placement of sand beneath the haunch at this side was relatively ineffective. (The pipe deforms preferentially into the haunch material and flattens at the invert due to the good bedding support at this point according to the inverted 'heart' shaped deformation profile).

Figure 6.1 shows that installing the same 250mm diameter PVC-U pipe in a poorly placed condition with loosely placed sidefill and backfill material (Test 250PPL1)
resulted in a tensile strain of 2369με at the pipe invert. This magnitude of strain was based upon the recording of the largest absolute value, with respect to Step 1, during the buried pipe test programme of 2820με. Figure 6.1 also shows that the measurements taken at six of the seven remaining strain gauge locations (i.e. not 135°) were all comparable to those recorded during Test 250WPL1. This shows that the lack of material fed beneath the haunch caused considerable flattening at the invert, and that whilst the sidefill material was placed symmetrically the behaviour was notably asymmetrical with poorer support at 135° than 225°.

Measured internal pipe wall circumferential strains are also shown for the application of 70kPa uniform cyclic surface stress during the 1st, 100th and 500th cycles (Steps 7, 9 and 11 respectively) for both the buried PVC-U pipe tests (168WPL2.5 and 168WPL3.5) in which this type of loading was conducted (Table 6.3). These measurements have been shown for Test 168WPL3.5, using Step 5 as the datum readings, in Figure 6.3. The results show that an elliptical strain response occurred at each of the three cycles recorded, and that the increase in magnitude of strain is not linear with the number of cycles applied. This finding concurred with that of Rogers (1985), who found that static loading tended to cause a deformed shape that was wholly dictated by the surround, whereas cyclic loading tended to cause an elliptical response.

6.1.2 Axial strain response of buried pipes

Due to the large Poisson's ratio of PVC-U, quoted in Table 5.5 as 0.3, it was necessary to assess whether the response of the buried pipes to external loading approximated to plane strain or plane stress conditions. The implication of this distinction is shown by equation 5.8, as the imposed load for plane strain conditions is 10% greater than that for plane stress. To assess this, the relative axial and circumferential strain values have been compared in Table 6.4. The values shown were recorded under the application of the largest surface of 150kPa (Step 17), as the largest strains were recorded at this stage of each test. To aid the interpretation of plane strain or plane stress conditions a description of the pipe response is also included.
During the parallel plate tests, the pipe was not surrounded by soil. The pipe deformed to an ellipse and negligible uniform hoop compression was recorded. The maximum axial strains were low, less than 48με thus corresponding to 3% of the hoop strain, and were consistent between the front and mid-length sections, indicating that the pipe was distorted in plane along its whole length.

Due to the use of a foam plug at the rear of the buried pipe specimens, different restraint conditions were established at either end of the pipe once it was placed into the test tank. The low stiffness of the foam plug allowed the pipe to expand or contract axially, contrasting with the rigid nature of the glass face at the other end. The results from Test 250WPL1* highlight the difference between these restraint conditions, as a smaller tensile axial strain was recorded by the front gauges (i.e. at the glass face) than at the central section. Table 6.4 shows that the external tensile axial strain (747με) was comparable to the magnitude of the compressive external hoop strain at the springline (-734με), thus indicating plane stress conditions. This conclusion is also supported by the internal pipe wall circumferential strain readings (Table 6.1) which show that whilst the pipe was surrounded by loosely placed sand, a uniform hoop compression similar in magnitude to the axial response was recorded (-1113με from the four gauges). Similar behaviour was recorded at the front section, except that the magnitudes of strain were reduced.

Table 6.4 shows that for the remaining nine tests in which both internal and external pipe wall strains were recorded (i.e. except Test 250PPLA1), the maximum axial strain response was low, typically less than 250με. This compares to the more variable response of the circumferential gauges, which were influenced by the nature of the soil surround. Table 6.4 shows that the WP tests typically deformed to an elliptical or heart shape, whilst the PP test deformed to an inverted heart shape. Surrounding the pipe with loosely placed sand reduced the difference between the measurements of axial and hoop strain, compared to those recorded during the parallel plate tests, as the uniform hoop component was higher. Surrounding the pipe with dense sand, or as shown by Test 250WPL2 subjecting the loose state sand to a higher confining stress by
increasing the initial cover depth to pipe diameter ratio, increased the uniform hoop compression further, as compressive, rather than tensile, external hoop strains were recorded whilst the pipe deformed to an ellipse. Except on completion of the installation of the pipe during both WPD tests, the magnitudes of hoop and axial strains were comparable for all the tests thus indicating plane stress conditions.

6.1.3 Wall friction
A major factor influencing the accuracy of the results obtained was the influence of the glass interface at which plane measurements were taken. This was assessed in part by conducting a repeat test (250WPL1*) in which strain gauges were attached at both the glass-pipe interface and mid-length sections.

To determine the angle of friction between glass and dry Leighton Buzzard sand, a series of tests was conducted using a 300 by 300 by 160mm deep shear box. A 67mm thick plywood packer was used to ensure that the top of the 300 by 300 by 13mm thick toughened glass was flush with the top of the bottom section of the shear box. Tests were conducted using loose and dense Leighton Buzzard sand at four levels of normal stress corresponding to the overburden stress used during the buried pipe tests. From the typical results shown in Figure 6.4, it can be seen that the steel surround of the shear box resulted in a constant rate of increase (determined using a linear regression analysis) of shear stress to occur after typically a shear strain of 3%, rather than a constant value of shear stress at failure. Whilst noting this approximation, angles of friction between the glass and dense and loose sand were measured as 31° and 27° respectively, see Figure 6.5.

Using these angles of friction (and assuming similar frictional losses against the remaining three smooth steel walls) the reduction in applied stress was determined using the shear plane method (equation 3.5). As this method relates to a two-dimensional situation, to calculate the frictional effects of all four walls separate analyses were conducted assuming both 1.5 and 1.0m wide trenches. The predicted applied stress at the centre of the test tank was determined by adding the shear stresses from both analyses and subtracting them from the 'prism load'. To determine the
applied stress at the glass interface, the previously calculated applied stress from the 1.5m wide trench analysis was used to determine the effect of the 1.0m wide tank sides. Assuming that $K_s = 1 - \sin \phi'$ (Jaky, 1948), where $\phi'$ is the effective angle of friction for Leighton Buzzard sand, this value of applied stress was used to calculate the shear stress due to the glass face.

The resulting predicted applied stress values at both instrumented sections are shown in Table 6.5 using the Krynine proposed value of $K$ (equation 2.3) and Marston's assumed value of $K = K_s$. Table 6.6 compares the predicted applied load at the glass-pipe interface and mid-length sections with those inferred from pipe wall strain measurements during Test 250WPL1*, using Step 3 as the datum readings and plane stress conditions. It shows that the degree of loading (see Section 6.6.4) at the central section was consistently greater than that at the glass interface by approximately 10% (the values varying between 6 and 17%).

To examine whether this difference was due to inaccuracies in the method of predicting the applied load or was due to a variation in soil support conditions, the internal and external circumferential pipe wall strains at both the front and central section are presented in Figure 6.6. This shows that the internal strains at both sections were similar in magnitude, whilst the external strains at the central section greatly exceeded those at the front section. These results show that greater hoop compression and less distortional effects were recorded at the central, compared to the front, section. The contrasting response indicates that the surround at the front section was placed to a lower stiffness than that at the central section and this was due to glass face restricting the motion of the rake during the levelling operation. In view of this, a difference in the degree of loading of typically 10% between the two sections would therefore seem to be reasonable.

6.1.4 Arching calculations

To determine the degree of arching induced during each test, the approach defined by either Getzler et al (1968), see equation 2.5, or Valsangkar and Britto (1977 and 1978), see equation 3.29, could have been adopted. The latter approach is simply the
stress ‘imposed’ on the structure, divided by the ‘applied’ stress. Subtracting this value from unity yields the degree of the arching, so converting equation 3.29 to equation 2.5. As the degree of load reaching the structure was of direct interest, equation 3.29 was modified for this study by dividing the imposed stress at each of the springline positions by half of the applied stress. A degree of loading of unity, 100%, illustrates that the imposed stress was equal to the applied stress, hence no arching occurred. A value less than 100% indicates that positive arching occurred resulting in a reduction of stress at the springline.

The long-term behaviour of a buried pipe system under an applied stress depends upon the relative relaxation of the buried pipe and the surrounding soil. During these tests, the increments of surface stress were applied for a maximum of thirty minutes, during which there was no variation in pipe wall strain, even when the filling operation was halted resulting in the application of a soil cover overnight. As Section 6.1.1 previously described, the maximum recorded strain during the parallel plate and buried pipe tests occurred during Test 250PPL1 where external and internal pipe wall circumferential strains of -3667 and 2820 με (-0.37 and 0.28%) respectively were recorded at the pipe invert. As these are below a conservative elastic limit of 0.5% for PVC-U (Figure 5.19 and BPF 1971), it was assumed that the PVC-U pipes remained within the linear elastic region throughout the test programme. Thus an elastic modulus of PVC-U of 2.8 GPa was used in equation 5.5 to convert the measured hoop strains to stresses. At each stage of the tests, therefore, the measured internal and external circumferential wall strains were used to determine the imposed load at the springline in accordance with plane stress conditions, using Step 1 as datum readings. These values of imposed load were used with the values of applied load (see Section 6.1.3) to determine the values of degree of loading. The measured imposed load, predicted applied load and degree of loading (using Step 4 as datum readings) are presented for each step of each test in Appendix 5. These values are presented for each buried PVC-U pipe test in Table 6.7 based on the application of a uniform static surface stress of 150 kPa (Step 17), using Step 5 as the datum reading. The results show that for each test, the degree of loading was less than 100% thus indicating the occurrence of positive arching. Based upon measurements taken during Test 250WPL1*, a typical example of the relationship
between imposed and applied load during the application of uniform static stress is presented in Figure 6.7. This figure shows a linear relationship of imposed against applied load, the maximum applied load corresponding to a uniform surface stress of 150kPa, or approximately 10m of loose sand (Step 17).

Measured imposed load, predicted applied load and calculated degree of loading are presented in Table 6.8 for the application of 70kPa uniform cyclic surface stress during the 1st, 100th and 500th cycles (Steps 7, 9 and 11 respectively) for both the buried PVC-U pipe tests (168WPL2.5 and 168WPL3.5) in which this type of loading was conducted. These measurements have been shown for Test 168WPL3.5, using Step 5 as the datum readings in Figure 6.8. The results show a linear relationship between the imposed and applied load during the application of static surface stress, thus corresponding to the relationship shown in Figure 6.7 for Test 250WPL1*. During the application of the cyclic loading (corresponding to an applied load of -3.571kN/m), however, the imposed load on the pipe reduced as the number of cycles increased.

6.1.5 Vector and contour plots of total displacement fields

To show the total displacement field of the buried pipe and the surrounding soil, both vector and contour plots have been used. Throughout this study a positive horizontal displacement is denoted as to the right, whilst a positive vertical displacement is denoted as upward. Vector plots graphically illustrate the movement of each individual sand grain and each location of the pipe wall. Figure 6.9 shows a typical example of a vector plot in which the magnitudes of the displacements have been exaggerated by a scale of factor of ten. This was recorded during Test 250PPL1 and represents the displacements caused by the application of a uniform surface stress of 150kPa (Step 17), using the locations of the points on completion of the backfill (Step 5) as datum readings. The figure clearly shows that the displacement of the pipe crown is greater than that of the adjacent sidefill material, illustrating the occurrence of positive arching. In addition, the absence of sand support beneath the pipe haunches (caused by the omission of Step 2) is shown by the larger displacements of the pipe at these locations compared to those recorded at the pipe invert and directly illustrates the inverted 'heart' shaped deformation inferred from the strain gauge readings (Figure 6.1).
To facilitate simpler interpretation of the magnitude and distribution of these
displacements, horizontal and vertical displacement contour plots have been shown in
Figures 6.10 and 6.11 respectively. Comparing these two figures it is clear that the
magnitudes of vertical displacements (Figure 6.11) greatly exceed those of the
horizontal displacements (Figure 6.10) and that all the vertical displacements are
significant (i.e. greater than 0.5mm in magnitude). By comparison, the regions of
significant horizontal displacements occur only adjacent to the pipe wall and that as
expected, above the pipe crown the magnitudes of horizontal displacements are taken
as zero. Figure 6.11 clearly shows the promotion of positive arching above the pipe
crown that was indicated by the vector plot (Figure 6.9), as the soil above the pipe
crown settled by 4.0mm compared to 2.5mm at offsets of 200mm. Beneath the pipe
invert, however, the reverse settlement pattern was recorded as the soil settled by
1.0mm compared to 1.5mm at offsets of 200mm. This similarly indicates positive
arching, since the stresses on the sidefill will be greater than those beneath the pipe.

Contour plots of the horizontal and vertical displacement fields are presented for each
test during the application of surface stress, using the locations of points on completion
of the backfill (Step 5) as datum readings (Appendix 6). Typical examples of vertical
displacement contours are presented in Figures 6.12 and 6.13 to highlight the
behaviour of a buried rigid pipe subjected to static and cyclic surface stress. Figures
6.12 and 6.13 were recorded during Tests 120WPL1 and 120WPL2.5 respectively,
during the application of a 150kPa static surface stress and the 1000th cycle of 70kPa
surface stress. Typical examples of vertical displacement contours are Similarly
presented in Figures 6.14 and 6.15 for Tests 250WPL1 and 168WPL3.5 during the
application of a 150kPa static surface stress and 1000th cycle of 70kPa surface stress,
respectively.

The vertical displacement contour plots have been used to develop general
arrangement diagrams for the behaviour of the buried pipe systems. Figure 6.16 shows
the relative displacements for buried rigid and flexible pipe systems under static and
cyclic loading, in which the development of positive and negative arching strains are indicated.

For a rigid pipe, the greater stiffness of the pipe with respect to the surrounding soil induces negative arching. Figure 6.12 shows that during the application of 150kPa during Test 120WPL1 a negative arching settlement pattern was not recorded as the pipe and surrounding soil settled by approximately the same magnitude. In addition, the settlement pattern was complicated by the influence of the water bag arrangement, which is discussed in greater detail in Section 7.1. To overcome this problem, the cover depth was increased during Test 120WPL2.5 but the application of 150kPa static surface stress after the application of 1000 cycles of 70kPa surface stress resulted in a similar settlement pattern as that shown in Figure 6.12. These results indicated that the initial stiffness of the loose sand was comparable to that of the clay pipe and that the magnitude of the applied static surface stress was insufficient to mobilise measurable negative arching strains. Adopting Marston's (1930) interpretation, Figure 6.16a shows that at the critical plane the settlement of the surrounding soil is greater than that of the pipe. The influence of the pipe diminishes both vertically and laterally away from it resulting in the formation of a 'height of equal settlement' above the pipe crown and a uniform settlement pattern laterally away from it. The formation of negative arching strains results in the application of stresses on the buried pipe in excess of the overburden. This causes the invert of the pipe to settle relative to the adjacent material. Under cyclic loading, the pattern of behaviour shown during static loading (from Marston) was modified by the state of compaction of the soil beneath the pipe haunches. During these tests the sand was hand placed beneath these regions in a very loose state. Under repeated loading the greatest permanent deformation occurred beneath the pipe haunches as a consequence. This zone of compression leads to a 'funnelling' pattern of settlement causing the exterior prism to be increasingly 'dragged' around the rigid pipe.

For a flexible pipe, the greater stiffness of the surround compared to the pipe leads to an opposite settlement pattern. Figure 6.16b shows that the settlement of the soil above the pipe crown is greater than that of the surround. The resulting positive
arching strains reduce the applied stress on the pipe below that of the overburden causing the invert to settle less than that of the surrounding soil. This leads to the formation of a small zone of negative arching strains adjacent to the pipe haunches.

6.1.6 Settlement profiles
To aid the interpretation of the relative displacement of the buried pipe and surrounding soil, settlement profiles at the level of the pipe crown, defined by Marston as the critical plane, and the pipe invert have been determined from the vertical displacement contour plots. Combined with the measurements of diametral strain and imposed stress, these profiles provide an additional insight into the behaviour of the buried pipe and the surrounding soil since they provide an intermediate step in the analysis between the soil strain distributions and the buried pipe measurements.

Table 6.9 shows the measured settlements at the pipe crown and invert relative to those at lateral offsets for each of the eleven buried pipe tests during the application of a 150kPa uniform static surface stress (Step 17), using the locations of the sand grains and pipe wall at Step 5 as datum readings. The readings from the pipe crown level clearly show the occurrence of positive arching during the buried PVC-U pipe tests as the settlement of the pipe crown generally exceeds that of the surrounding soil. This was shown for all the buried PVC-U pipe tests except Tests I 68WPL2.5 and I 68WPL3.5 where the degree of positive arching was reduced by the increase of pipe stiffness compared to the 250mm diameter pipe and the cyclic loading conditioning the settlement pattern due to the presence of very loose material beneath the pipe haunches. By comparison, the converse of negative arching is shown for the buried clay pipe Test 120WPL2.5.

For Tests 168WPL2.5, 168WPL3.5 and 120WPL2.5, the equivalent measured settlements to those in Table 6.9 are presented for the 1st, 100th and 500th applications of 70kPa uniform surface stress (Steps 7, 9 and 11 respectively) in Table 6.10. Examination of the behaviour of both flexible and rigid buried pipes shows that the measured displacements at the crown and invert positions increased, decreasing exponentially with increasing number of applications (see Section 6.1.7). These values
also show that a similar pattern of behaviour was recorded from the soil displacements. Due to the use of the 168mm diameter PVC-U pipe during Tests 168WPL2.5 and 168WPL3.5, which has an increased stiffness compared with that of the 250mm diameter PVC-U pipe, the occurrence of positive arching is less clearly demonstrated as the pipe displacements at both levels typically approximate to those of the adjacent soil, although similar trends are apparent. During the clay pipe test (120WPL2.5) however, the occurrence of negative arching was clearly seen from the recorded soil settlements significantly exceeding those of the pipe settlement.

A typical example of a set of settlement profiles at the level of the pipe crown and invert are shown in Figure 6.17 from Test 250WPL1* during the application of a 150kPa uniform surface stress, using the position of the sand particles and pipe wall at Step 5 as datum readings. The figure shows that the buried 250mm diameter PVC-U pipe deformed under the application of the surface stress as a greater settlement was recorded at the pipe crown compared to that at the invert position (i.e. at an offset of 0mm). In addition, since the soil adjacent to the pipe crown was closer to the application of the surface stress than that adjacent to the invert the recorded soil settlements were greater. The influence of the buried pipe on the resulting settlement pattern is clearly shown at both levels, as the soil settlements dissipate in magnitude away from the pipe crown, whilst the converse occurred at the level of the pipe invert. These settlement profiles correspond to the general settlement pattern (see Figure 6.16b) for a buried flexible pipe system under the application of a static surface stress.

As the vertical settlement contour plot previously showed (see Figure 6.11), a more complicated settlement pattern was recorded during Test 250PPL1 compared to that during Test 250WPL1* (Figure 6.14). The settlement profiles for Test 250PPL1 are presented in Figure 6.18, which shows that the maximum settlement at the level of the pipe crown occurred at the pipe shoulder (offsets ±63mm) rather than at the pipe crown (see Figure 6.17). Similarly, at the level of the pipe invert the soil settlement at the pipe haunches (offsets ±63mm) is shown to be greater than that recorded at either the invert or at an offset of ±200mm. The resulting gradient of the settlement profiles from the points of maximum settlement (63mm) to those at offsets of 200mm are
shown to be greater in Test 250PPL1 than 250WPL1, indicating larger magnitudes of shear strain. These settlement profiles can be explained by the absence of soil support beneath the pipe haunches (i.e. omission of Step 2) causing the pipe to deform into these regions, resulting in a flattening of the pipe invert.

6.1.7 Pipe diametral strains

From the displacements measured on the inner pipe wall, both the diametral strain and overall pipe displacement have been calculated in the horizontal and vertical directions. A positive diametral strain being denoted as a decrease in the diameter of the pipe (as in Figure 1.1). The overall pipe displacements correspond to the rigid body behaviour of the pipe, and were determined as the average of the pipe crown and invert displacements for the vertical direction and both the springline locations for the horizontal direction. These four sets of measurements are presented for each buried pipe test in Appendix 7, using both the completion of the bedding and backfill operations (Steps 1 and 5) as datum readings.

Measured values of vertical diametral strain (VDS), horizontal diametral strain (HDS) and overall vertical and horizontal pipe displacement are shown on completion of the backfill (Step 5) and during the application of a 150kPa uniform surface stress for each of the nine PVC-U buried pipe tests in Table 6.11. These measurements are presented in Table 6.12 for the application of the 1\textsuperscript{st}, 100\textsuperscript{th} and 500\textsuperscript{th} cycles of 70kPa uniform surface stress during Tests 168WPL2.5, 168WPL3.5 and 120WPL2.5.

The values in Tables 6.11 show that on completion of the backfill (Step 5) a negative VDS was recorded in each of the PVC-U pipe tests except for Test 250PPL1. The largest values of -3.58 and -1.93\% were recorded during Tests 250WPD1\textsuperscript{\textdagger} and 250WPD1 respectively in which the sidefill material (Steps 3 and 4) was compacted after levelling. It should be noted that the only occasion when a buried pipe became raised from the bedding was during Test 250WPD1, this being manifested by the formation of a prominent voidage beneath the pipe invert and an overall vertical displacement of 5.58mm. Repeating this test (Test 250WPD1\textsuperscript{\textdagger}), in which a 4kg steel bar was placed at the pipe invert and the sidefill was compacted in 50mm thick layers
using a 1.5kg wooden tamper, ensured that the pipe remained in contact with the bedding material and thus the overall vertical pipe displacement was limited to 3.67mm. By contrast, placing the sidefill and backfill material in a loose state resulted in a maximum negative VDS of 0.45%.

Subsequent application of a surface stress (Steps 6 to 17) resulted in a negligible change of overall horizontal pipe displacement with respect to that measured on completion of the backfill (Step 5), but significant vertical displacement. Figure 6.19 shows a typical relationship of measured VDS and HDS for Test 168WPL2.5 during the application of cyclic and static surface stress. It shows that the VDS increased linearly during the application of the first phase of static surface stress (Step 7, corresponding to a maximum applied surface stress of 70kPa, or an applied load of approximately -8kN/m), whilst the HDS became constant after the application of a 50kPa surface stress (Step 6, approximately -6kN/m). Application of 500 cycles of 70kPa surface stress (Steps 7 to 12, approximately -8kN/m), resulted in a non-linear increase of both VDS and HDS which is shown by the larger increase of diametral strain after 100, compared to a further 400, cycles. Increasing the magnitude of the static surface stress to a maximum of 150kPa (Step 17, approximately -16.5kN/m) resulted in a non-linear increase of both VDS and HDS.

Figure 6.20 shows that during both Tests 120WPL1 and 120WPL2.5, the pipes settled approximately linearly during the application of static surface stress. This figure also shows that in correspondence with Figure 6.19, the increase in pipe settlement during Test 120WPL2.5 was non-linear with the number of cyclic applications, the greatest displacements occurring during the initial cycles.

6.1.8 Soil strains
Shear and volumetric soil strains were determined by a computer program from the displacement of discrete points using the method outlined by Roscoe et al (1963), see Appendix 8. The program calculates four components of strain based upon horizontal and vertical displacements in both the horizontal and vertical planes. Due to the lack of clarity of the horizontal displacements, however, the two corresponding components of
strain have been omitted in the calculation of soil strains for all tests except during Test 120WPL2.5.

The volumetric strains were defined as positive for an extension, whereas the sign of shear strain was reversed between either side of the vertical plane to produce a mirror image pattern. For this reason, on the LHS of the vertical pipe axis the occurrence of positive and negative arching strains is denoted by positive and negative signs respectively, whereas on the RHS the notation of the shear strain is reversed. The philosophy behind the computer program and comparisons undertaken to check the precision and the accuracy of the resulting distributions is described in Appendix 8.

Using the vertical displacements only, the shear and volumetric soil strains recorded during Test 250PPL1 due to the application of the 150kPa uniform static surface stress are shown in Figures 6.21 and 6.22 respectively, using the positions of the sand grains and pipe wall at Step 5 as datum readings. The vertical displacement contour plot for this test (Figure 6.11) showed that the pipe deformed excessively at the pipe haunch regions and this subsequently caused the pipe crown to settle significantly with respect to the surrounding soil. The resulting shear strain distribution (Figure 6.21) showed the occurrence of very clearly defined positive arching strains, extending from the pipe wall into the sidefill as negative strains were recorded to the RHS of the pipe, whilst positive signs were shown to the LHS side. Maginitudes of up to 9% were recorded beneath the pipe haunches, which corresponded to the largest magnitudes in the buried pipe test series, and the 2% contours of strain are shown to extend vertically by 60mm and 100mm above the pipe crown on the LHS and RHS of the pipe respectively. The associated volumetric strain distribution (Figure 6.22) shows that negative, compressive strains were recorded around the complete circumference of the pipe wall. The excess deformation of the pipe haunches is shown by the peak compressive strains of 12% and 22% at the RHS and LHS respectively, these values similarly being the largest recorded volumetric strains during the buried pipe test series. Directly above the pipe crown a region of dilation was recorded, denoted by positive volumetric strains of up to 2%.

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Figures 6.23, 6.24 and 6.25 show the shear strain distributions during Tests 250WPL1, 250WPDL1 and 250WPL2, respectively, during the application of the 150kPa uniform static surface stress (Step 17), using Step 5 as datum readings. In addition to Figure 6.21, these four shear strain distributions have been used as the basis of the general arrangement diagrams shown in Figure 6.26. This shows schematic strain distributions that have been related to the shape of the deformed pipe inferred from the internal circumferential strain gauge readings (see Section 6.1.1).

The occurrence of negative arcing was recorded from the displacement data during the buried clay pipe test, 120WPL2.5. Figure 6.27 shows the shear strain distribution recorded during this test under the application of 500th cycle of 70kPa uniform surface stress (Step 11), using Step 5 as datum readings. In contrast to the shear strain distributions recorded during the buried PVC-U pipe tests, Figure 6.27 indicates the formation of negative arcing strains via negative and positive strains extending from the LHS and RHS pipe shoulders respectively. Although not continuous, these strains are shown to extend vertically 150mm above the pipe crown, forming an apex. Between these two zones of negative arcing strain, and directly above the pipe crown, a 0% shear strain region is recorded.

6.2 TUNNEL TEST RESULTS

6.2.1 Introduction

Two experiments were conducted to simulate tunnel construction by propagating ground movements by a tail void loss. Strain gauge measurements and a set of photographs were taken at each stage of the withdrawal process. The aim of this section is to present these measurements, and in Section 7.7 to discuss them, particularly with respect to other studies of ground movements around tunnels and the buried pipe results presented herein.

6.2.2 Strain gauge measurements

As discussed in Section 5.9, the inner tube remained restrained against the glass throughout each test. The strain gauge measurements correspondingly recorded this
restraining action and not the imposed stress on the lining due to the soil overburden and thus are not presented.

6.2.3 Soil displacements
A uniform tail void of 25mm was used in both tunnel tests. Due to the large magnitude of sand displacements induced by this void, it was not possible to view the same sand particles between two sequential stages of the withdrawal within a small area adjacent to the lining (termed the zone of unobservable movement and shown by the area marked in Figure 6.28). During Test T2 an attempt was made to reduce the size of the unobservable area by placing strands of five, 2mm diameter reflective beads adjacent to the glass face and surrounded by sand (see Section 5.5). The similarity of the displacements of sand grains and beads in similar areas indicated that this aim had been achieved, thus vector and contour plots created for Test T2 are based upon movements of both bead and sand particles. Whilst not influencing significantly the extent of the zone of the unobservable movement, the use of beads did permit more displacements to be recorded within a narrow area immediately adjacent to it.

6.2.3.1 Extent of displacements
Since it was not possible to record the position of the same sand grain at each stage throughout the withdrawal process, the digitising process was conducted on the basis of identifying the same sand grain on sequential photographs (i.e. Steps 1-2, then Steps 2-3, etc. and not Steps 1 through to the end). Vertical and horizontal sand displacement contour plots are shown for both T1 and T2 in Appendix 9. Positive vertical and horizontal displacements are denoted as upward and to the right respectively. The maximum horizontal and vertical settlements are derived from these contour plots and are presented in Table 6.13 for each sequential step of withdrawal to the nearest 1 and 5mm respectively (i.e. equalling the intervals of settlement contours). Plotting the displacements in sequential steps, only, allowed the maximum vertical and horizontal extents of both horizontal and vertical movements to be determined. This method does not, however, allow the cumulative displacements to be shown (i.e. with reference to the initial position, Step 1), which can only be determined by adding the separate sequential displacements together at a desired point. The extents and
directions of the vertical and horizontal movements have been summarised in a general arrangement diagram shown in Figure 6.29. The dimensions denoted by the symbols A, B, C, D, E, F and G in Figure 6.29 have been measured from the contour plots and are summarised in Tables 6.14 to 6.16.

Figure 6.29 indicates that for both tests the induced vertical movements occurred in a 'bulb' shape extending from the invert of the lining to near the ground surface. The vertical displacements were observed to reach the ground surface after a withdrawal distance of 91 and 25mm during tests T1 and T2 respectively. In addition, after a withdrawal distance of 91mm during T2 the lowest point of the extent of vertical displacement occurred above the pipe springline, which is indicated by a positive value of dimension C. As expected the movements were convergent (i.e. towards the lining), as indicated by the downward arrow. The position of the maximum settlement was, however, observed to occur at the lining shoulders, rather than at the crown, resulting in a twin-peak settlement pattern adjacent to the lining. Above the lining crown the formation of two zones of maximum displacement was seen to converge, resulting in a single, peak settlement. During T2, the formation of a single peak settlement trough was also observed after a withdrawal of 91mm (see Table 6.13).

The pattern of the horizontal movements was however more complicated, both in terms of distribution and direction. Figure 6.29 shows that either side of the springline two bulbs of horizontal movement were observed. The more clearly defined upper bulb was of greater extent than the lower bulb and the magnitudes of displacement within it were greater.
Figure 6.1 - Internal Pipe Wall Strain Distribution for Tests 250WPD1\(^{\wedge}\), 250WPL1 and 250PPL1 under 150kPa Uniform Static Stress (Step 17), Step 5 as datum
Figure 6.2 Types of Deformation and Associated Internal Strain Distribution
Figure 6.3 - Circumferential Pipe Wall Strain Distribution for Test 168WPL3.5 under 70kPa Uniform Cyclic Surface Stress (Steps 7, 9 and 11), Step 5 as datum
Figure 6.4 - Relationship of Shear Stress against Shear Strain for Dense and Loose Leighton Buzzard Sand against Glass at a Normal Stress of 220 kN/m²
Figure 6.5 - Mohr-Coulomb Failure envelope for Dense and Loose Leighton Buzzard Sand against Glass
Figure 6.6 - Comparison of Internal and External Circumferential Strains at Front and Central sections during Test 250WPL1* under static surface stress, using Step 5 as datum readings
Figure 6.7 - Relationship of Applied and Imposed Load at Front and Mid-Length sections during Test 250WPL1*, Datum as Step 5
Figure 6.8 - Relationship of Applied and Imposed Load during Test 168WPL3.5, Datum as Step 5
Figure 6.9  Effect of 150kPa static surface stress in TEST 250PPL1 (Step 17), Datum Step 5
Figure 6.10  PIPE TEST No 250PPL1  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

Figure 6.11  PIPE TEST No 250PPL1  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 6.12 PIPE TEST No 120WPL1  
STEP 5 - 17  
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

Figure 6.13  PIPE TEST No 120WPL2.5  STEP 5 - 13
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle
Figure 6.14  PIPE TEST No 250WPL1
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 6.15  PIPE TEST No 168WPL3.5  STEP 5 - 13
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle
Figure 6.16  Schematic settlement contours associated with buried rigid and flexible pipes under static and cyclic loading.
Figure 6.17 - Comparison of Settlements measured at Pipe Crown and Invert Levels during Test 250WPL1* under 150kPa Uniform Static Surface Stress (Step 17), Datum Step 5
Figure 6.18 - Comparison of Settlements measured at Pipe Crown and Invert Levels during Test 250PPL1 under 150kPa Uniform Static Surface Stress (Step 17), Datum Step 5
Figure 6.19 - Relationship of Vertical and Horizontal Diametral Strains (Datum as Step 1) against Applied Load (Datum as Step 5) during Test 168WP1.2.5
Figure 6.20 - Relationship of Vertical Displacement (Datum as Step 1) against Applied Load (Datum as Step 5) during Tests 120WPL1 and 120WPL2.5
Figure 6.21  Shear strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), Datum Step 5
Figure 6.22  Volumetric strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), Datum Step 5

-VE DENOTES COMPRESSION

100 mm
Figure 6.23  Shear strain distribution around Test 250WPL1 under 150kPa surface stress (Step 17), Datum Step 5
Figure 6.24 Shear strain distribution around Test 250WPDI under 150kPa surface stress (Step 17), Datum Step 5
Figure 6.25  Shear strain distribution around Test 250WPL2 under 150kPa surface stress (Step 17), Datum Step 5.
Examples:--
Tests 250WPD1* and 250WPL1 RHS
ELLIPtical DEFORMATION
Test 250WPL2
'HEART'
Test 250PPL1
'INVERTED-HEART'

Figure 6.26  Schematic shear strain distributions and associated type of deformation (Exaggerated scale)
Figure 6.27  Shear strain distribution around Test 120WPL2.5 under 500th cycle of 70kPa surface stress (Step 11), Datum Step 5
Figure 6.28  Zone of unobservable movement
Figure 6.29  General arrangement of ground movements during tail void simulated tunnel tests
Table 6.1  Internal circumferential pipe wall strains ($x 10^{-6}$) for PVC-U pipes, except Test 250PPLA1 which is based on external values, on completion of backfill (Step 5), datum as Step 1

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Gauge Location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>250PPLA1</td>
<td>214</td>
</tr>
<tr>
<td>250WPDI</td>
<td>-672</td>
</tr>
<tr>
<td>250WPL1</td>
<td>-400</td>
</tr>
<tr>
<td>250PPL1</td>
<td>-275</td>
</tr>
<tr>
<td>250WPDI*</td>
<td>-95</td>
</tr>
<tr>
<td>250WPL2</td>
<td>-345</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>-88</td>
</tr>
<tr>
<td>(Mid-Way)</td>
<td>-95</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>86</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>29</td>
</tr>
</tbody>
</table>

Notes for Tables 6.1 to 6.12:–

Unless otherwise stated, strain gauge and imposed stress measurements are taken from the front section of pipe.

N/A  Measurement not applicable. G/F  Strain gauge fault.  Sign convention for strains:-  Positive = tensile

----- Denotes Test 250WPL1\* front and mid-way section measurements
Table 6.2  Internal circumferential pipe wall strains (x 10^-5) for PVC-U pipes, except Test 250PPLA1 based on external values, during application of a uniform static surface stress of 150kPa (Step 17), datum as Step 1

<table>
<thead>
<tr>
<th>Test Code</th>
<th>0°</th>
<th>45°</th>
<th>90°</th>
<th>135°</th>
<th>180°</th>
<th>225°</th>
<th>270°</th>
<th>315°</th>
</tr>
</thead>
<tbody>
<tr>
<td>250PPLA1</td>
<td>41</td>
<td>-410</td>
<td>112</td>
<td>-348</td>
<td>-2682</td>
<td>-348</td>
<td>4</td>
<td>101</td>
</tr>
<tr>
<td>250WPD1</td>
<td>-1180</td>
<td>-723</td>
<td>-332</td>
<td>78</td>
<td>-2348</td>
<td>385</td>
<td>-249</td>
<td>-1063</td>
</tr>
<tr>
<td>250WPL1</td>
<td>-871</td>
<td>-286</td>
<td>-1216</td>
<td>-421</td>
<td>-219</td>
<td>-1390</td>
<td>-972</td>
<td>-688</td>
</tr>
<tr>
<td>250PPL1</td>
<td>-636</td>
<td>-610</td>
<td>-920</td>
<td>400</td>
<td>2820</td>
<td>-1446</td>
<td>-909</td>
<td>-746</td>
</tr>
<tr>
<td>250WPD1*</td>
<td>-1152</td>
<td>-263</td>
<td>337</td>
<td>G/F</td>
<td>-1496</td>
<td>-203</td>
<td>526</td>
<td>-595</td>
</tr>
<tr>
<td>250WPL2</td>
<td>-876</td>
<td>-1236</td>
<td>-948</td>
<td>-918</td>
<td>-20</td>
<td>-3</td>
<td>-1380</td>
<td>-990</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>-356</td>
<td>N/A</td>
<td>-1269</td>
<td>N/A</td>
<td>-634</td>
<td>N/A</td>
<td>-1074</td>
<td>N/A</td>
</tr>
<tr>
<td>(Mid-Way)</td>
<td>-825</td>
<td>N/A</td>
<td>-1298</td>
<td>N/A</td>
<td>-949</td>
<td>N/A</td>
<td>-1377</td>
<td>N/A</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>561</td>
<td>-265</td>
<td>-860</td>
<td>G/F</td>
<td>990</td>
<td>-801</td>
<td>-1214</td>
<td>-666</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>723</td>
<td>-716</td>
<td>-1045</td>
<td>-296</td>
<td>653</td>
<td>-546</td>
<td>-1002</td>
<td>-327</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
Table 6.3  Internal circumferential pipe wall strains (x 10\(^{-5}\)) for PVC-U pipe during the application of uniform cyclic stress of 70kPa (Steps 7 to 12), datum as Step 1

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Description</th>
<th>Step No</th>
<th>Strain Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>168WPL2.5</td>
<td>1(^{st}) On</td>
<td>7</td>
<td>192 -261 -225 G/F 271 -426 -730 -191</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>1(^{st}) Off</td>
<td>8</td>
<td>218 -132 -6 G/F 384 -221 -308 -90</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>100(^{th}) On</td>
<td>9</td>
<td>377 -167 -369 G/F 692 -545 -716 -382</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>100(^{th}) Off</td>
<td>10</td>
<td>392 -82 -222 G/F 806 -337 -383 -275</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>500(^{th}) On</td>
<td>11</td>
<td>547 -79 -531 G/F 1006 -531 -712 -463</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>500(^{th}) Off</td>
<td>12</td>
<td>546 -26 -363 G/F 1089 -350 -425 -369</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Description</th>
<th>Step No</th>
<th>Strain Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>168WPL3.5</td>
<td>1(^{st}) On</td>
<td>7</td>
<td>65 -428 -460 -200 -12 -347 -735 -154</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>1(^{st}) Off</td>
<td>8</td>
<td>96 -353 -302 -73 130 -233 -526 -93</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>100(^{th}) On</td>
<td>9</td>
<td>388 -413 -602 -100 341 -315 -625 -195</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>100(^{th}) Off</td>
<td>10</td>
<td>381 -358 -482 23 500 -225 -469 -163</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>500(^{th}) On</td>
<td>11</td>
<td>598 -421 -728 -95 644 -410 -610 -186</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>500(^{th}) Off</td>
<td>12</td>
<td>567 -389 -640 21 806 -302 -515 -187</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
Table 6.4 Comparison of measured external axial and hoop strains, and shape of deformed pipe during application of 150kPa surface static stress (Step 17), using completion of bedding and backfill (Steps 1 and 5 respectively) as datum readings.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Measured Strain (με) Step 17, datum readings at Step 1</th>
<th>Measured Strain (με) Step 17, datum readings at Step 5</th>
<th>Pipe Shape Response Step 17, datum readings at Steps 1 and 5 from eight internal hoop gauges</th>
<th>Plane Stress or Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>250WPD1</td>
<td>Axial: -52, Hoop: -755</td>
<td>Axial: -165, Hoop: -130</td>
<td>Reverse Ellipse</td>
<td>Stress</td>
</tr>
<tr>
<td>250WPL1</td>
<td>Axial: 249, Hoop: 95</td>
<td>Axial: 334, Hoop: 327</td>
<td>RHS-Ellipse</td>
<td>Stress</td>
</tr>
<tr>
<td>250PPL1</td>
<td>Axial: 102, Hoop: 21</td>
<td>Axial: 179, Hoop: 218</td>
<td>LHS-Inverted 'Heart'/Y</td>
<td>Stress</td>
</tr>
<tr>
<td>120WPL1</td>
<td>Axial: -215, Hoop: -160</td>
<td>Axial: -121, Hoop: -20</td>
<td>Inverted Heart</td>
<td>Stress</td>
</tr>
<tr>
<td>250WPD1*</td>
<td>Axial: 133, Hoop: -182</td>
<td>Axial: 42, Hoop: -136</td>
<td>Inverted Ellipse</td>
<td>Stress</td>
</tr>
<tr>
<td>250WPL2</td>
<td>Axial: 90, Hoop: -342</td>
<td>Axial: 226, Hoop: -197</td>
<td>Heart</td>
<td>Stress</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>Axial: 175, Hoop: -188</td>
<td>Axial: 169, Hoop: -182</td>
<td>Ellipse (4 gauges)</td>
<td>Stress</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>Axial: 151, Hoop: 184</td>
<td>Axial: 161, Hoop: 161</td>
<td>Ellipse</td>
<td>Stress</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>Axial: 36, Hoop: 51</td>
<td>Axial: 27, Hoop: 45</td>
<td>Ellipse</td>
<td>Stress</td>
</tr>
</tbody>
</table>

See notes for Table 6.1 and Reverse Ellipse indicates a negative VDS (i.e. extension of vertical diameter)
Table 6.5  Predicted stress for Test 250WPL1 at central section, across pipe crown using shear plane method (Step 3 datum).

<table>
<thead>
<tr>
<th>Applied prism stress (kN/m²)</th>
<th>Calculated uniform stress at pipe crown (kN/m²)</th>
<th>Total shear stress (kN/m²)</th>
<th>Predicted applied load (kN/m) per springline</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Krynine K (Eqn. 2.3) Marston, Ks, Krynine Marston</td>
<td>Krynine Marston Krynine Marston</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.5m wide 1.0m wide 1.5m wide 1.0m wide</td>
<td>Krynine</td>
<td>Marston</td>
</tr>
<tr>
<td>-53.75</td>
<td>-48.3 -45.7 -50.5 -49.0</td>
<td>13.5 7.9</td>
<td>-5.0 -5.7</td>
</tr>
<tr>
<td>-103.75</td>
<td>-93.0 -88.0 -97.5 -94.5</td>
<td>26.5 15.6</td>
<td>-9.7 -11.0</td>
</tr>
<tr>
<td>-153.75</td>
<td>-137.7 -130.3 -144.4 -139.9</td>
<td>39.5 23.2</td>
<td>-14.3 -16.3</td>
</tr>
</tbody>
</table>

Table 6.6  Comparison of measured and predicted loads during Test 250WPL1 at the glass and central sections

<table>
<thead>
<tr>
<th>Applied prism load (kN/m)</th>
<th>Predicted applied load (kN/m) Krynine Marston</th>
<th>Measured load (kN/m) Krynine Marston</th>
<th>Degree of Loading (%) Krynine Marston</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Glass Centre Glass Centre</td>
<td>Glass Centre Error Glass Centre Error</td>
<td></td>
</tr>
<tr>
<td>-13.4</td>
<td>-9.5 -10.0 -10.0 -11.4</td>
<td>-3.8 -5.0</td>
<td>40 50 10 38 44 6</td>
</tr>
<tr>
<td>-25.9</td>
<td>-18.4 -19.3 -19.2 -22.0</td>
<td>-7.4 -10.6</td>
<td>40 55 15 38 48 10</td>
</tr>
<tr>
<td>-38.4</td>
<td>-27.2 -28.6 -28.4 -32.6</td>
<td>-10.2 -15.7</td>
<td>38 55 17 36 48 12</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
Table 6.7 Degree of Loading, Applied and Imposed Load for PVC-U pipes during the application of a uniform static surface stress of 150kPa (Step 17), using Step 5 as datum readings.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Number of Springline Hoop Thrust Readings</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250WPD1</td>
<td>-13.245</td>
<td>1</td>
<td>-3.637</td>
<td>27</td>
</tr>
<tr>
<td>250WPL1</td>
<td>-13.245</td>
<td>1</td>
<td>-2.948</td>
<td>22</td>
</tr>
<tr>
<td>250PPL1</td>
<td>-13.245</td>
<td>1</td>
<td>-2.325</td>
<td>18</td>
</tr>
<tr>
<td>250WPDI</td>
<td>-13.245</td>
<td>2</td>
<td>-2.381</td>
<td>18</td>
</tr>
<tr>
<td>250WPL2</td>
<td>-11.844</td>
<td>2</td>
<td>-4.560</td>
<td>39</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>-13.245</td>
<td>2</td>
<td>-4.748</td>
<td>36</td>
</tr>
<tr>
<td>(Mid-Way)</td>
<td>-13.872</td>
<td>2</td>
<td>-7.488</td>
<td>54</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>-8.249</td>
<td>2</td>
<td>-4.133</td>
<td>49</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>-7.652</td>
<td>2</td>
<td>-3.573</td>
<td>47</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
Table 6.8  Degree of Loading, Applied and Imposed Load for PVC-U pipes during the application of uniform cyclic surface stress of 70kPa (Steps 7 to 12), using Step 5 as datum readings.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Description</th>
<th>Step No</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Number of Springline Hoop Thrust Readings</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>168WPL2.5</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; On</td>
<td>7</td>
<td>-3.850</td>
<td>2</td>
<td>-2.216</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; Off</td>
<td>8</td>
<td>0</td>
<td>2</td>
<td>-0.643</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>100&lt;sup&gt;th&lt;/sup&gt; On</td>
<td>9</td>
<td>-3.850</td>
<td>2</td>
<td>-2.116</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>100&lt;sup&gt;th&lt;/sup&gt; Off</td>
<td>10</td>
<td>0</td>
<td>2</td>
<td>-0.786</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>500&lt;sup&gt;th&lt;/sup&gt; On</td>
<td>11</td>
<td>-3.850</td>
<td>2</td>
<td>-1.891</td>
<td>49</td>
</tr>
<tr>
<td></td>
<td>500&lt;sup&gt;th&lt;/sup&gt; Off</td>
<td>12</td>
<td>0</td>
<td>2</td>
<td>-0.648</td>
<td>N/A</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; On</td>
<td>7</td>
<td>-3.571</td>
<td>2</td>
<td>-2.290</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; Off</td>
<td>8</td>
<td>0</td>
<td>2</td>
<td>-1.294</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>100&lt;sup&gt;th&lt;/sup&gt; On</td>
<td>9</td>
<td>-3.571</td>
<td>2</td>
<td>-1.818</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>100&lt;sup&gt;th&lt;/sup&gt; Off</td>
<td>10</td>
<td>0</td>
<td>2</td>
<td>-1.022</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>500&lt;sup&gt;th&lt;/sup&gt; On</td>
<td>11</td>
<td>-3.571</td>
<td>2</td>
<td>-1.676</td>
<td>47</td>
</tr>
<tr>
<td></td>
<td>500&lt;sup&gt;th&lt;/sup&gt; Off</td>
<td>12</td>
<td>0</td>
<td>2</td>
<td>-1.108</td>
<td>N/A</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
### Table 6.9  Summary of Pipe and Soil Displacements due to application of 150kPa Uniform Surface Static Stress (Step 17), using Step 5 locations as datum readings.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Displacements (mm) at offsets (mm) at Pipe Crown level</th>
<th>Displacements (mm) at offsets (mm) at Pipe Invert level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-200</td>
<td>0</td>
</tr>
<tr>
<td>250PPLA1</td>
<td>-2.5</td>
<td>-3.6</td>
</tr>
<tr>
<td>250WPD1</td>
<td>-2.0</td>
<td>-2.8</td>
</tr>
<tr>
<td>250WPL1</td>
<td>-2.0</td>
<td>-2.1</td>
</tr>
<tr>
<td>250PPL1</td>
<td>-2.0</td>
<td>-2.9</td>
</tr>
<tr>
<td>120WPL1</td>
<td>-1.8</td>
<td>-2.1</td>
</tr>
<tr>
<td>250WPD1^</td>
<td>-1.5</td>
<td>-1.8</td>
</tr>
<tr>
<td>250WPL2</td>
<td>N/A</td>
<td>-2.8</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>-2.3</td>
<td>-2.8</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>-3.5</td>
<td>-3.6</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>-4.0</td>
<td>-3.4</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>-5.0</td>
<td>-2.5</td>
</tr>
</tbody>
</table>

See notes for Table 6.1

Additional notes for Tables 6.9 and 6.10

- Offsets measured from pipe crown and invert positions.
Table 6.10  Summary of pipe and soil displacements due to application of 70kPa uniform surface cyclic stress (Steps 7 to 12), using Step 5 locations as datum readings.

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Description</th>
<th>Step No</th>
<th>Displacements (mm)</th>
<th>Displacements (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>at offsets (mm) at Pipe Crown level</td>
<td>at offsets (mm) at Pipe Invert level</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-200</td>
<td>0</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>1st On</td>
<td>7</td>
<td>-0.8</td>
<td>-1.2</td>
</tr>
<tr>
<td></td>
<td>1st Off</td>
<td>8</td>
<td>-0.5</td>
<td>-0.9</td>
</tr>
<tr>
<td></td>
<td>100th On</td>
<td>9</td>
<td>-1.1</td>
<td>-2.0</td>
</tr>
<tr>
<td></td>
<td>100th Off</td>
<td>10</td>
<td>-1.1</td>
<td>-1.8</td>
</tr>
<tr>
<td></td>
<td>500th On</td>
<td>11</td>
<td>-2.0</td>
<td>-2.6</td>
</tr>
<tr>
<td></td>
<td>500th Off</td>
<td>12</td>
<td>-2.0</td>
<td>-2.5</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>1st On</td>
<td>7</td>
<td>-1.0</td>
<td>-1.1</td>
</tr>
<tr>
<td></td>
<td>100th On</td>
<td>9</td>
<td>-2.0</td>
<td>-2.0</td>
</tr>
<tr>
<td></td>
<td>500th On</td>
<td>11</td>
<td>-2.5</td>
<td>-2.5</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td>1st On</td>
<td>7</td>
<td>-0.8</td>
<td>-1.0</td>
</tr>
<tr>
<td></td>
<td>100th On</td>
<td>9</td>
<td>-2.0</td>
<td>-1.4</td>
</tr>
<tr>
<td></td>
<td>500th On</td>
<td>11</td>
<td>-4.0</td>
<td>-1.6</td>
</tr>
</tbody>
</table>

See notes for Table 6.1
Table 6.11  Diametral strain and overall displacement on completion of backfill (Step 5) and application of uniform surface stress of 150kPa (Step 17), datum as Step 1

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Completion of backfill (Step 5)</th>
<th>Overall</th>
<th>Overall</th>
<th>150kPa Uniform Static Stress (Step 17)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VDS (%)</td>
<td>HDS (%)</td>
<td>Vertical Displacement (mm)</td>
<td>Horizontal Displacement (mm)</td>
</tr>
<tr>
<td>250PPLA1</td>
<td>-0.41</td>
<td>1.08</td>
<td>-0.40</td>
<td>-2.26</td>
</tr>
<tr>
<td>250WPDI</td>
<td>-1.93</td>
<td>1.58</td>
<td>5.58</td>
<td>-1.68</td>
</tr>
<tr>
<td>250WPL1</td>
<td>-0.20</td>
<td>0.44</td>
<td>0.60</td>
<td>-0.37</td>
</tr>
<tr>
<td>250PPPL1</td>
<td>0.03</td>
<td>0.23</td>
<td>-0.17</td>
<td>-0.22</td>
</tr>
<tr>
<td>120WPL1</td>
<td>N/A</td>
<td>N/A</td>
<td>0.08</td>
<td>0</td>
</tr>
<tr>
<td>250WPDI1*</td>
<td>-3.58</td>
<td>3.13</td>
<td>3.67</td>
<td>1.12</td>
</tr>
<tr>
<td>250WPL2</td>
<td>-0.45</td>
<td>1.69</td>
<td>0.47</td>
<td>N/A</td>
</tr>
<tr>
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<td>-0.03</td>
<td>0.28</td>
<td>0.09</td>
<td>0.06</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>-0.27</td>
<td>-0.08</td>
<td>0.04</td>
<td>0.21</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>-0.16</td>
<td>0.07</td>
<td>0.07</td>
<td>0.33</td>
</tr>
<tr>
<td>120WPL2.5</td>
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<td>N/A</td>
<td>0</td>
<td>0</td>
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</table>

See notes for Table 6.1
Table 6.12  Diametral strain and overall displacement during the application of uniform cyclic stress of 70kPa (Steps 7 to 12), datum as Step 1

<table>
<thead>
<tr>
<th>Test Code</th>
<th>Description</th>
<th>Step No</th>
<th>VDS (%)</th>
<th>HDS (%)</th>
<th>Overall Vertical Displacement (mm)</th>
<th>Overall Horizontal Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>168WPL2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st On</td>
<td></td>
<td>7</td>
<td>0.04</td>
<td>-0.22</td>
<td>-0.85</td>
<td>0.35</td>
</tr>
<tr>
<td>1st Off</td>
<td></td>
<td>8</td>
<td>0.01</td>
<td>-0.33</td>
<td>-0.58</td>
<td>0.20</td>
</tr>
<tr>
<td>100th On</td>
<td></td>
<td>9</td>
<td>0.47</td>
<td>-0.71</td>
<td>-1.34</td>
<td>0.43</td>
</tr>
<tr>
<td>100th Off</td>
<td></td>
<td>10</td>
<td>0.35</td>
<td>-0.69</td>
<td>-1.22</td>
<td>0.38</td>
</tr>
<tr>
<td>500th On</td>
<td></td>
<td>11</td>
<td>0.60</td>
<td>-0.75</td>
<td>-1.82</td>
<td>0.45</td>
</tr>
<tr>
<td>500th Off</td>
<td></td>
<td>12</td>
<td>0.70</td>
<td>-0.75</td>
<td>-1.65</td>
<td>0.42</td>
</tr>
<tr>
<td>168WPL3.5</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st On</td>
<td></td>
<td>7</td>
<td>0.16</td>
<td>-0.25</td>
<td>-0.71</td>
<td>0.25</td>
</tr>
<tr>
<td>100th On</td>
<td></td>
<td>9</td>
<td>0.59</td>
<td>-0.67</td>
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<td>0.43</td>
</tr>
<tr>
<td>500th On</td>
<td></td>
<td>11</td>
<td>0.71</td>
<td>-0.93</td>
<td>-1.66</td>
<td>0.33</td>
</tr>
<tr>
<td>120WPL2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st On</td>
<td></td>
<td>7</td>
<td>N/A</td>
<td>N/A</td>
<td>-1.01</td>
<td>0</td>
</tr>
<tr>
<td>100th On</td>
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<td>N/A</td>
<td>N/A</td>
<td>-1.38</td>
<td>0</td>
</tr>
<tr>
<td>500th On</td>
<td></td>
<td>11</td>
<td>N/A</td>
<td>N/A</td>
<td>-1.57</td>
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</table>

See notes for Table 6.1
Table 6.13  Maximum displacements of sand particles for tunnel tests (T1 and T2). The values are quoted to the nearest 5mm and 1mm for the vertical and horizontal displacements respectively.

<table>
<thead>
<tr>
<th>Step</th>
<th>Withdrawal Distance (mm)</th>
<th>T1</th>
<th>T2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Vertical</td>
<td>Horizontal (Area E/F)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LHS</td>
<td>RHS</td>
</tr>
<tr>
<td>1 - 2</td>
<td></td>
<td>-20</td>
<td>-20</td>
</tr>
<tr>
<td>2 - 3</td>
<td></td>
<td>-10</td>
<td>-10</td>
</tr>
<tr>
<td>3 - 4</td>
<td></td>
<td>-15</td>
<td>-10</td>
</tr>
<tr>
<td>4 - 5</td>
<td></td>
<td>-15</td>
<td>-15</td>
</tr>
<tr>
<td>5 - 6</td>
<td></td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>6 - 7</td>
<td></td>
<td>-15</td>
<td>-10</td>
</tr>
<tr>
<td>7 - 8</td>
<td></td>
<td>-20</td>
<td>-15</td>
</tr>
<tr>
<td>8 - 9</td>
<td></td>
<td>-15</td>
<td>-10</td>
</tr>
<tr>
<td>9 - 10</td>
<td></td>
<td>-15</td>
<td>-10</td>
</tr>
<tr>
<td>10 - 11</td>
<td></td>
<td>-30</td>
<td>-20</td>
</tr>
<tr>
<td>11 - 12</td>
<td></td>
<td>-20</td>
<td>-15</td>
</tr>
<tr>
<td>12 - 13</td>
<td></td>
<td>-10</td>
<td>-10</td>
</tr>
<tr>
<td>13 - 14</td>
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<td>14 - 15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 6.14  Observed extents of vertical sand displacements for tunnel tests (T1 and T2) as defined in Figure 6.29.

| Steps | Withdrawal Distance (mm) |  |  |  |  |  |  |  |
|-------|--------------------------|---|---|---|---|---|---|
|       |                          | A | B | C |  | A | B | C |
|       |                          | LHS | RHS |     |     | LHS | RHS |     |
| 1 - 2 | 4                        | -0.78 | 0.95 | 1.20 | -0.50 | -0.76 | 0.74 | 1.65 |
| 2 - 3 | 7                        | -0.89 | 0.82 | 1.65 | -0.50 | -0.85 | 0.91 | 1.92 |
| 3 - 4 | 10                       | -0.76 | 0.76 | 2.16 | -0.50 | -0.82 | 0.87 | 2.47 |
| 4 - 5 | 15                       | -0.87 | 0.80 | 2.27 | -0.50 | -0.85 | 0.87 | 2.37 |
| 5 - 6 | 20                       | -0.89 | 0.91 | 2.78 | -0.50 | -0.95 | 0.82 | 2.26 |
| 6 - 7 | 25                       | -0.89 | 0.87 | 2.56 | -0.50 | -0.85 | 0.93 | 3.00 |
| 7 - 8 | 29                       | -0.89 | 1.03 | 3.34 | -0.50 | -0.87 | 0.91 | 3.00 |
| 8 - 9 | 32                       | -0.89 | 0.93 | 4.39 | -0.50 | -0.87 | 0.91 | 3.00 |
| 9 - 10| 37                       | -0.89 | 0.87 | 4.47 | -0.50 | -0.89 | 0.93 | 3.00 |
| 10 - 11| 54                      | -0.91 | 0.93 | 4.33 | -0.50 | -0.93 | 0.93 | 3.00 |
| 11 - 12| 72                      | -0.97 | 0.93 | 4.54 | -0.50 | -1.03 | 0.93 | 3.00 |
| 12 - 13| 91                      | -0.97 | 0.93 | 5.00 | -0.50 | -0.97 | 0.93 | 3.00 |
| 13 - 14| 120                     | -0.97 | 0.93 | 3.00 | 0.62 | -0.99 | 0.93 | 3.00 |
| 14 - 15| 150                     | -0.99 | 0.93 | 3.00 | 0.56 |
Table 6.15  

Observed extents of horizontal sand movements for tunnel test T1 as defined in Figure 6.29.

<table>
<thead>
<tr>
<th>Steps</th>
<th>Distance (mm)</th>
<th>Withdrawal</th>
<th>Observed extent / diameter of inner tube</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LHS</td>
<td>RHS</td>
</tr>
<tr>
<td>1 - 2</td>
<td>4</td>
<td>-0.62</td>
<td>0.70</td>
</tr>
<tr>
<td>2 - 3</td>
<td>7</td>
<td>-0.41</td>
<td>0.70</td>
</tr>
<tr>
<td>3 - 4</td>
<td>10</td>
<td>-0.68</td>
<td>0.72</td>
</tr>
<tr>
<td>4 - 5</td>
<td>15</td>
<td>-0.49</td>
<td>0.64</td>
</tr>
<tr>
<td>5 - 6</td>
<td>20</td>
<td>-0.82</td>
<td>0.74</td>
</tr>
<tr>
<td>6 - 7</td>
<td>25</td>
<td>-0.47</td>
<td>0.72</td>
</tr>
<tr>
<td>7 - 8</td>
<td>29</td>
<td>-0.70</td>
<td>0.64</td>
</tr>
<tr>
<td>8 - 9</td>
<td>32</td>
<td>-0.70</td>
<td>0.64</td>
</tr>
<tr>
<td>9 - 10</td>
<td>37</td>
<td>-0.72</td>
<td>0.78</td>
</tr>
<tr>
<td>10 - 11</td>
<td>54</td>
<td>-0.72</td>
<td>0.62</td>
</tr>
<tr>
<td>11 - 12</td>
<td>72</td>
<td>-0.78</td>
<td>0.62</td>
</tr>
<tr>
<td>12 - 13</td>
<td>91</td>
<td>-0.72</td>
<td>0.68</td>
</tr>
</tbody>
</table>
Table 6.16  Observed extents of horizontal sand movements for tunnel test T2 as defined in Figure 6.29.

<table>
<thead>
<tr>
<th>Steps</th>
<th>Withdrawal Distance (mm)</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LHS</td>
<td>RHS</td>
<td>LHS</td>
<td>RHS</td>
</tr>
<tr>
<td>1 - 2</td>
<td>4</td>
<td>-0.25</td>
<td>0.62</td>
<td>0.89</td>
<td>1.09</td>
</tr>
<tr>
<td>2 - 3</td>
<td>7</td>
<td>-0.72</td>
<td>0.72</td>
<td>0.76</td>
<td>1.07</td>
</tr>
<tr>
<td>3 - 4</td>
<td>10</td>
<td>-0.72</td>
<td>0.72</td>
<td>0.87</td>
<td>0.82</td>
</tr>
<tr>
<td>4 - 5</td>
<td>15</td>
<td>-0.74</td>
<td>0.72</td>
<td>0.78</td>
<td>0.76</td>
</tr>
<tr>
<td>5 - 6</td>
<td>20</td>
<td>-0.72</td>
<td>0.82</td>
<td>0.78</td>
<td>0.78</td>
</tr>
<tr>
<td>6 - 7</td>
<td>25</td>
<td>-0.74</td>
<td>0.72</td>
<td>0.72</td>
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</tr>
<tr>
<td>7 - 8</td>
<td>29</td>
<td>-0.74</td>
<td>0.72</td>
<td></td>
<td>0.72</td>
</tr>
<tr>
<td>8 - 9</td>
<td>32</td>
<td></td>
<td></td>
<td>-0.91</td>
<td>0.72</td>
</tr>
<tr>
<td>9 - 10</td>
<td>37</td>
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<td>0.76</td>
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</tr>
<tr>
<td>10 - 11</td>
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<tr>
<td>11 - 12</td>
<td>72</td>
<td>-0.72</td>
<td>0.93</td>
<td>0.93</td>
<td>0.97</td>
</tr>
<tr>
<td>12 - 13</td>
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<td>-0.74</td>
<td>0.72</td>
<td>1.48</td>
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<td>13 - 14</td>
<td>120</td>
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<td>0.76</td>
<td>1.44</td>
<td>1.30</td>
</tr>
<tr>
<td>14 - 15</td>
<td>150</td>
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</table>
7. DISCUSSION OF RESULTS

The aim of this chapter is to discuss the results previously summarised via typical plots and general arrangement diagrams. The method of placing the sidefill around a buried pipe and how it influences the behaviour of the pipe-soil structure under subsequent applied stresses is described in Section 7.1, which assess also the repeatability achieved by identical sand placement techniques. Sections 7.2 and 7.3 describe the influence of the pipe stiffness and depth of soil cover above the pipe crown respectively. The behaviour of the pipe-soil structure under static and cyclic uniform applied surface stresses is described in Sections 7.4 and 7.5 respectively. From these results comparisons are made between the measured behaviour of the model tests and those predicted using Marston's Load theory, the Iowa deflection method and the TRL method in Section 7.6. Section 7.7 discusses the soil movements recorded during the tunnel tests in comparison with those recorded by other laboratory based studies and field measurements reviewed in Chapter 3. An analysis is then presented of the similarities between the behaviour of the model buried pipes and tunnels.

7.1 EFFECT OF INSTALLATION

7.1.1 Introduction
The aim of this section is to assess how the installation procedure influences the behaviour of a buried pipe-soil structure under the subsequent application of a surface stress. Section 5.6 described the installation procedures used to simulate good and poor site practice, the test programme described in Section 5.8 being presented in Tables 5.9, 5.11 and 5.12. The notable differences between the well placed (WP) and poorly placed (PP) surrounds were that the sand was fed beneath the pipe haunches and was built up in layers in the former. For the WP cases, the sand was either levelled only to achieve a loose (L) condition or was compacted to achieve a dense (D) condition. In the first PPL test the sand was introduced into the tank asymmetrically (PPLA), which also resulted in differences in behaviour in comparison to the second test where the sand was placed evenly either side of the pipe (PPL).
Sections 7.1.2, 7.1.3 and 7.1.4 examine the relative behaviour of the pipes according to the use of the WPL, PPL and WPD installation procedures. The results from the WPL surrounded tests being discussed initially to assess the repeatability of the installation procedures and the measuring techniques. In each of these sections the response of the pipe to sidefill placement (Steps 1 to 4) is discussed initially. As the vector plots in Figures 7.11 and 7.13 show, the displacement of the soil particles during these steps was typically negligible so only the behaviour of the pipe is outlined. The subsequent behaviour of the pipe as the overburden stresses are applied (Steps 5 to 18) is thereafter examined. This discussion is limited to in Section 7.1.2 to highlight the repeatability of measurements during Tests 250WPL1 and 250WPL1*, in Section 7.1.3 to highlight the effect of omitting the placement of soil support beneath the pipe haunches (Step 2) during Tests 250PPL1 and 250PPLA1, and in Section 7.1.4 to highlight the effect of placing a 4kg steel bar at the pipe invert during Test 250WPD1 compared to 250WPD1. Section 7.1.5 then discusses the behaviour of the pipe-soil structure to examine the influence of the installation procedures under loading effects. The analysis presented herein expands upon that presented in Rogers et al (1996) which was based on the behaviour of the buried pipe only.

7.1.2 Well-Placed Loose (WPL) installations
The WPL method was the mostly frequently used of the four installation procedures, being used in seven of the eleven buried pipe tests. Assessment of repeatability has been conducted by comparing the behaviour of the pipe between tests during installation, and in a complete test (250WPL1) which was repeated.

7.1.2.1 Installation of PVC-U pipe
The installation of PVC-U pipe using the WPL technique occurred in Tests 250WPL1 (and 250WPL1*), 250WPL2, 168WPL2.5 and 168WPL3.5, using cover depths to diameter ratios of 1, 2, 2.5 and 3.5 respectively. Pipe stiffness was varied by using two pipes of different diameters (250 and 168mm respectively) and wall thickness. The installation procedures were identical for Steps 1 to 4 and thus pipe responses can be examined for similarity.
Using measurements at Step 1 (pipe positioned on the bedding) as a datum, the results of the internal circumferential pipe wall strains and pipe deflections for Step 2 (sand fed beneath pipe haunches) showed no significant change, as might be expected. This is illustrated in Table 7.1, which shows readings for Steps 1-2 that lie within the limits of precision measurement.

Raising the level of the sidefill to the pipe springline (Step 3) caused the horizontal diameter of the pipes to decrease and the vertical diameter to increase (i.e. positive HDS and negative VDS). This was because the lateral earth pressure of the sidefill was acting on pipes that were unrestrained in the vertical plane. Table 7.1 shows that the negative VDS values for the 250mm diameter pipes was -0.60, -0.34 and -0.07% with corresponding positive HDS of 0.65, 1.05 and 0.34% respectively. The differences between these measurements demonstrate the inherent variability in pipe response under even the most closely controlled conditions and concomitantly, and that the initial stiffness of the sidefill was likely to have varied between these tests. For example, the larger magnitudes of diametral strain of the first test (250WPL1) indicate greater lateral pressure of the sidefill and so greater initial sidefill stiffness than the latter test (250WPL1*).

These conclusions also agree with those drawn from the strain gauge measurements. As the sensitivity of these measurements is greater than that of the pipe displacements, they can be used to elucidate relative differences between the response of the pipe during installation with a greater degree of confidence. The internal strain distributions for these tests (Figure 7.1) confirms the general observation of a negative VDS response as compressive strains are observed at the crown and invert (0° and 180°), whilst tensile strains are observed at the springline (90° and 270°). Importantly, it indicates that the strain distributions for each test are broadly similar (being approximately elliptical about the vertical axis, i.e. the inverse of Figure 6.2a) whilst specific differences, in accordance with those from the measured displacements (for example the strains for 250WPL1* are considerably lower than those for 250WPL1), are readily discernible. Detailed observations of the shape of negative deformation (or...
vertical diametral elongation) are not warranted, however, since the magnitudes of the strain measurements are relatively small.

Figure 7.1 also shows that on the basis of the strain response of the pipes in the vertical plane (0 and 180°), the increased stiffness of the 168mm above that of the 250mm diameter pipe resulted in significantly reduced pipe wall strain. Whilst the strain distributions during both 168mm pipe tests were similar, greater strain was consistently recorded during Test 168WPL2.5 indicating that the initial soil stiffness was greater than that in Test 168WPL3.5. Table 7.2 summarises the main results at this step. It highlights the strong relationship between the magnitudes of VDS and pipe wall strain where significant deformations occurred, the largest wall strains corresponding to the largest diametral strains.

Raising the level of the sidefill to that of the pipe crown (Step 4) did not greatly alter the response of the pipes. This is shown by the internal strain distributions at Step 4 in Figure 7.2, which are broadly similar to those shown for the respective tests for Step 3 in Figure 7.1. The only significant change is the decrease in compressive strain at the pipe invert as the small soil load causes the pipe to bend and the pipe invert consequently to flatten. Broad agreement is also shown between the results for the two 168mm pipes, as it is for the 250mm pipes.

Thus comparing the strain gauge responses of the same type of pipe at a comparable stage in the installation procedure showed that they were broadly similar, indicating that the procedure was repeatable. The VDS and HDS data were more variable and less sensitive to subtle changes. Variations in the results illustrate that the initial stiffness of the sidefill varied between tests. These variations have been incorporated in the analysis conducted hereafter to aid the discussion of the influences on pipe performance.
7.1.2.2 Repeatability of WPL tests

In order to investigate further the repeatability of the buried pipe tests and allow the accuracy of the measuring techniques and the water bag arrangement to be assessed, Test 250WPL1 was repeated (Test 250WPL1*).

Figures 7.3 and 7.4 show the measurements of internal and external circumferential pipe wall strain respectively, at the pipe-glass interface for different levels of overburden stress. Figure 7.3 indicates that the pipe in Test 250WPL1* deformed elliptically, although only 4 circumferential gauges were available to determine shape, with a well defined trend of increasing compressive stress with load. Greater compressive strains were generally recorded in this test than in 250WPL1, although at the crown the latter test exhibited a consistent trend of increased compressive strains with load. This appears to have occurred in response to a greater flattening at the invert causing a relative increase in the tensile response at the haunches also in Test 250WPL1. The pipe can thus be said to have deformed to an inverted "Y" shape in response to the loading (see Figure 6.2). This in turn indicates that the haunch support was poorer in Test 250WPL1 than in 250WPL1*. Figure 7.4 shows the external circumferential strains, for the Right Hand Side (RHS) of the pipe for Test 250WPL1 and a comparison with 250WPL1* at the RHS springline (90°). Good correlation is shown at this location as the measured strains during both tests were tensile and below 200µε in magnitude. The strains for 250WPL1* were move tensile, which corresponds to the greater compressive strains recorded on the internal surface (Figure 7.3) and the elliptical nature of the deflection. The pattern of increased compressive (hoop) strain under increasing applied surface load is also replicated. The "Y" shaped deformation pattern for 250WPL1 is also well illustrated (being the inverse of that in Figures 6.2 and 7.3 since it is the external strains being presented in Figure 7.4).

In view of these results, reasonable agreement would therefore be expected between the imposed stresses inferred from strain measurements at the RHS springline during both tests. A comparison of the calculated (from measurements, see Section 5.3.2) and theoretical applied stress (see Section 6.1.3) at the springline locations is shown in Figure 7.5. It should be noted that the imposed stress was only calculated at the RHS
springline during Test 250WPL1, compared to both sides during 250WPL1*.

Comparing the RHS (90°) springline readings it is clear that the differences between the initial soil stiffness created during the installation procedure were largely removed during the first increment of surface stress of 50 kN/m². At the maximum surface stress of 150 kN/m² (Step 17) the difference between these two readings was 0.367 kN/m, which is within the precision of the measuring technique of Test 250WPL1 of ±0.4 kN/m. The imposed stress in Test 250WPL1* at the LHS (270°) springline was consistently greater than that at the RHS, the difference being 1.066 kN/m during the application of 150kPa surface stress. It would thus appear that, whilst the sidefill and backfill material were placed in as uniform manner as possible during these tests, the recorded asymmetrical loading during Test 250WPL1* was greater than the difference between the RHS springline in the repeat tests.

Using Step 4 as a datum to remove differences created during installation, the diametral strains for both tests during the application of overburden stresses have been calculated (Table 7.3). This shows that the difference between the two sets of HDS readings was typically within the precision range of the measuring technique of ±0.1%, whilst the VDS readings from Test 250WPL1* generally increases evenly as the load increases up to 100kPa, and that the final 50kPa increment produces a smaller additional deflection. The initial VDS due to installation is significantly different, but as the magnitude of the surface stress increases, the difference between the respective diametral strains reduces. This is due to both the differences in the initial sidefill stiffness between the tests being removed and the induction of larger diametral strains, thus diminishing the significance of errors in measurement.

The final comparison between these two tests is made on the basis of the total pipe and sand displacements due to the application of a surface stress of 150 kN/m² (i.e. the difference between Steps 5 and 17). Horizontal and vertical displacement contour plots are shown in Figures 7.6 and 7.7 for Test 250WPL1 and 7.8 and 7.9 for Test 250WPL1*, respectively. Table 7.3 shows a consistent pattern of lateral expansion of the pipe diameter, which as Figures 7.6 and 7.8 illustrate, was resisted by the sidefill causing horizontal soil displacements to extend from the pipe crown to the haunch.
positions and generally reaching a maximum adjacent to the pipe springline and shoulders. Whilst the magnitude of these displacements approximates to that of the precision of the measuring technique (±0.36mm), only narrow extents of movements are indicated as the displacements dissipate over an approximate lateral distance of 35 mm, or one seventh of the pipe diameter.

By contrast, more significant distributions of vertical displacements are seen in Figures 7.7 and 7.9, with increased magnitudes of displacements of up to 3mm. These two figures show similar magnitudes and distributions of displacements for the two tests, with the propagation of positive arching being manifested by the dissipation of displacements laterally from the pipe crown. The reduction in stress on the pipe is indicated at the pipe invert which is shown to settle less than the soil adjacent to it. It is noted that away from the influence of the pipe, the soil settled uniformly with depth. A discussion of the corresponding shear and volumetric strain plots is presented in Section 7.1.6.

7.1.3 Poorly-Placed Loose (PPL) installations

The poorly placed (PP) installation method was adopted for Tests 250PPLA1 and 250PPL1, both using a 250mm diameter PVC-U pipe and a cover to diameter ratio of 1. During Test 250PPLA1 the sidefill was placed asymmetrically, whilst during Test 250PPL1 it was placed uniformly either side of the pipe. The application of an asymmetrical soil load during installation was achieved by moving the funnel, connected to the sand conveyor belt, to an offset to the right of the centreline of the dispersal unit. By placing the pipe on a flat, compacted bedding, a bedding angle of 15° was created. This compares to an approximate bedding angle of 35° following Step 2 in the well-placed tests. For the PPL tests, the filling regime resulted in the creation of two prominent voids beneath the pipe haunches, the effects of which were manifested in the results obtained throughout both tests.

When filling the tank up to the pipe crown level (Step 4) the load imbalance in Test 250PPLA1 caused the pipe to be translated by 2.9mm to the left. This was associated with a reduction in the horizontal diameter by 1.40% and a corresponding increase in
the vertical diameter by 0.49%. The absence of Step 3 during this test means that the
development of any greater negative VDS during this filling process cannot be
determined, but it is evidenced by the development of an external tensile strain at the
pipe crown during Step 4 (see Figure 7.10). For this test only, the circumferential
strain distribution was defined using eight external, rather than internal, strain gauges.
To infer the deformed pipe shapes from Figure 6.2, therefore it is necessary to invert
the strain response. Figure 7.10 shows that an ellipse pattern, typical of a negative
VDS and shown at Steps 3 and 4 for the WPL tests (Figures 7.1 and 7.2 respectively),
did not occur as a result of the small soil loading. Instead, the strain distribution shows
that during Step 4, the left-hand side (LHS) of the pipe (180° to 360°) deformed to an
approximately "square" shape, whereas the RHS (0° to 180°) deformed to an inverted
"Y" shape. This contrast can be explained by the relative density, and hence the passive
resistance, of the sidefill. The overall displacement of the pipe to the left resulted in the
material adjacent to the RHS of the pipe achieving a looser state compared with that
on the LHS, since the pipe displaced under Rankine's active earth pressure condition.
The more uniformly placed material to the (now more restrained) LHS of the pipe
resulted in a denser, stiffer sidefill and the pipe subsequently deformed to a "square"
shape.

In comparison to the pipe translation of 2.9mm when installing Test 250PPLA1, the
more uniform nature of the filling process in Test 250PPL1 resulted in a translation of
the pipe to the left by only 0.39mm. In addition, the magnitudes of the HDS and VDS
were considerably smaller at 0.31 and -0.09%, compared to 1.40 and -0.49% for
250PPLA1. The even nature of the filling operation also led to the observation of an
inverted 'Y' response on both sides of the pipe due to the creation of a negative VDS
during Step 3 and the deformation of the pipe at the haunch positions under the
application of the small soil load.

During both tests, the application of soil and surface overburden stresses (Steps 4 to
17) was only resisted vertically by the bedding at the pipe invert. This pivotal action,
coupled with a lack of support beneath the haunches, resulted in the creation of large
internal circumferential tensile strains at the invert (180°). During Test 250PPL1 an
absolute tensile strain of $2820\mu e$ was recorded at the pipe invert under the maximum surface stress (150 kPa, Step 17), which represented an increase of $2369\mu e$ with respect to 250mm placed soil cover (Step 5). These values represented the largest absolute, and change, of strain recorded during the buried pipe test programme. The resulting pipe and soil displacements are discussed in Section 7.1.5 where they are also compared to tests in which material was hand placed beneath the pipe haunch (i.e. WP tests).

7.1.4 Well Placed Dense (WPD) Installations

7.1.4.1 Pipe Installation (Steps 1 to 4)

The WPD installation procedure was used for Tests 250WPD1 and 250WPD1*.

During 250WPD1 compaction of the sidefill during Steps 3 and 4 caused the pipe (which was relatively light) to be raised completely from the bedding material. Consequently a repeat test (250WPD1*) was conducted in which a 4kg steel bar was placed inside the pipe, which kept the pipe invert in contact with the underlying bedding.

During Test 250WPD1, sand was carefully placed beneath the pipe haunches prior to compaction of the surround in layers. The displacement vectors in Figure 7.11 show, however, that compaction of the sidefill material at the springline level (Step 3) caused the whole pipe to displace upwards by 6mm, thus creating a void beneath the pipe invert. A reduction in the horizontal diameter by 0.28% causing a negative VDS of 0.26% occurred due to the compaction at this point. This pipe response is clearly seen in Figure 7.12, which shows the development of low internal compressive strains at the pipe springline (90° and 270°), associated with larger compressive strains at both the pipe crown and invert locations (0° and 180°) after Step 3.

A second important feature of Figure 7.11 is that the pipe is shown to have translated to the left by 0.90mm. This was due to the application of compactive effort to the sidefill adjacent to the RHS of the pipe before it was applied to that on the LHS. The pipe thus displaced laterally into the looser material on the LHS. This situation was not remedied by the subsequent compaction to the LHS, because the sidefill at the RHS
had become stiffer than when the sand was first placed. This argument is similar to that due to asymmetrical filling described in Section 7.1.3 for Test 250PPLA1.

Placing of the sand from the springline to the crown and its subsequent compaction, again initially to the sidefill adjacent to the RHS of the pipe (Step 4), resulted in a continuation of the behaviour seen at Step 3. The pipe displacements (Figure 7.13) show that the longitudinal axis of the pipe displaced upwards by 0.6mm and to the left by 0.8mm during Step 4. Although the applied compactive effort was the same for both steps, the translation of the pipe during Step 4 was less than that observed during Step 3 due to the presence of compacted material beneath the springline and the application of a small (confining) soil load. During Step 4 the increased lateral earth pressure acting on the pipe due to the compaction caused a further increase in the negative VDS (to 1.76%) as the horizontal diameter decreased and the vertical diameter consequently increased, by 4 and 5mm respectively. This was associated with elliptical deformation about the vertical axis (i.e. the inverse of Figure 6.2a, see Figure 7.12), which shows internal compressive strains at both the pipe crown and invert (0° and 180°) and corresponding internal tensile strains at the pipe springline (90° and 270°).

Weighting the pipe invert during Test 250WPD1 caused the invert of the pipe to remain in contact with the underlying sand bedding, thereby eliminating void formation. Raising the level of the sidefill in 50mm thick increments resulted in a consistent reduction of the horizontal diameter of the pipe and a corresponding increase in the vertical diameter as each 50mm layer was installed. Figure 7.14a shows an interesting progression of internal circumferential strains during installation. For example, at the pipe crown and invert (0° and 180°) the largest increases in compressive strain were recorded when the elevation of the sidefill was below that of the pipe springline (i.e. the radius of curvature reduced worst at this point). In contrast, the largest changes in strain at the springlines (90° and 270°) occurred as the sidefill was raised from a level 25mm below, to 25mm above it. This corresponded also to the largest changes in the values of diametral strain, as the imposed lateral earth pressure beside the pipe reached a maximum whilst the pipe was resisted vertically by a
negligible soil load (i.e. was largely unresisted vertically). On completion of sidefill placement (i.e. Step 4) the pipe had deformed elliptically about the vertical axis, as with 250WPD1 but with an almost perfect correlation to the expected pattern. The effect of keeping the pipe invert in contact with the bedding was a greater VDS (-3.44%) than that for 250WPD1 (-1.76%). A VDS of 3.44% was the largest magnitude of diametral strain recorded during the buried pipe test programme.

7.1.4.2 Pipe loading (Steps 5 to 17)
Application of the sand overburden stress (Step 5) followed by the surcharge loading (Steps 6 to 17) resulted in the pipe in Test 250WPD1 being forced into the void beneath its base. This is illustrated by the internal circumferential strain distribution (Figure 7.12) where a large compressive strain of -2400με was recorded at the pipe invert. Due to the application of overburden soil stress (Step 5) the pipe translated downwards by 1mm and derived support (i.e. resistance to movement) from the surround material, particularly at the haunches. This can be seen from the tensile strains induced at the haunches (135° and 225° in Figure 7.12) modifying the elliptical response of a negative VDS at Step 4 combined with only a small increase in compression at the invert (180°). The bending action facilitated by this support is shown by the increase in the horizontal diameter (-0.16%) whilst the deformation of the pipe at the invert into the void beneath resulted in a continued increase in the vertical diameter (-0.18%). It should be noted here that interpretation of behaviour solely by VDS and HDS measurements would prove very difficult, whereas strain gauge readings provide a clear indication of structural response to installation and loading.

By creating an initial imbalance in the sidefill stiffness during Steps 3 and 4, the application of the initial increment of 50kPa surface stress (Step 6) resulted in smaller tensile strains (hence larger lateral pipe displacements) recorded at the RHS (90°) springline as it deformed into the looser soil. As with Test 250PPLA1 discussed previously, this imbalance was removed under the application of higher magnitudes of static surface stresses since approximate strains were recorded at each springline. These observations are supported by the contour plots of vertical displacement, shown
in Figures 7.15 and 7.16 for the application of a static surface stress of 50 and 100 kPa, Steps 6 and 15, respectively. An asymmetric settlement pattern is shown in Figure 7.15 at Step 6 by the horizontal nature of 1 mm settlement contour emanating from the RHS pipe shoulder into the looser sidefill material, compared to the more vertical nature of the same contour from the LHS shoulder. Increasing the surface stress to 100 kPa (Figure 7.16, Step 15) resulted in a more symmetrical settlement pattern.

The effect of avoiding voidage beneath the pipe invert and any significant difference in sidefill stiffness (due to reduced sidefill thickness) in Test 250WPD1*, thereby creating a more competent surround, is clearly demonstrated by the response of the pipe-soil system to the applied overburden stresses. Figure 7.14b shows the internal circumferential pipe wall strains during Test 250WPD1* indicating a highly stable pipe into which solely hoop compression is induced when the applied stresses increase. Figure 7.17 shows the imposed stresses calculated for these two tests and indicates that from similar readings at full sidefill (Step 4) the stress increase was greater in 250WPD1 than 250WPD1*. This distinction is clearly emphasised by the error bands associated with the respective readings.

7.1.5 Effect of installation on the response to applied stresses

7.1.5.1 Effect of soil overburden

On completion of the installation procedure (Step 4) the pipe was covered by soil overburden of varying thickness (Step 5) which was then subjected to applied surface stresses (Steps 6 to 17). This section aims to examine the response of the pipes in all of the tests to the overburden soil stress, whereas Section 7.1.5.2 examines the response to the surface surcharge stresses. To assess how the imposed load on the buried pipes varied between tests, the data for Step 5 have been plotted for all tests except 250WPD1 (in which the pipe invert raised from the bedding) with respect to the reading at Step 4, or ‘full sidefill’ reading (Figure 7.18). It should be noted that the changes in stress (thus load) are small, and in some cases are within the range of acceptable precision of the readings (and therefore possibly insignificant), but the patterns achieved are consistent.
Comparing the results of Tests 250WPL1*, 250WPL1 and 250WPL2 shows that all the responses were linear and similar in magnitude after the application of 250mm of sand. On the basis of both springline readings, the imposed load in Test 250WPL1* exceeded that in Test 250WPL2 (by 0.09kN/m) indicating initially lower sidefill stiffness in Test 250WPL1*. The greater load recorded during Test 250WPL1* compared to 250WPL1 was as Section 7.2.1.1 described previously, due to the inferring of load from a single strain measurement at the RHS (90°) springline (rather than from diametrically opposite measurements as in Test 250WPL1*) in Test 250WPL1. This is highlighted by the greater difference of load between the two springline readings during Test 250WPL1* of 0.071kN/m, compared to that between the two tests from the RHS springline readings of 0.046kN/m, thus indicating similar initial sidefill stiffness during Tests 250WPL1* and 250WPL1.

Figure 7.18 shows that for a similar application of 250mm of sand the increase in imposed load in Tests 250PPL1 and 250WPDL1 was significantly lower than those for Tests 250WPL1*, 250WPL1 and 250WPL2. As Section 7.1.5.2 describes in more detail, the lack of material beneath the pipe haunches in Test 250PPL1 and the increased initial sidefill stiffness due to compaction in Test 250WPDL1 resulted in an increased imbalance of stiffness between the pipe and the surround leading to greater applied load being transferred to the relatively stiffer surround.

Increasing the stiffness of the PVC-U pipe by using a 168mm diameter pipe (SDR = 48), would be expected to lead to a reduction in the degree of positive arching compared to of the more flexible 250mm diameter pipe (SDR = 93). This would correspond to a relationship between imposed and applied load closer to that of the line of equality. Figure 7.18 shows that the results of Tests 168WPL2.5 and 168WPL3.5 provide conflicting evidence of this, with the gradient of the line in Test 168WPL3.5 exceeding those of the 250mm pipe tests, whilst the opposite trend is indicated for Test 168WPL2.5. This difference in response indicates a variation in the installation procedure between the tests, which was highlighted in Section 7.1.2, with the surround in Test 168WPL2.5 being initially more competent than that in Test 168WPL3.5. As the height of fill is increased, the pipe in Test 168WPL2.5 takes an
increasing proportion of the applied load such that for the last increment the trend is
towards the line of equality. In contrast in Test 168WPL3.5 there is an initially linear
degree of loading that is greater than the soil 'prism' overburden. Above a height of
340mm (or 2D), however, a non-linear relationship is shown with the degree of
arching increasing rapidly as the stiffness of the sidefill increases and the line of
equality again being approached. The implication of these results is that the increasing
self-weight of the soil appears to begin to remove the initial imbalance of the soil
stiffness created during the installation as the response of the pipes converge.

The circumferential pipe wall strain response of the clay pipe during both Tests
120WPL1 and 120WPL2.5 was generally below the level of the precision of the
measuring technique during installation but was highly variable during loading. No
conclusions could therefore be drawn from the strain gauge measurements and, as a
consequence the imposed stress measurements.

7.1.5.2 Effect of the static surface stress
To assess the influence of the installation procedure under the application of static
surface stresses the results have been separated to discuss (in order) the imposed load,
diametral strains, settlement profiles and soil strain measurements. With all other
factors being equal, a pipe placed in a WPD surround would be expected to experience
a reduced proportion of the applied stress than in a WPL or PPL surround. Due to the
formation of a large void beneath the pipe invert during the installation of Test
250WPD1, however, the results for this test were adversely affected and thus only the
results from Test 250WPD1\ will be considered in this discussion. Similarly, the results
from Test 250WPL1* will be used since it was possible to calculate the imposed load
at both springline locations rather than at one springline position, as in 250WPL1. The
results from Test 250PPL1 are compared with Tests 250WPD1\ and 250WPL1* as
the sidefill was placed in a similar, uniform manner yet without careful haunch
distribution. The imposed load and diametral strain results from Test 250WPL2 have
also been included to show the effect of increasing the cover to diameter ratio from 1
to 2 and have been discussed in Section 7.3.
Figure 7.19 shows the average imposed loads against applied load for all four tests, using Step 4 readings as the datum. The importance of compacting the sidefill can be clearly seen as the imposed load in Test 250WPDI is consistently below that in Test 250WPL1*. This corresponds to a greater degree of arching being shed to the sidefill in the compacted surround. In addition, these results indicate that the imposed load on the pipe in Test 250PPL1 was nearly identical to that in Test 250WPDI1. The effect of not specifically placing material beneath the pipe haunches is seen effectively to reduce the stiffness of the pipe, making the pipe more easily able to deflect and thereby to initiate arching. Whilst this method of installation reduces the imposed load on the pipe detrimental effects are, however, observed at the pipe invert. For example, the measured internal tensile circumferential strain at the pipe invert with respect to the same datum was 2369με, whereas those for Tests 250WPDI1 and 250WPL1* were -130 and -633με respectively.

Figure 7.20 shows the values of VDS and HDS during the application of static surface stresses for the same four tests. For the three tests placed at a C/D ratio of unity, the largest values of VDS were recorded during Test 250PPL1 at every stage. It would appear that the lack of support beneath the pipe haunches in the PPL surround reduced the load capacity of the pipe resulting in the largest pipe deflections, although only a small amount compared with the WPL surround. In contrast, the smallest increase of VDS occurred in the most competent surround. A similar, though less consistent, pattern of behaviour for the three tests with a C/D ratio of unity was observed from the HDS readings. Both sets of readings show that the increase of diametral strain was not linear with applied load. This non-linear relationship of diametral strain against applied stress was as expected, because as the pipe deforms the applied stress is shed to the sidefill material, increasing its stiffness further, thereby limiting the potential for increased lateral expansion of the pipe under subsequent applied stresses.

Combining the results of imposed load and diametral strain, it is seen that the WPD surround performs best. The observation of least VDS with maximum positive arching demonstrates that a greater proportion of the applied stress is transferred to the stiffer sidefill, which, as it becomes even stiffer, reduces the potential for HDS and hence
observed VDS. This is highlighted by comparing the measured soil settlement profiles under maximum surface stress in Figures 7.21a and 7.21b for the crown and invert, respectively. The displacement at the pipe crown was smaller for the WPD surround (-1.8 mm) than either 250WPL1 test (-2.8 and -2.0 mm), which were in turn smaller than the PPL surrounds (-2.8 and -3.6 mm). The effect of compacting the sidefill is also shown at an offset of 200 mm from the centreline, where the settlements are smaller (-1.5 mm) than those of either 250WPL1 test (-2.5 mm). Similarly, at the pipe invert displacements of -1.1 mm were recorded in the WPD test compared to -1.4 mm during either 250WPL1 (Figure 7.21b), illustrating that less load was imposed on the WPD pipe. Positive arching patterns are shown in these three tests as the magnitude of the settlements dissipate laterally from the pipe crown but increase slightly from the pipe invert. The PPL test data were more erratic, particularly at the invert, but again broadly conformed to the positive arching pattern.

Examining the profiles in more detail, it is apparent that the behaviour outlined above for the well-placed surrounds contrasts with that for the two PPL tests. The pivotal action of the pipe invert to the applied overburden stresses is shown in Figure 7.21b by increased displacements beneath the haunches, the flattening of the pipe invert being evident from the differences in displacement between the invert and haunches. This behaviour is also manifested at the pipe crown, which is shown to displace more than those of comparable tests in which support beneath the pipe haunches was provided. At offsets greater than 100 mm from the pipe centreline similar soil displacements were recorded in PPL and WPL surrounds. This distance is equivalent to half the pipe diameter, and illustrates the localised influence that a change in the installation has on behaviour of the surrounding soil.

Comparison of shear and volumetric soil strain is based upon the vertical settlement contours under the maximum surface stress (150 kPa, Step 17) using a datum of full cover (Step 5). The shear strain plots for Tests 250WPD1*, 250WPL1* and 250PPL1 are shown in Figures 7.22, 7.23 and 7.24, respectively. Using the convention described in Section 6.1.8, positive arching strains are indicated by positive strains on the LHS of the central axis of the pipe and by negative strains on the RHS. Negative arching
strains are therefore indicated by the reverse of this notation. To compare these plots to the shape of pipe deformation shown in Figure 6.2, Figure 7.25 shows the internal circumferential pipe wall strain distribution for Tests 250WPD1, 250WPL1 (i.e. not 250WPL1* due to the use of only four strain gauges at each section; thus assuming similarity of behaviour between the two tests as shown in Section 7.1.2), 250WPL2 and 250PPL1.

Examination of each of the shear strain plots shows that an approximately vertical 0% shear strain line extends from the pipe crown to the water bag, separating the formation of positive arching strains on either side of the pipe. The occurrence of a height of equal settlement during these tests would be shown by a prominent horizontal 0% shear strain line but this is absent in all cases. Positive arching strains are shown originating from the pipe between the shoulders and haunches, extending into the adjacent sidefill. These 'ear' shaped distributions are similar to those that have been observed above tunnels constructed in granular material both from field and laboratory installations (see Figures 3.61 and 3.73 respectively). Due to the greater displacement of the soil laterally away from the pipe invert than at the invert, negative arching strains are observed extending from the pipe haunches. The regions of positive and negative arching strains are shown to be delimited by a 0% line extending at approximately 45° to the horizontal from the pipe haunches. The complete distribution for this typical pattern corresponds to the shear strain pattern shown in Figure 6.26a for Test 250WPD1 (Figure 7.22) and is associated with a pipe deforming elliptically (i.e. positive VDS). The associated settlement distribution is shown in Figure 6.16b for WP, flexible pipe surrounds under static stress. Variations to this basic shear strain distribution due to changes in the installation procedure are also shown in Figure 6.26.

The most clearly defined shear strain pattern is shown in the PPL surround in Figure 7.24. Due to the absence of support beneath the pipe haunches the pipe was observed to flatten at the invert and thus settle at the crown (Figures 7.21a and b) resulting in a column of soil directly above the crown settling markedly with respect to the sidefill. This is clearly highlighted by the -2% shear strain contour extending from the RHS shoulder to a distance 100mm above the pipe crown. The shape of this distribution is
strains are therefore indicated by the reverse of this notation. To compare these plots to the shape of pipe deformation shown in Figure 6.2, Figure 7.25 shows the internal circumferential pipe wall strain distribution for Tests 250WPD1, 250WPL1 (i.e. not 250WPL1* due to the use of only four strain gauges at each section, thus assuming similarity of behaviour between the two tests as shown in Section 7.1.2), 250WPL2 and 250PPL1.

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more vertical in nature than the ‘ear’ shape distribution associated with a pipe deforming elliptically. A similar, though discontinuous, area of 2% shear strain occurs in the WPL surround in Figure 7.23 for Test 250WPL1*, indicating a less well defined zone of shear. Figure 7.24 (for 250PPL1) also shows that these positive arching strains extended around the circumference of the pipe wall down to between the haunch and invert locations. A maximum shear strain of -9% was recorded at the RHS pipe haunch as the pipe wall deformed into the void beneath. The lateral extent of the shear planes is also shown to be the greatest in the PPL surround. Figure 7.25 shows that an inverted ‘heart’ shape distribution occurred in this test (see Figure 6.2c) and thus the associated shear strain pattern in Figure 7.24 has been assumed to correspond to this shape of pipe deformation and is shown in Figure 6.26c.

In comparison with Figure 7.24, the shear strains shown for the WPL installation condition in Figure 7.23 are smaller and less clearly defined. Discrete regions of 2% shear strains are shown above the pipe shoulders bounded by 1% shear strain lines extending from the pipe crown and shoulders. Similar magnitudes of shear strain are recorded in the sidefill zones either side of the pipe. The positive arching strains are shown to occur around the circumference of the pipe above the pipe haunches. Below the RHS haunch, a region of negative arching strains is observed extending into the sidefill. A similar pattern is also shown on the RHS of the pipe for the WPL surround in Test 250WPL1 (see Figure 7.44), which as Figure 7.25 shows deformed approximately to an elliptical pattern. This pattern of shear strain distribution thereby agrees with that shown in Figure 6.26a. Thus it is clear that the magnitude and extent of the positive arching shear strains were greater in the PPL than the WPL installation, implying greater positive arching stresses would correspondingly be induced. For the same magnitude of applied stress this would result in a reduction of imposed stress on the pipe, which is shown to occur in Figure 7.19.

The smallest degree of shear strain of these three types of plots was observed in the WPD surround (Figure 7.22). The prominent zones of positive arching strains occurring beneath the springlines are generally smaller and the 1% shear strain contours, which extend to the surface in Figures 7.23 and 7.24 are of more limited

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extent. Significant differences occur between the LHS and RHS in Figure 7.22 in terms of negative arching strains, the pattern at the RHS corresponding to the elliptical deformation patterns shown in both Figures 7.25 and 6.25a.

Whilst the extent and magnitude of shear strains during this WPD test were less than those observed with loosely compacted surrounds, Figure 7.19 shows that the imposed stress on the pipe was comparable to that in Test 250PPL1 and was significantly lower than that in 250WPL1*. Applying compaction to the sidefill increases the initial density of the surround and causes greater interlock between the sand particles. For the same value of shear strain, greater shear stresses are required to overcome the resistance to shear in a densely compacted, compared to loosely compacted, sand. The increased initial density of the sidefill created by compaction therefore induced greater arching stresses for reduced magnitudes and extents of shear strain in the WPD surround.

The volumetric strain contours for Tests 250WPD1, 250WPL1* and 250PPL1 are shown in Figures 7.26, 7.27 and 7.28, respectively. Using the convention of negative for compressive strain, these figures show in general that the largest compression occurred close to the pipe wall. Due to the greater stiffness of the sidefill compared to the pipe, these strains are shown generally to dissipate in magnitude away from the pipe. In the WPD and WPL tests the pattern of volumetric strain in the sidefill and bedding are shown to be more consistent. Large compressive strains are shown beneath the haunches in the PPL surround (Figure 7.28), reaching a maximum of -12 and -22% at the LHS and RHS respectively, and greatly exceeding typical values of up to -4% in the WPL and -2% in the WPD surrounds.

In each of the figures a continuous 0% volumetric strain contour is shown in the sand layer overlying the pipes, which separates the compressive region within the sidefill from the less compressive, and sometimes dilational, zone above the pipe crown. This was most evident in the PPL surround (Figure 7.28) where the 0% volumetric strain contour is shown as a circle of slightly smaller diameter than that of the pipe, centred directly above it and extending approximately from the pipe crown. Dilation occurs within this zone induced by the near vertical shear strain band of approximately 2% on
either side of it (Figure 7.24). A similar, though less well defined, pattern occurs to a lesser extent in the WPL test (Figure 7.27), whereas the zone of dilation is confined to the area immediately above the pipe in the WPD surround (Figure 7.26). The volumetric strain patterns close to the surface are affected by the action of the water bag, as described in Section 5.2.2, and are thus not significant to the pipe-soil system response.

7.2 EFFECT OF PIPE STIFFNESS

It would be expected that with all other factors being equal an increase in pipe stiffness would result in an increase in imposed stress on the pipe. In addition, as the stiffness of the pipe exceeds that of the sidefill negative arching strains would be expected to be induced. To assess this, the results from Tests 250WPL2 (PVC-U), 168WPL2.5 (PVC-U) and 120WPL2.5 (Vitrified clay) will be used. Only the results up to a surface stress of 70kPa are used from these tests as cyclic loading occurred thereafter in Tests 168WPL2.5 and 120WPL2.5 (see Section 7.5). In view of the difficulties experienced with measuring pipe wall strains, and hence inferring imposed stresses on clay pipes, only the imposed stresses for the two PVC-U pipe tests are compared. Shear and volumetric strain contours are then compared for each of these three tests under the application of a static surface stress of 70 kPa.

Due to the difference in the diameter of the pipes it is necessary to compare the imposed load in terms of degree of loading. This comparison is based upon a datum of Step 4 and the response to surface stresses of 50 and/or 70 kPa, at which level the relationship between applied (theoretical) and imposed load (inferred from strain gauge data) is essentially linear. The shear strain plots (discussed later) indicate that this is because the soil is still within the elastic range. The average imposed load from springline measurements in Test 250WPL2 was -3.117kN/m at Step 7, compared to -0.555 kN/m at the datum (Step 4). The applied load at each springline was -6.191kN/m, corresponding to a degree of loading of 41%. Following the same procedure, the results for Test 168WPL2.5 were 55 and 59% at Steps 6 and 7 respectively. These results show, as expected, that with all other factors being equal,
increasing the pipe stiffness increases the imposed stress on the pipe, which would be expected to result in a reduction in the level of positive arching strains.

The shear strain distributions for the 70kPa static surface stress in Tests 250WPL2, 168WPL2.5 and 120WPL2.5 are shown in Figures 7.29, 7.30 and 7.31 respectively. These will be examined to determine whether the arching strains become more negative in nature as the stiffness of the pipe increases, as expected. The occurrence of positive arching in Test 250WPL2 (Figure 7.29) is shown by positive arching strains extending upwards and outwards from the pipe shoulder. In addition, a prominent zone of negative arching strains was recorded below the pipe haunch. Similar behaviour, although not quite so distinct, is shown in Figure 7.30 for the stiffer 168mm pipe. A plane of equal settlement, indicated by a continuous horizontal line of 0% shear strain, is observed to exist at a distance of between 50 and 200mm above the pipe crown. Since the distribution is less prominent and the magnitude of positive arching strains is lower, the effect of increasing the pipe stiffness was observed to be as expected.

Figure 7.31 shows the magnitude of the shear strains in the soil surrounding the clay pipe were typically less than 2%, but, unlike those in Figure 7.29, no continuous areas of strains of this magnitude are shown. It would appear, therefore, that a combination of the relative equality between the stiffness of the rigid pipe and loose surround at this level of static surface stress (equivalent to approximately 5.0m of soil) resulted in the promotion of limited magnitude and distribution of shear strain. It will be later demonstrated (Section 7.5), however, that under the application of repeated uniform surface stress negative arching strains of up to 10% were recorded during this test.

7.3 INFLUENCE OF COVER DEPTH
The effect of increasing the soil cover depth was shown by the downward yielding trapdoor experiments to decrease the level of imposed stress on a buried structure at the same level of applied total stress (i.e. with the addition of surface stress). This is because the vertical extent of the positive arching stresses induced in the overlying soil are not restricted by the presence of the ground surface. Terzaghi’s (1936) trapdoor experiments showed that for a yielding soil mass a C/D ratio greater than 2.5 is
required before the influence of the cover depth, and thus the ground surface, diminishes. Getzler et al (1968) showed that by inducing arching stresses by the application of a surface stress to a buried structure-soil system of varying relative stiffness, rather than the trapdoor, under the same magnitude of surface stress the degree of arching increased with cover depth, reaching an approximately constant value as the C/D ratio exceeded unity. As Hoeg (1968) and Pender (1981) state, amongst others, a C/D ratio of unity is usually quoted as a minimum (for an elastic soil) before the boundary conditions influence the behaviour around a buried structure.

The influence of cover depth on pipe response to static surface stresses is illustrated in Tests 250WPL1* and 250WPL2 placed at soil cover/diameter ratios of 1 and 2 respectively. As Section 7.1.5.2 previously described, the results from Test 250WPL1* will be used since the imposed load was calculated at both springlines, rather than at the RHS springline as in Test 250WPL1, and as Section 7.1.2 also described, due to the similarity between the results of these two tests based upon common strain gauge data. Figure 7.19 shows that near identical average imposed loads were recorded during Tests 250WPL1* and 250WPL2. Using the full cover depth (Step 5) as datum readings to assess the influence of cover depth, at the maximum surface stress (Step 17, 150kPa) the degree of loading was 36 and 39% for Tests 250WPL1* and 250WPL2 respectively. Whilst these results disagree with the expected pattern shown in Figures 2.11 and 3.25, the closeness of the results support the finding of Getzler et al (1968) and the assumption of Hoeg (1968) and Pender (1980) that the degree of arching reaches an approximately constant value as the C/D ratio exceeds unity. This conclusion is also supported by the recording of VDS during Test 250WPL2 only marginally exceeding those in Test 250WPL1* (Figure 7.20). Due to the measurement of pipe wall displacement at the RHS springline only during Test 250WPL2, the precision of the resulting HDS measurements are correspondingly reduced which is highlighted by the more erratic relationship in Figure 7.20 compared to that recorded from both springlines in Test 250WPL1*.

Increasing the C/D ratio to 2.5 and 3.5 in Tests 168WPL2.5 and 168WPL3.5 led to a similar pattern of behaviour to that between Tests 250WPL1* and 250WPL2. Figure
7.36 shows that the imposed load on the deeper pipe (Test 168WPL3.5) only marginally exceeded that on the shallower pipe (168WPL2.5) during the application of 70kPa static surface stress (Step 7), resulting in a degree of loading of 64 and 58% respectively based on full soil cover (Step 5) as datum readings. Figure 7.37 shows that prior to cyclic loading (i.e. Step 7), the VDS readings were identical (being 0.31 and 0.32% for Tests 168WPL2.5 and 168WPL3.5 respectively), whilst deviation between the HDS readings only marginally exceeded the precision level of 0.124% (being -0.14 and 0.32% for Tests 168WPL2.5 and 168WPL3.5 respectively. The similarity in behaviour is further supported by the approximately equal settlements at the level of the pipe invert in the two 250mm pipe tests (Figure 7.32) and in the 168mm pipe tests (Figure 7.51b).). This similarity of behaviour between these two sets of 250 and 168mm pipe tests would appear therefore to support those assumptions made by Pender and Hoeg. This in turn implies that the soil behaved in an elastic manner.

Discussion regarding the variation of shear strain distribution with increased cover depth is limited due to greater clarity to those from the 250, compared to 168mm, WPL pipe tests. Examining the shear strain distributions, Figure 7.23 shows the contours for Test 250WPL1* extending to the level of the water bag under the application of a static uniform surface stress of 150 kPa. Comparison with those for a similar installation with a greater cover depth, and under an increasing vertical stress, would thus help to elucidate better the soil response. Figures 7.29 and 7.33 show the shear strain distributions for Test 250WPL2 (i.e. with twice the cover depth) at uniform surface stresses of 70 and 150 kPa respectively, whilst Figure 7.34 shows the volumetric strain distribution at 150kPa. The shear strain distributions indicate positive arching, shown by the negative values, extending from the pipe springline and shoulder into the sidefill and backfill zones. Below the pipe haunch, negative arching strains were evidenced by the positive strains. The positive arching strains under 70kPa (Figure 7.29) extend vertically above the pipe crown by 120 mm, or 0.5D, and laterally by approximately 70 mm, or 0.28D. The extent of this zone is comparable to that for Test 250WPL1 under 50 and 100kPa (discussed later, see Figures 7.42 and 7.43).
For simplicity, the shear strain distribution at a surface stress of 150 kPa (Figure 7.33) has been separated into 4 areas (denoted as areas 1-4). Area 1 shows a region of positive arching strains (indicated by the negative strains) extending from the crown and shoulder to a radial distance of 100mm. Area 3 is shown directly above area 1 and shows a region of negative arching strains (indicated by positive strains). Area 2 is shown to extend laterally from both areas 1 and 3 and extend above area 3 and below area 1. The boundary of area 2 is therefore indicated by a zero contour extending 100mm laterally from the pipe springline and extending vertically to a distance of 250mm above the pipe crown (or 200mm from the water bag) thereby delimiting the vertical extent of area 3. Area 4 shows a region of negative arching strains (indicated by the positive strains) extending below into the bedding from the pipe haunch and invert, and laterally to area 2 by a zero contour extending at angle of approximately 45° from the pipe haunch.

Increasing the surface stress to 150 kPa (Figure 7.33) increases the concentration of discrete areas of positive arching strains of -2% shear strain above and to the side of the pipe shoulder (denoted as area 1). In comparison to Test 250WPL1* (Figure 7.23) this zone appears more vertical in nature as the crown settles with greater respect to the relatively stationary pipe springline. This observation agrees with the response of the internal circumferential strain distribution in Figure 7.25 under a surface stress of 150 kPa of a 'heart' shaped distribution for Test 250WPL2 compared to elliptical pipe deformation for RHS of Test 250WPL1 and in Figure 6.6 for 250WPL1*. The relative differences between these two types of deformation is that the 'heart' shaped response indicates improved support beneath the pipe springline whilst the elliptical response indicates poorer support at the pipe springline. The improved support at the pipe springline in Figure 7.33 for Test 250WPL2 is shown by the absence of significant shear strains extending around the pipe circumference to the position of the pipe haunch. For this reason, Figure 6.26b shows the general shear strain distribution determined from Figure 7.33 associated with a 'heart' shaped pipe deformation pattern.
Above and to the right of area 1 (i.e. area 2), isolated areas of positive arching strains are recorded by the -2% shear strains which form a narrow ‘ear’ shape at an angle of 45° from above the pipe shoulder to a height of approximately 200mm below the water bag. This alternative position of the vertical extent of the shear strains corresponds to the elevation shown in the volumetric strain distribution (Figure 7.34) as a near continuous horizontal 0% boundary line extends across the width of the measured area, 200 mm below the water bag. As the vertical displacement contour plot in Figure 7.35 show, this elevation is associated with a plane of equal settlement of -3 mm. Figure 7.34 also shows that directly above the pipe crown an area of dilation is observed extending laterally to approximately the edge of the pipe and vertically to the plane of equal settlement. A similar area of dilating soil was also observed during Test 250PPLA1 (Figure 7.24) and was associated by the formation of shear strains bounding the sides of this area. Comparing the behaviour of Tests 250WPL2 and 250WPL1* it would appear, therefore, that increasing the depth of cover led to the formation of a more vertical concentration of positive (-2%) shear strains adjacent to the pipe shoulder and that a clear height of equal settlement was formed with the deeper installation.

7.4 EFFECT OF STATIC SURFACE STRESSES

The next two sections consider the effect of the surface stress regime on the behaviour of the buried pipes. This section describes the effects of increasing the magnitude of the uniform static surface stress, whilst Section 7.5 describes the effect of repeated application of uniform surface stresses.

7.4.1 Imposed load

Figure 7.19 shows that the imposed load on the 250mm diameter PVC-U pipes in PPL, WPL and WPD surrounds increases approximately linearly with the applied surface stress up to the maximum value of 150kPa, or an equivalent C/D ratio of approximately 27. For the 168mm diameter PVC-U pipes the maximum equivalent C/D ratio was 41. Figure 7.36 shows that for the 168mm pipe tests that prior to the application of cyclic loading, equivalent to a C/D ratio of approximately 20, increases of static surface stress resulted in a linear increase of imposed stress. However, after
the application of repeated (or cyclic) loading, the application of higher equivalent soil loading resulted in smaller increases of imposed load (i.e. the gradient of the line in Figure 7.36 decreases) and thus a reduction in the degree of loading on the pipe. The repeated loading thus caused the sidefill to stiffen and thus a greater degree of arching to occur under load.

7.4.2 Diametral strain
In contrast to the typically linear increase of imposed load, Figure 7.20 indicates that for the 250mm diameter PVC-U pipe tests the increase of VDS exponentially decreases with increasing applied surface stress in the loose surrounds. The HDS readings were, however, more variable, although those for the WPD installation similarly exhibited the increase of HDS decreasing exponentially. A similar, though less distinct trend of increasing diametral strain decreasing exponentially is also shown in Figure 7.37 for the 168mm diameter PVC-U pipe tests in WPL surrounds. As implied by Figure 7.36, the effect of the repeat loading was to reduce the rate by which the diametral strain increased (in the case of 168WPL3.5 to nearly zero) relative to that prior to cyclic loading.

7.4.3 Hoop compression and distortional bending
The results outlined above indicated that during the PVC-U pipe tests the imposed load increased linearly, whilst the increase of diametral strain decreased exponentially with increases in applied static surface stress. These trends are consistent with the field trial described by Spannagel et al (1974) and Adams et al (1989). During these trials it was shown that as the stiffness of the sidefill increased, the proportion of the transfer of applied stress away from the pipe increased also through hoop compression as the influence of distortional bending diminished. To investigate this further the results of the internal pipe wall circumferential strains reported herein have been used to calculate the hoop compression and distortional effects throughout each test. As problems were experienced with the RHS haunch gauge (135°) during Tests 250WPD1^ and 168WPL2.5 the observation of approximate symmetry led to the adoption of the LHS haunch position (225°) readings being substituted.
Figure 7.38 shows that the hoop compression increased linearly with applied stress during each PVC-U pipe test where measurements were recorded from eight, evenly spaced, internally mounted circumferential strain gauges, and that the relationships mirror those for imposed load shown in Figures 7.19 and 7.36. To calculate the degree of distortional bending the respective values of hoop compression have been deducted from each of the internal circumferential pipe wall strains. This approach has been used for Tests 250WPL1, 168WPL2.5 and 168WPL3.5 in Figures 7.39, 7.40 and 7.41 respectively. The strain response during static loading alone has been determined in these graphs. Each figure shows that the initial increments of surface stress result in the largest increase of ring bending and that further increments result in reduced increases of ring bending. This relationship therefore is consistent with that of the diametral strain measurements. Figure 7.39 also shows that under the application of static surface stress only, the deformation of the pipe in Test 250WPL1 was dictated by the nature of the surround. This is evidenced by the recording of an elliptical/inverted 'Y' pipe deformation pattern on the RHS of the pipe (0° to 180°) and an inverted 'heart' shaped deformation pattern on the LHS of the pipe (180° to 360°). As Rogers (1985) stated, application of a cyclic loading tends to conditions the surround causing the pipes to deform elliptically as shown by the pipe responses in Figures 7.40 and 7.41 for Tests 168WPL2.5 and 168WPL3.5 respectively.

7.4.4 Shear and volumetric strains
The aim of this section is to investigate how the shear and volumetric soil strains change as the magnitude of surface stress increases. From the imposed stress readings it is seen that the degree of loading remains approximately constant as the applied stresses increase. The degree of arching therefore remains constant requiring the mobilisation of increased magnitudes of positive arching stresses. This is examined in Figures 7.42, 7.43, 7.44 which show the shear soil strains and 7.45, 7.46, 7.47 which show the volumetric soil strains for Test 250WPL1 under the application of 50, 100 and 150 kPa uniform surface stress, respectively. The patterns of the shear strain distributions are initially described to enable the nature and direction of the load path to be defined and compared to the regions of compression and expansion.
Figure 7.42 shows that under a uniform surface stress of 50 kPa, a near continuous 'ear' shaped zone of positive shear strain, of 2% in magnitude, occurs around the pipe shoulders to the elevation of the pipe crown. This zone extends around the pipe circumference to the pipe haunches, below which the formation of negative arching strains are observed, and laterally to the edge of the measured area. Above the level of the pipe crown to a level approximately 100mm above it, strains of typically 1% in magnitude are observed, whereafter the strains approach zero, indicating that, at this stage, the water bag is not limiting the vertical extent.

Increasing the uniform surface stress to 100 kPa (Figure 7.43) resulted in a continuation of this trend. Whilst the maximum shear strain remained unaltered at ±2%, the clarity of the zones of positive and negative arching strains improved with the number and size of isolated areas of strain increasing. This indicates that as the magnitude of surface stress increased both the magnitude and extent of the shear strains increased also, thus causing a greater transfer of stress from above the pipe crown to the sidefill material. This is highlighted at the RHS of the pipe where an increased area of continuous positive arching strains (-2%) is recorded. In addition, the zone of negative arching strains beneath the haunch extend laterally beneath the pipe invert.

Figure 7.44 shows further increases in the magnitude and extent of shear strain as the surface stress was raised to 150kPa, with the 1% contour reaching the elevation of the water bag. At this stage the pattern of distribution on the RHS of the pipe corresponds closely to that of an elliptical pattern observed previously in Test 250WPL1 in Figure 7.23 and less clearly to 250WPD1 in Figure 7.22. Whilst a similar general pattern is observed on the LHS, greater vertical extents of 2% strain occur corresponding to the more inverted heart shaped distributions of Figure 7.24.

The volume changes due a uniform surface surcharge of 50 kPa are shown in Figure 7.45. As the pipe settles relative to the sidefill, thereby inducing the positive arching stresses, compressive strains are recorded around the pipe wall which are in excess of those generally observed in the adjacent sidefill. Below the pipe invert and haunches an
area of compression is recorded. This figure also shows two zones of dilation occurring above the pipe crown and more consistently beneath the RHS pipe haunch. This latter area is due to the bedding beneath the pipe haunch settling relative to the pipe which was shown by the negative arching strains. Above the pipe crown an approximately horizontal contour of 0% volumetric strain is observed extending on either side of the pipe to the edge of the measured area. This line separates the zones of compressive strain below it from the predominately zero and occasional dilation zones of volumetric strain in the overlying soil.

By increasing the surface stress to 100kPa the trends are more clearly defined (Figure 7.46) with a central region of dilating sand above the pipe crown being bound by 0% strain lines extending from the pipe shoulders to the water bag at an angle of approximately 45°. Below these boundaries and besides the pipe is a region of compression (typically between 0 and -2%). Symmetrical zones of dilation are shown directly beneath both pipe haunches, bounded by areas of compression to the side of the springline and below the invert.

A different pattern is shown in Figure 7.47 as the overburden stress is increased to 150kPa. The contour of zero volumetric strain seen previously extending from the pipe crown is observed approximately 100mm above it. Around the whole pipe wall the soil is observed to be in compression, which is consistent with the conclusions drawn from the internal circumferential pipe wall strains discussed in Section 7.4.3. Maximum compressive strains of approximately 4% were recorded adjacent to the pipe wall, most notably beneath the pipe invert and haunches positions. Above the zero compression line dilation effects are observed extending from the water bag.

7.5 EFFECT OF CYCLIC LOADING
7.5.1 Introduction
Cyclic loading was performed during Tests 168WPL2.5, 168WPL3.5 and 120WPL2.5 by repeated application of a uniform surface stress of 70kPa for up to a maximum of 1000 applications. Throughout this section the transient response is defined as that recorded during the application of surface stress with respect to that prior to it, the
permanent response is that recorded after the application of surface stress (i.e. zero applied surface stress) with respect to that recorded prior to loading, whilst the recoverable response is that between the transient and permanent responses. To measure these responses, the behaviour of the pipe and surrounding soil would require to be recorded before, during and after each application of surface stress. During these tests the response was recorded during and after the application of surface stress at various stages enabling the permanent and recoverable imposed load to be recorded during Tests 168WPL2.5 and 168WPL3.5 (i.e. not 120WPL2.5 due to erratic strain gauge readings), whilst the transient response was recorded between stages. The pipe response was recorded in each of these three tests during the application of surface stress at various stages, in addition to the permanent response in Test 168WPL2.5 and after 1000 applications during Tests 168WPL3.5 and 120WPL2.5. Table 7.4 presents the values of imposed load at various stages of this cyclic loading together with diametral strains for the two plastic pipe tests, and the overall vertical displacement in the case of the clay pipe test using Step 5 as datum readings. This section initially describes the pipe response during the two plastic pipe tests and then via soil strain distributions, compares the results of the PVC-U and clay pipes at a C/D ratio of 2.5.

7.5.2 PVC-U pipe tests

7.5.2.1 Imposed load and diametral strains

Based upon the average springline wall strains, Table 7.4 and Figure 7.48 show that during both Tests 168WPL2.5 and 168WPL3.5 after the first application of surface stress the magnitude of the transient imposed load on the pipe reduced with repeated loading. This is emphasised by the increase of transient load on the pipe during the first cycle of -2.216 and -2.290kN/m for Tests 168WPL2.5 and 168WPL3.5, respectively, reducing to -1.473 and -0.524kN/m between the first and 100th cycle, by -1.105 and -0.654kN/m, between the 100th and 500th cycle and by -0.254kN/m between 500th and 1000th cycle during Test 168WPL3.5. With respect to Step 5 as datum readings, Table 7.4 shows that these changes correspond to a degree of loading of 58 and 64% for Tests 168WPL2.5 and 168WPL3.5, respectively during the application of the first cycle, reducing to 55 and 51% after 100 cycles and to 49 and 47% after 500 cycles. As the transient imposed load on the pipe decreased, the degree of positive arching would
have correspondingly increased. Figures 7.36, 7.38 and 7.48 indicate that as the transient imposed load decreased under repeated loading, the uniform hoop compression would also have decreased. Figure 7.48 indicates that although the transient imposed load on the pipe decreased with repeated loading after the first application, the permanent imposed load on the pipe was more variable, showing both increases, but typically decreases. Increases indicate that increased permanent negative arching strains were induced, whilst decreases indicate that increased permanent positive arching strains were induced.

Figure 7.49 shows the measured values of VDS and HDS for these load regimes. It is clear that the increase of VDS and HDS decreased exponentially with repeated loading, tending to an equilibrium value after the application of 500 to 1000 cycles. This finding agrees with the laboratory test data presented by Rogers et al (1995) and reproduced in Figure 3.31. Unlike the imposed load readings, there is not a large recoverable response of the pipe which is indicated by a difference between the VDS and HDS values during and after (i.e. permanent) loading of less than 0.1% during both tests.

From these results it would appear that under the application of surface stress the plastic pipe deforms relative to the adjacent sidefill inducing positive arching strains, as evidenced in Figure 7.48 by the measured imposed load being less than that of the overburden (-3.850 and -3.571kN/m for Tests 168WPL2.5 and 168WPL3.5 respectively). On removal of the applied stress the elastic recovery of the plastic pipe is greater than that of the sidefill, which has been subjected to greater permanent settlements. The stiffness of the sidefill thereby increases inducing greater negative arching strains, as evidenced by the permanent imposed load on the pipe in Figure 7.48. Repeated loading results in an increase of permanent and transient deformation which decreases exponentially for both the plastic pipe and sidefill material. The transient deformation of the plastic pipe exceeds that of the sidefill throughout this process, inducing greater positive arching strains.
7.5.2.2 Settlement during and after loading

To investigate this further, this section describes the changes in settlement of the pipe and the sidefill material during and after loading in Test 168WPL2.5. Settlement profiles rather than soil strain distributions have been used for this purpose mainly to emphasise more simply the differences between the two states. Figures 7.50a and b show the settlement profiles at the elevation of the pipe crown and invert respectively.

Both figures show that the increase of pipe crown and invert settlement decreased exponentially with repeat loading. This is emphasised by the measured settlements at the pipe crown of 1.15, 2.0 and 2.6mm, with corresponding invert settlements of 0.63, 0.75 and 1.13mm during the application of 1st, 100th and 500th cycles respectively. As the number of applications increased the measured difference in settlement during and after an application of load converged, indicating that the magnitude of the elastic recovery of the pipe decreased with repeated loading. These results therefore illustrate the increase of VDS decreasing exponentially with repeat loading and the reduction in the elastic recovery of the pipe that were both shown in Figure 7.49.

Figure 7.50b indicates that similar settlement profiles were recorded either side of the pipe invert, throughout this load sequence which contrasts with the behaviour shown in Figure 7.50a at the elevation of the pipe crown. To the LHS of the pipe crown, positive arching strains are shown as the pipe crown settles more than the sidefill, whilst negative arching strains are generally recorded on the RHS. Based upon the relative settlement of the soil during the application of surface stress to the LHS of the pipe crown, particularly immediately adjacent to it, Figure 7.50a shows that the positive angular strain increased with repeated loading. For the same magnitude of applied surface stress larger positive arching strains would be induced, reducing the imposed stress on the pipe. This settlement pattern supports the imposed stress readings shown in Figure 7.48. To the RHS of the pipe however, a positive arching behaviour is observed during the application of the 1st cycle, whilst negative arching strains are thereafter. This transition from positive to negative arching is not reflected by a significant increase in the measured imposed load at the RHS springline. Table 7.4 shows that the RHS springline (90°) reading during loading remained relatively
constant throughout the loading regime. This contrasts to the reduction in imposed load at the LHS pipe springline (270°) which resulted in the net reduction of imposed load with repeated loading.

At the pipe invert (Figure 7.50b) the application of repeated loading led to greater soil settlements being recorded beneath the pipe haunches. This was particularly shown during the 500th cycle, where soil settlements of -1.50mm were recorded compared to -1.15mm at the pipe invert and -1mm at offsets of ±200mm from the pipe centreline. As this behaviour was observed to a reduced, and more symmetrical, extent during Test 168WPL3.5, it has been included in the general settlement pattern for a buried flexible pipe under the application of cyclic loading, shown in Figure 6.16b. It appears to be attributed to the method of placement which leads to an area of weaker material beneath the pipe haunches compared to the adjacent sidefill. Differences between the method of placement beneath these regions on either side of the pipe may account for the different settlement patterns observed at the level of the pipe crown during this test. The observation of negative arching strains around the circumference of the RHS of the pipe explains the absence of a reduction in the imposed load during repeated loading, compared to that on the LHS. In addition, it is noted that the permanent imposed load on the RHS increases from -0.819 to -0.902 kN/m due to the application of 500 cycles whereas the more positive arching behaviour on the LHS of the pipe causes a slight reduction from -0.721 to -0.649 kN/m.

Both Figures 7.50a and b show clearly that the elastic recovery of the pipe was greater than that of the surround. This would explain the observation of an increase in permanent imposed load on the pipe which reaches an equilibrium value corresponding to that of the permanent settlements.

7.5.2.3 Influence of cover depth
By comparing the results of Tests 168WPL2.5 and 168WPL3.5, having C/D ratios of 2.5 and 3.5 respectively, the influence of cover depth can further be assessed (see Section 7.3). Figure 7.36 shows that the pipe sustains a higher stress due to overburden and static stress up to 70kPa when buried deeper. This appears to
contradict the expected behaviour of the imposed load decreasing as the cover depth increases under the same applied uniform surface stress. It may however, be due to the greater initial sidefill stiffness that was formed during installation of Test 168WPL2.5. Application of increased magnitudes of surface stress appear to remove these differences, however, as evidenced in Figure 7.36 by the convergence of the curves between 50 and 70kPa.

Under repeated loading, however, the reduction of imposed load during Test 168WPL3.5 was greater than that recorded during Test 168WPL2.5 such that almost identical readings were obtained thereafter. This indicates that repeated loading caused the installations to become uniformly stiff, and as the extra stress due to the additional 250mm soil is negligible, then at C/D ratios greater than unity identical behaviour would be expected. The comparable readings of diametral strain shown in Figures 7.37 and 7.49 support these findings as the changes in VDS and HDS during both tests are practically identical.

The influence of increasing the cover depth was more clearly shown during the cyclic loading regime. Figure 7.48 clearly shows that the average imposed load during repeated loading on the pipe in Test 168WPL2.5 was greater than that of the deeper installation of Test 168WPL3.5. However, the permanent imposed load is shown to be greater during the deeper test, indicating that the stiffness of the sidefill increased at a greater rate causing greater load shedding.

To investigate the relative behaviour of these two tests further Figures 7.51a and b show the settlement profiles at the pipe crown and invert respectively. It is apparent from this comparison that the settlement of the pipe invert during Test 168WPL3.5 was consistently less that that during Test 168WPL2.5. The settlement profiles for Test 168WPL3.5 were also more symmetrical. Furthering the discussion on the influence of the soil stiffness beneath the pipe haunches, during Test 168WPL2.5 the asymmetrical settlement pattern was associated with a convergence of the two imposed load measurements during loading (see Table 7.4), as the surround became uniformly stiff. However, the result of this settlement pattern was to cause the two permanent
imposed loading readings to diverge as the imposed load at the RHS springline increased from -0.729 after the 1st cycle to -0.812 kN/m after the 500th cycle due to the negative arching pattern, whilst the reading at the LHS associated with slight positive arching remained relatively unaltered at approximately -0.5 kN/m. Whilst a generally symmetrical settlement pattern was observed during Test 168WPL3.5, greater soil settlement was recorded beneath the pipe haunch at the RHS compared to LHS. The resulting negative arching strains at the RHS led to a relatively stable imposed load readings at the RHS springline during and after repeated loading (typically -2.0 and -1.5 kN/m respectively), whilst at the LHS the more positive arching strain pattern, as expected, decreased markedly the imposed load.

7.5.3 Soil strains
Shear and volumetric strain contours have been determined for Tests 168WPL2.5 and 120WPL2.5 during the 1st, 100th and 500th application of a uniform surface stress of 70 kPa. Due to the use of a WPL surround and a C/D ratio of 2.5, these tests allow a direct comparison to be made between the behaviour of buried PVC-U and clay pipes (and hence pipe stiffness) under cyclic loading. The only complication is due to the use of a smaller diameter clay pipe than that of the plastic pipe, which, for the 70 kPa surface stress, leads to equivalent C/D ratios of 29 and 20 respectively.

It was explained in Section 7.2 that during Test 168WPL2.5 active arching and passive arching strains were indicated during the first cycle of the 70 kPa surface stress (see Figure 7.30), although no major concentrations of shear strains were recorded. This was consistent with the settlement profiles shown in Figures 7.50a and b. The volumetric strains shown in Figure 7.52 under the application of the first cycle of 70 kPa surface stress indicates only a single major concentration of -2% strain within the pipe bedding. The absence of shear and volumetric strain is also shown in the permanent vertical displacement contour plot (i.e. Step 5-8) after the removal of surface stress (Figure 7.53) as a constant region of -0.5 mm settlement was recorded around the pipe circumference. This distribution of permanent settlement therefore resulted in a lack of major concentrations of shear and volumetric strain (Figures 7.54
and 7.55 respectively) which have not be added to those recorded at the 100th (Step 8-9) or 500th cycle (Steps 8-11).

The application of the first cycle of uniform surface stress during Test 120WPL2.5 (clay pipe) was also described in Section 7.2 where, similarly to Test 168WPL2.5, no major concentrations of soil strains were recorded. This is highlighted in the shear strain distribution (Figure 7.31), which indicates the occurrence of both passive and active arching strains to the LHS and RHS of the pipe respectively.

The major effect of the applying a further 100 cycles of the 70kPa surface stress during Test 168WPL2.5 was the change from positive to negative arching strains recorded within the sidefill adjacent to the RHS pipe shoulder. This is shown by the shear strain distribution in Figure 7.56. This zone of negative arching strain (indicated by the positive values) is confined vertically by a near continuous horizontal 0% shear strain contour approximately 80mm above the pipe crown, and laterally 40mm from the pipe springline. Below this and adjacent to the RHS pipe haunch, a significant area of positive arching strains is shown reaching a maximum magnitude of -4%. A similar area is also shown adjacent to the LHS pipe haunch. Whilst it is less clearly defined, negative arching strains are recorded at the pipe wall. This pattern of behaviour is due to a narrow area of soil settling relative to the pipe into the looser placed material beneath the pipe haunch. As Figure 7.57 shows, at the 500th cycle this pattern of shearing has become more prominent. The observation of soil flowing around the pipe, particularly into the haunch zones, is highlighted by the compressive strains recorded beneath the pipe haunches in Figures 7.58 and 7.59 during the application of the 100 and 500th cycle respectively. A similar pattern of settlement beneath the pipe haunches was also observed during Test 120WPL2.5 (see shear strain patterns in Figures 7.60 and 7.61) as the vertical displacement contours in Figures 7.64 to 7.66 clearly show. These displacement patterns have been used to formulate the general settlement pattern of a rigid pipe subjected to cyclic loading (Figure 6.16a). This type of displacement beneath the pipe haunches therefore appears to be due mainly to the application of cyclic loading, as it was not evident during the application of surface static stress using the 250mm diameter PVC-U pipe.
As well as increasing the magnitude of the shear strains adjacent to the pipe haunch, the application of a further 400 cycles (to 500 cycles) in Test 168WPL2.5 (Figure 7.57) exaggerated the general pattern developed after 100 cycles. Application of 100 and 500 cycles of the 70kPa uniform surface stress in Test 120WPL2.5 resulted in the establishment and enhancement (respectively) of negative arching strains adjacent to the buried clay pipe. These shear strains are shown in Figures 7.60 and 7.61 for the 100 and 500th cycles respectively and the associated volumetric strains are shown in Figures 7.62 and 7.63. Due to measurement of significant horizontal displacements at these steps, these plots were determined from both horizontal and vertical displacements.

The respective contours of vertical displacements (Figures 7.64 and 7.65) show that the presence of a rigid rather than a flexible buried pipe causes the sidefill to settle relative to the pipe. As with the plastic pipe tests, the lateral influence of the pipe is seen to extend to the edge of the measured area. The greatest relative settlement of the sidefill occurs close to the pipe, whereafter the settlement decreases with distance. The patterns of the contour lines indicate that the surround appears to flow around the pipe extending down to beneath the pipe haunches, with the more rigid bedding settling in a more uniform manner. Nevertheless, the rigid inclusion does result in greater bedding settlement beneath the pipe invert, as expected. The influence of the pipe is also observed directly above the pipe crown, where settlements are duly lower than those above the sidefill and the soil will consequently stiffen to a greater degree. This will subsequently lead to the attraction of further load if cyclic loading progresses resulting in the progressive development of negative arching strains that extend vertically from the crown towards the surface.

The formation of two exterior prisms settling relative to a stiffer interior (i.e. the prism above the pipe) in the manner described by Marston (1930), is therefore shown in the general settlement patterns in Figure 6.16a. Consistent with this theory, an approximate height of equal settlement is shown above the pipe crown. By including the vertical settlement contour plot after a 1000 cycles (Figure 7.66), it can be seen
that the elevation of this plane above the pipe crown increases as the number of cycles increases (being 160, 190 and approximately 250 to 300mm above the pipe crown in Figures 7.64 to 7.66, respectively). The increasing vertical extent of the shear planes above the pipe crown with the number of loading cycles is also clearly shown in Figures 7.60 and 7.61. The extent and distribution of these shear planes corresponds with those indicated by the upward trapdoor experiments conducted by Ladanyi and Hoyaux (1969) shown in Figure 2.7b. Maximum negative shear strains of 9 and 15% were recorded at the pipe-soil boundary in Figures 7.60 and 7.61 respectively. For the same value of applied surface stress, the increased vertical extent and magnitude of passive shear strains will result in an increase of imposed stress on the pipe.

7.6 COMPARISON OF BURIED PIPE RESULTS TO DESIGN THEORIES

7.6.1 Introduction

Section 3.1 described the historical development of buried pipe design in which Marston's load theory, the Iowa deflection formula and the TRL design method were all outlined. The results from the buried pipe tests in this study have been compared to the US development of Marston's load theory and the Iowa deflection formula in Section 7.6.2, and to the UK development of the TRL method in Section 7.6.3. In addition, these results could be compared to those from finite element analyse, although this lay beyond the scope of the current study.

7.6.2 Marston load theory and Iowa deflection method

Marston's load theory provides the basis for determining the applied load on a 'rigid' pipe by using the shear plane method to calculate the degree of negative arching. Theoretically it could also be used to determine the degree of positive arching with 'flexible' pipe installations, thereby indicating the level of redistribution of stress from the overburden into the sidefill. Presently, values of $E'$ are based on back-calculations of pipe deflection assuming an initial applied prism load and are assumed to be constant with applied stress (i.e. horizontal and vertical, if elliptical deformation is assumed, deflection is linear with applied stress). The aim of this section is to investigate these relationships using the test data, and where possible, to back-calculate terms used in both Marston's load theory and the Iowa deflection theory.
7.6.2.1 Back-calculated values of relative settlement \((r_{ad})\) and load coefficient \((C_c)\)

The installation condition used for the buried pipe tests in this study relate to Marston's positive projection (negative projection refers to pipes in trenches). Figure 3.4 shows the relationship between relative settlement \((r_{ad}\) defined by equation 3.4), cover/diameter \((C/D)\) and the load coefficient \((C_c)\) for this installation condition. These relationships assume values of \(K_{\mu} = 0.13\) and 0.19 for the trench and projection conditions respectively. Adopting Marston's value of \(K = K_{\mu}\), values of \(K_{\mu}\) of 0.12 and 0.10 were recorded for the loose and dense Leighton Buzzard sand. These values of \(K_{\mu}\) therefore represent the lower limit of use for the projection condition. Back-calculated values of \(r_{ad}\) have been determined from the vertical displacement contour plots and back-calculated values of \(C_c\) from the imposed load readings at the pipe springline using Step 5 as datum readings. Due to the lack of reliable imposed load calculations from the two clay pipe tests, it is not possible to relate the measured values to those predicted from the method.

Figures 7.67 and 7.68 show the comparison of imposed load calculated from the strain gauge measurements against \(C_c\) for the static and cyclic loading regimes respectively. For the clay pipe tests it was possible to determine values of relative settlement during Test 120WPL2.5 but, due to the influence of the water bag arrangement, not during Test 120WPL1. Measurements of the pipe invert settlement \((s_f)\) during the clay pipe tests have been recorded directly from the vertical displacement plots assuming the vertical diametral shortening of the pipe \((d_e)\) to equal zero, whereas for plastic pipe tests both \(s_f\) and \(d_e\) were measured. The vertical displacement contour plots for these tests show that the lateral influence of the pipes had not diminished at the edge of measured area. Values of sidefill settlement \((s_m)\) and original ground level settlement \((s_o)\) are assumed to be the settlement of the exterior prism away from the influence of the pipe. Measurement of these two values at the extremes of the digitised area will consequently lead to reduced magnitudes of relative settlement for both the clay and plastic pipe tests. The effect of this can be assessed by comparing the values of \(C_c\) determined from these values of relative settlement, shown in Figures 7.69 and 7.70 for
the static and cyclic loading regimes respectively with the values of $C_c$ determined from the imposed load calculations (Figures 7.67 and 7.69 respectively).

Figures 7.67 and 7.69 represent the results of measured $C_c$ and $r_{ad}$ respectively, due to the application of uniform surface stress, overlain by Marston's relationships. As stated in Section 7.4 a linear relationship of applied and imposed load was typically recorded during these plastic pipe tests. Figure 7.67 shows that this corresponded to a constant value of $r_{ad}$ being recorded. The greatest degree of arching was recorded during Tests 250PPL1 and 250WPD1* (i.e. lowest $C_c$), which led to the lowest measurements of $r_{ad}$ of less than -2.0. In comparison with Test 250WPD1* the lack of compaction to the sidefill and backfill during Test 250WPL1* increased the degree of loading on the pipe as shown by the increase of $r_{ad}$ to -2.0. In Test 250WPL1, however, the recording of a single imposed load measurement at the RHS springline only, led to lower values of $r_{ad}$ than those recorded during the repeated Test 250WPL1* (where imposed load was measured at both springlines), which were subsequently closer to those of the WPD surround. For each of these tests, therefore, 'incomplete' trench installations are indicated by the results of measured $C_c$ and $r_{ad}$, which would lead to the expected observation of a plane of equal settlement within the overlying soil. This was not, however, apparent during the tests placed at a C/D ratio of unity due to the use of the water bag arrangement (rather than soil) for the application of additional C/D ratios. Increasing the soil cover depth from 250 to 500mm decreased the imposed load on the pipe slightly, as seen by the pipe invert response during Test 250WPL2 compared to 250WPL1* in Figure 7.32, as the growth of vertical positive arching stresses into the overlying soil was not limited by the water bag. In addition, increasing the pipe stiffness by the use of the 168mm, compared to the 250mm, diameter PVC-U pipe decreased the relative settlement to approximately -1.0 during Test 168WPL2.5 compared to a value close to -2.0 during Test 250WPL2, which were carried out with comparable C/D ratios.

The relative settlements determined from the imposed load readings were greater than those measured from the vertical displacement contours. For example, the $r_{ad}$ values from the imposed load readings in Tests 250WPL1 and 250WPL1* were...
approximately -2.0 or less, whilst \( r_{ed} \) values determined from the settlements were greater than -1.0. This discrepancy is due the use of exterior prism settlements measured relatively close to the buried pipe, and thereby still influenced by the pipe, which correspondingly reduced the values of relative settlement. Importantly, however, the relationships between the tests is approximately maintained. For the C/D=1 tests, the settlement-based \( r_{ed} \) values indicate that 'incomplete' installations (i.e. not a complete trench condition, therefore the plane of equal settlement is below the ground surface) would be expected if the load on the pipe was solely due to sand overburden. As the existence of a plane of equal settlement was not established by the contour plots, the limitations of using a water bag arrangement to simulate additional heights of fill are evident. For the tests with a C/D ratio of 2 or more 'incomplete' installations were recorded by both the imposed load and settlement based \( r_{ed} \) values, as was the establishment of a plane of equal settlement above the pipe crown. It can be concluded that the use of a water bag arrangement to simulate a soil surcharge is only valid (i.e. produces a fully accurate response) at C/D ratios greater than approximately 1-2, thus concurring with Hoeg (1968) and Terzaghi's (1936) results, since above this ratio the presence of the water bag does not limit the vertical extent of the arching strains.

Considering now the effects of cyclic loading Figure 7.70 shows that the use of a clay pipe in Test 120WPL2.5 resulted in a relative settlement of +0.1 during the first application of the uniform surface stress of 70 kPa. This positive value indicates the occurrence of negative arching. As the cyclic loading is applied the value of \( r_{ed} \) is seen to increase (although with the effects of each cycle the rate of increase progressively decreases) until an \( r_{ed} \) greater than +1.0 is reached after 1000 cycles. This implies that the degree of loading on the pipe therefore increases similarly, an observation that cannot due to the erratic behaviour of the strain gauge reading during the clay tests be compared to the imposed load readings. Application of a higher magnitude of uniform surface stress (150kPa) resulted in \( r_{ed} \) returning to +0.1, i.e. the value recorded prior to the application of cyclic loading. It would be expected that because shear strains of up to 10% in magnitude were recorded during the application of the 500\textsuperscript{th} cycle that the soil had begun yielding, resulting the imposed load increasing faster than that applied, leading to an increase in \( r_{ed} \) after the cyclic loading regime.
Figure 7.70 shows that an increase of $r_{ad}$ was also recorded in Tests 168WPL2.5 and 168WPL3.5 during cyclic loading, indicating an increase of imposed load on the pipe. This behaviour contradicts the conclusion from the imposed load measurements during the application of loading, which are seen to decrease as the cyclic loading progresses (Figure 7.6S). The negative arching $r_{ad}$ values (Figure 7.70) were recorded primarily because the negative arching settlement pattern at the pipe invert was more dominant than the positive arching at the pipe crown. This influence is seen to be maintained during Test 168WPL2.5 after the cyclic loading regime, whilst in Test 168WPL3.5 the $r_{ad}$ value returns to the pre-cyclic loading level of -0.1. The results from these three tests highlight in particular the importance of defining the lateral limit of the exterior prism, as the free-field settlements ($s_m$ and $s_d$) in these tests were greatly influenced, particularly at the elevation of the pipe invert, by the buried pipe.

7.6.2.2 Back-calculated values of $E'$

As with the TRL method, successful use of the Iowa deflection formula is dependent on the selected value of soil stiffness ($E'$ in the case of the Iowa formula) as this defines the behaviour of a buried 'flexible' pipe-soil system. The aim of this section is to present values of $E'$ derived from the PVC-U pipe tests to assess variations during static and cyclic loading, and due to support conditions and cover depth. Using Equation 3.8, back-calculated values of $E'$ are presented in Table 7.5 based upon measurements of horizontal deflection and vertical deflection inferred using an elliptical pipe deformation pattern as the only comparison. These have determined according to measured values of bedding constant ($K$) and the applied prism load, which as Section 6.1.3 described incorporated the influence of wall friction from the test tank. For the PPL tests, a bedding angle of $15^\circ$ was recorded at Step 1, which relates to a bedding constant of 0.108. By hand feeding sand beneath the pipe haunches the bedding angle was increased to $35^\circ$ at Step 2, which was further increased to $90^\circ$ by raising the elevation of the sidefill to the pipe springline. For the WPL and WPD tests, bedding constants of 0.1 and 0.083 have been used corresponding to the angles measured at Steps 2 and 3 respectively.
The general trend (although by no means consistent) is of $E'$ increasing, with the rate of increase decreasing as the static surface stress increases. This behaviour corresponds to the pattern of HDS and VDS increase and is particularly noticeable in the two PPL tests. Table 7.5 shows that the back-calculated values of $E'$ were almost identical throughout both Tests 168WPL2.5 and 168WPL3.5, and at 150kPa in Test 250WPD1^ (i.e. where the readings of VDS and HDS were significant) where the pipe in these tests deformed elliptically (see Table 6.4). Table 6.4 also shows that the deviation from the elliptical deformation pattern was most noticeable during the two PPL tests in which the pipes deformed according to the inverted 'heart' shape. As the inverted 'heart' shape is characterised by excessive deformation at the pipe haunches, this reduces the degree of lateral deformation at the pipe springline compared to that of the elliptical shape which, as Table 7.5 shows for these two tests, leads to values of $E'$ from the HDS readings being approximately twice those from the VDS measurements. Where the deviation from the elliptical pipe response is not as acute as in the two PPL tests (for example in the two 250WPL1 tests), the increase of $E'$ from the HDS, compared to VDS, readings is reduced.

From the results of the tests installed with a loose surround it appears that a value of $E'$ of approximately 30MPa from the HDS readings (or approximately 20MPa from the VDS readings) was generally applicable for the initial increment of uniform surface stress of either 50 or 70kPa. (The exception to this was for Test 250WPL1, which was considered to be have a stiffer surround as a result of the installation procedure, see Section 7.1.2). By comparing these values to those originally presented by Howard (1977) and reproduced in Table 3.3, it appears that these are in close agreement with class 'c' material at >85% Standard Proctor density or class 'h' sand at >95% Standard Proctor density. Increasing the initial relative density of the Leighton Buzzard sand used for the sidefill and backfill from 85 to 92% of its maximum density by compaction resulted, on the basis of the 150kPa readings during Test 250WPD1^, in an approximate doubling of the $E'$ values recorded in the two 250WPL1 tests. The values in Table 3.3 for a relative density of nearly 95% therefore appear conservative in relation to the approximate value of $E'$ of 60MPa recorded during Test 250WPD1^.
Applying the cyclic load in Tests 168WPL2.5 and 168WPL3.5 is shown to result in a progressive reduction of $E'$ as the cycles are applied, matching the pattern of VDS increase shown in Figure 7.49.

### 7.6.3 TRL method

The TRL method was described in Section 3.2.3 and can be used to determine ring deflection ($\delta$), total thrust (N), and bending moment (M), of the buried pipe by use of Equations 3.19 to 3.21 respectively. This section describes how the measured values of pipe deflection and pipe wall thrust from the buried pipe tests were used to back-calculate the soil modulus parameter ($E_s^*$) and the dead load lateral earth pressure ratio ($K_d$). As the previous section highlighted, the value of soil modulus is modified by the nature of the applied stress regime, the method of installation and degree of compaction to the surround. In addition it would also be expected that the value of $K_d$ would vary with the stress regime applied. Values of $K_d$ approaching zero imply more distortional conditions as a result of the use of uncompacted backfills, whilst values of $K_d$ of unity imply more uniform loading conditions. As discussed in Section 7.4, as the uniform surface stress increased the tendency to distortional bending decreased since the stiffness of the sidefill increased. The decrease in the significance of distortional bending during static loading would lead to an increase in the value of $K_d$.

It is not possible, however, to determine from these results the variations of both of these factors. Due to the greater importance of the soil modulus term, as shown also by the Iowa formula, the method adopted was to consider the variation of $E_s^*$ for a constant value of $K_d$. To aid the definition of $K_d$ back-analysis was initially conducted using the results from Test 250WPL1*. This was chosen as its criteria and performance matched closely those of the design method (Gumbel 1983).

Three possible methods were identified for back-calculation of $E_s^*$ from measured deflection and are denoted M1, M2 and M3 respectively. These are:

1. To assume the measured VDS was equal to the first order distortional component ($\delta_{y1}$) only. From Equation 3.25 the value of $E_s^*$ could be determined. As the
contribution of uniform deflection is omitted this method mirrors that of the Iowa method except for the use of distortional stress rather than applied vertical stress.

2. From Equation 3.23 the uniform hoop deflection ($\delta_2$) could be determined. By measuring VDS the first order distortional component ($\delta_{y1}$) could be found and used as above to find $E_s^*$. This method is suitable for cases where the value of HDS was not measured.

3. From the measured values of VDS and HDS in addition to the predicted value of $\delta_2$, an average value for $\delta_{y1}$ is obtained and used to find $E_s^*$.

For each of these cases the second-order distortional component of deflection was not calculated. The loads $P_z$ and $P_y$ were determined from the applied vertical stresses using the method described in Section 6.1.3. In addition, a value of $K_d$ of 0.4 was assumed which represented an average value suggested by Gumbel for a loose state sand and was approximate to the $K_o$ value, assuming $K_o=1-\sin\phi$ (Jaky, 1948), where $\phi$ is the effective angle of friction for Leighton Buzzard sand. Table 7.6 presents the results of $E_s^*$ for each of the three methods for each of the buried plastic tests. The first method is shown consistently to yield the smallest value of $E_s^*$ because the distortional deflection is necessarily largest (i.e. no reduction due to uniform hoop deflection is included). In cases where a larger value of $E_s^*$ was obtained by the third, compared to the second, method the measured value of HDS was not in correspondence with the measured VDS.

Figure 7.71 shows the measured hoop thrust during Test 250WPL1* compared to that predicted using equation 3.20 from the back-calculated values of $E_s^*$ using method 3 (and incorporated into the flexural and compressive stiffness ratios, equations 3.14 and 3.15), for no-slip (NS) and full-slip (FS) conditions and values of $K_d$ between 0.3 and 0.5. Excellent correlation is shown between the measured and predicted values assuming full-slip conditions and a $K_d$ value of 0.4. As expected, a significant increase of predicted thrust is predicted when the alternative no-slip condition is used as the shear stresses predicted at the interface are transmitted to the pipe. Figure 7.72 shows the same method used for Test 250WPD1*, which was identical to Test 250WPL1*, except the surround was compacted (i.e. dense). It is shown that, as with the
uncompacted case, the closest prediction to the measured values occurred when the back-calculated values of $E_*$ were used with a full-slip analysis and a $K_d$ value of 0.4.

In view of the close correlation between the measured and predicted values of thrust, this approach was repeated for the remaining plastic pipe tests using the back-calculated values of $E_*$ shown in Table 7.6, and $K_d = 0.4$. Figure 7.73 shows excellent correlation for the full-slip condition for both Tests 250WPL2 and 250WPL1*, in which both pipes deformed approximately elliptically, thereby inducing arching strains distributions that were only marginally different (see Figures 6.25a and b). In contrast, the measured thrust reading (from the single springline strain gauge in this test) during Test 250PPL1 is shown to be over-predicted by the full-slip condition. This difference is attributed to the behaviour of the pipe deviating significantly from the elliptical deformation pattern assumed by the theory to an inverted-heart shaped pattern (see Figures 6.25a and c) in which there was a lack of sand support beneath the pipe haunches. This led to the propagation of greater positive arching strains in comparison with that of an elliptically deforming pipe, thus resulting in a greater reduction of load on the pipe.

Increasing the pipe stiffness by the use of the 168mm diameter PVC-U pipe in Tests 168WPL2.5 and 168WPL3.5 resulted in a generally good correlation between the measured and predicted values. This is because the well-placed nature of the surround around the pipe coupled with cyclic loading promoted approximate elliptical pipe deformations. The slight underestimation of thrust in Test 168WPL2.5 implied a more intermediate condition between the extremes of full-slip and no-slip. The development of greater negative arching strains adjacent to the pipe haunches during the cyclic loading sequence of this test was thought to promote this deviation from a full-slip, hence no shear stress transfer to the pipe, condition.

In conclusion, these results appear to be in broad agreement with the recommendations stated by, amongst others, Gumbel (1982) that a ductile material is modelled by a full-slip condition, whilst a brittle material is conservatively modelled by a no-slip condition. These slip conditions can be explained by the respective shear strain.
distribution patterns. For a buried 'flexible' pipe installation prominent positive arching strains were shown around the pipe circumference above the pipe haunches, whilst only a small extent of negative 'dragging' strains were recorded below the pipe haunches. The converse distribution was shown from the buried clay pipe tests under cyclic loading only, illustrating the assumption of no-slip. The only deviation from full-slip condition for a flexible pipe was recorded during cyclic loading where the establishment of prominent negative arching strains particularly beneath the pipe haunches implied a more intermediate condition between full-slip and no-slip.

By analysing the results of $E_*$ in Table 7.6 it is clear that similar conclusions are drawn for the TRL method to those previously stated for the Iowa formula. Both methods assume that the value of soil modulus remains constant resulting in the prediction of a linear increase of pipe deflection with applied load. The results from both analyse show that a value of soil modulus, $E'$ or $E_*$, for a buried flexible pipe installation increases, albeit with a decreasing exponential trend, during the application of linearly increasing static stresses, whilst decreasing in the same manner under cyclic stresses of the same magnitude.

The use of a single value of soil modulus for a relevant value of static vertical crown stress would therefore tend to over-predict the values of vertical and, particularly, horizontal deflection at a higher stress level. It would also for the TRL method, correspondingly over-predict the value of pipe hoop thrust. Whilst not clearly shown from these tests due to the insignificant deflections during the application of 50 and 100kPa in Test 250WPD1, for the same pipe and installation conditions the relationship between $E_*$ and the flexural and compressive stiffness ratios would thus indicate that the lower degree of loading recorded during Test 250WPD1 resulted in a lower pipe deflection and rate of increase, than that of the higher degree of loading case shown in both 250WPL1 tests. This is because the establishment of a higher initial sidefill stiffness by compaction during Test 250WPD1 resulted in a more highly efficient method of load transfer by uniform hoop compression (thus less deflection and rate of deflection) than the loose surround, 250WPL1, tests which experienced greater distortional bending. Similarly, during cyclic loading the converse increase in the
observed diametral strain and decrease in transient imposed load would not be modelled by a single value. As both Tests 168WPL2.5 and 168WPL3.5 showed, the increased soil stiffness created by cyclic loading would lead to the use of a higher $E$ value for measuring the response of a pipe prior to static loading after cyclic loading. Similarly, comparing cyclic to static loading cases at the same stress level would lead to the use of a lower $E$ value.

The importance of back-calculated values of soil modulus was also emphasised by the process of achieving successful correlation between measured and predicted thrusts using the TRL method. Excellent correlation was shown when the buried pipe deformed to an approximately elliptical pipe shape, highlighting the need in practice for well-placed surrounds even when compaction is not applied when pipelines are designed according to this method.

7.7 COMPARISON OF BURIED PIPE RESULTS TO TUNNEL RESULTS

7.7.1 Introduction
The aim of this section is to discuss the results of the two tunnel tests conducted in this study. Section 7.7.2 describes the patterns and extents of the soil displacements, which are compared with those recorded by Chapman (1993) in Section 7.7.3. Section 7.7.4 thereafter presents a preliminary comparison between tunnel tail void displacements and those recorded during the buried pipe tests.

7.7.2 Soil displacements in tunnel tests
The magnitudes and extents of soil displacements around the lining tube were presented in Section 6.3.3 (see Tables 6.13 to 6.16) using the notation described in Figure 6.29. Figures 7.74 and 7.75 show the vertical and horizontal extents of the vertical movements against distance of withdrawal of the outer (shield) tube respectively, with Figure 7.75 additionally showing the horizontal extent of the maximum vertical movement. Figure 7.76 shows the vertical extent of the vertical and horizontal movements against distance of withdrawal of the outer (shield) tube. Figure 7.77 shows the horizontal extent of the horizontal movements and additionally, the
horizontal extent of the maximum horizontal movement which can be compared to the horizontal extent of the maximum vertical movements shown in Figure 7.75.

Figure 7.74 shows that the withdrawal distances of the outer tube during the two tests (being 91 and 150mm for Tests T1 and T2 respectively) was insufficient to eliminate its influence on the observed ground movements as movements were still being recorded. It also shows that these withdrawal distances were sufficient for the induced vertical movements to reached the surface, as shown by the values of B reaching 5 and 3 for Tests T1 and T2 (at C/D ratios of 4.5 and 2.5) of 5 and 3 respectively. Prior to reaching the ground surface, the vertical extent of the vertical movements was marginally greater in Test T2, installed at a shallower depth, than the deeper installation, Test T1. This indicates that for the dense state surround, the extent of movement diminishes with increased C/D ratio as the confining stress is increased, thus increasing the degree of dilation. Figure 7.74 also shows that the dimension that defines bottom zone of vertical movement (C), is measured above the crown of the lining (rather than at the lining invert, C/D=-0.5) from a distance of withdrawal of 120mm (0.62D) during Test T2. The occurrence of vertical movements above the lining crown only, as indicated by the convergence of lengths B and C in this test, shows that the influence of the outer tube on the ground movements was diminishing.

Figure 7.75 shows that the lateral extent of the vertical movements was approximately 0.90D throughout, corresponding to a column of failing soil above the pipe crown of width equal to only 1.8D. With respect to predicting the lateral extent of soil movements using the error function curve (see Equation 3.31), the results support the use of a low value of trough width parameter (i, see Equation 3.32), resulting in a concentrated zone of displacements. Comparing the extents of movement occurring during both tests, it is apparent that greater extents were observed during Test T2 installed at a shallower depth. This supports the conclusion that an increase in cover depth increases the confining stress, which in turn, results in increased dilation effects and reduced extents of movement. A similar conclusion was drawn from the PVC-U pipes Tests 250WPL1* and 250WPL2, placed at C/D ratios of 1 and 2 respectively, where the increased confining stress for the deeper installation led to reduced extents
of movements and a more vertical pattern of positive arching strains extending from the pipe shoulders.

The lateral position of the maximum vertical displacements (Figure 7.75) would be zero for a convergent settlement pattern with a maximum displacement above the crown. It shows, however, that the position of the maximum vertical movement was largely constant at 0.45D throughout the withdrawal process. The main exception to this is at the start of the withdrawal process, where the maximum movement originates at approximately the shoulder of the lining (at a distance of 0.353D) and at the end of Test T2 where the zone of maximum movement is shown directly above the lining crown. This indicates that during the early stages of withdrawal, the maximum displacements emanated from the shoulders, rather than as expected; from the crown position. The observation therefore of two settlement peaks is in agreement with the results by Cording et al (1977) (see Figure 3.40), which was also performed using a concentric tail void loss method. It does not, however, support the convergent settlement patterns observed by Chapman (1993) (see Figure 3.45), or those observed in practical tunnel contracts. A single peak above the crown was, however, observed during Test T2 after a withdrawal of 91mm (or approximately 0.5D). This appears to be due to the two streams of movement from the shoulders forming into a single central zone of movement.

To explain the origins of this movement pattern they have to be analysed with respect to the closure of the large 'V' shaped air void beneath the lining. This was formed as a result of the linear nature of the tail void and the angle of repose of the sand. Whilst the reduction in the size of this was not complete until a withdrawal of approximately 30 mm during both tests, its influence is minimal when it is compared to the size and nature of the volume loss created by the tail void. Reducing the tail void around the base of the lining would led to a more convergent settlement pattern typical of tunnels constructed in both sands and clays.

To emphasis the vertical nature of the soil displacements, the vertical extent of the vertical and horizontal movements are shown in Figure 7.76. It shows clearly that the
vertical extent of the vertical movement consistently exceeds that of the horizontal movements, as indicated in Figure 6.29. The lateral extent of the horizontal and vertical movements appear to be approximately constant at about 0.8D as Figure 7.77 shows. The results do however highlight greater variability with the horizontal movements in comparison to the vertical movements which is due primarily to the zone of horizontal movement extending marginally beyond and above the zone of unobservable movement. The maximum horizontal movement, like the maximum vertical movement, is observed to originate from the shoulder of the tunnel, 0.353D.

In conclusion, it is shown that the vertical and horizontal extents of the horizontal movements are concentrated within a zone of 1D around the lining. This would imply that during calculations of angular strain to determine the likely damage to structures, horizontal movements need not be considered outside a zone 1D above and to the sides of the centre of the lining. Beyond this zone, the angular strains from vertical displacements need only be considered.

7.7.3 Comparison of results with pipejacking tests
A direct comparison of these results can be made with those from the pipejacking tests by Chapman (1993) by adding together the displacements recorded between each of the four incremental steps up to a withdrawal of 10mm, thereby equalling the jacking distance by Chapman. The resulting horizontal and vertical displacement contours during T1 (C/D=4.5) are shown in Figures 7.78 and 7.79. These figures are compared, respectively, to those shown in Figures 7.80 and 7.81 recorded during Test OPJ7 by Chapman. Both Tests T1 and OPJ7 were performed using a C/D ratio of 4.5, an overcut of approximately 0.2 and a dense Leighton Buzzard sand.

Figure 7.78 shows during T1 the extent of horizontal movements were restricted to a radial distance of approximately 170mm (or 0.9D) from the centre of the lining, which is approximate to that of the zone of unobservable movement (Figure 6.28), whereas Figure 7.80 shows that magnitudes of up to 1mm were recorded at a distance of 2D above the pipe crown in Test OPJ7. A maximum horizontal displacement of 4mm was
recorded at the pipe shoulder during Test T1, which is similar in magnitude to that in Test OPJ7, but these dissipate very rapidly.

Closer correlation between the two tests is, however, shown by the vertical displacement contour plots. During both Test T1 (Figure 7.79) and Test OPJ7 (Figure 7.81), vertical displacements of 15mm were recorded approximately 50mm above the pipe crown and shoulder regions. The distribution of these contours in Figure 7.79 indicate that the maximum displacements were propagated from the pipe shoulders and extended laterally above the pipe crown. This would indicate a convergent settlement pattern characterised by a central settlement peak of width equal to that of the lining diameter. The 10 and 15mm contours of vertical displacement are also approximately similar between the two tests, with the exception that those in Figure 7.79 peak above the shoulders rather than directly above the crown.

7.7.4 Comparison of behaviour around tunnels and buried pipes
The comparison between the behaviour around the buried pipe and tunnel tests is based upon the recorded settlement patterns and not (due to the restraining force applied to the lining) on the strain gauge readings, which in the buried pipe tests were used to infer imposed load measurements. The soil displacement patterns recorded during the two tunnel tests (T1 and T2), showed a convergent settlement pattern with two areas of maximum movement originating from the pipe shoulders. This was due to the use of a concentric volume loss distribution around the complete circumference of the lining which promoted settlements below the pipe springline. Measurements of soil movements around practical tunnels indicate a convergent settlement pattern with a maximum settlement (i.e. single peak) recorded at the pipe crown. Whilst these differences are noted, the general pattern of behaviour is similar indicating a convergent settlement pattern of narrow width and an absence of soil movements adjacent to this zone of influence (see Figures 6.27 and 6.28). The resulting shear strain distribution would be similar to those associated with a single peak convergent settlement pattern, an example of which is shown from Hansmire (1975) in Figure 3.35b from a tunnel constructed in granular material. Figure 3.35b shows the development of positive arching strains induced by the convergent settlement pattern.
forming an 'ear' shape that extends from the lining shoulders. Due to the absence of soil displacements below and adjacent to the lining invert, negligible shear strains are recorded. Based upon the imposed load measurements and positive arching strain patterns observed during the PVC-U pipe tests, this shear strain pattern would cause a transfer of in-situ soil stress from above the tunnel lining to the region adjacent to the lining springline. As the settlement patterns during Tests T1 and T2 are similar to those recorded by Hansmire (1975) (Figure 3.35a) it can be concluded that whilst the imposed load on the model lining could not be measured (due to the restraining force applied to the lining), the applied stresses at the lining boundary would have been a fraction of the in-situ soil stresses. Whilst the stiffness of the steel lining would have made it act in a manner similar to a rigid buried pipe, the settlement patterns observed in the clay pipe Test 120WPL1 (see Figure 6.12) under a static surface stresses of 150kPa (i.e. greatly in excess of the applied soil loading in the tunnel tests) indicate that the resulting lining-soil interaction would have had a negligible effect on the observed settlement pattern. The observed settlement patterns during Tests T1 and T2 were therefore due solely to the tail void loss, whilst the influence of the resulting tunnel-soil interaction was negligible.

Comparing these settlement patterns with those from the buried pipes tests, it is clear that the high surface stress applied during the buried pipe tests resulted in the settlement of the soil and buried pipe beneath it, contrasting with the narrow zone of settlement observed above the tunnel tests induced by the tail void loss. These contrasting patterns correspondingly highlight the differences in the position of application and magnitude of the applied stresses (i.e. that the applied stresses at the distant boundary, i.e. buried pipe tests, greatly exceeded those at the tunnel interface, i.e. tunnel tests). The closest correlation under static loading conditions between the two types of tests was with the 250mm diameter WP pipe tests. During these tests the pipes deformed approximately elliptically resulting in a convergent settlement pattern above the pipe crown (see Figure 6.14 for Test 250WPL1). The resulting positive arching strains (Figure 7.44) were shown to extend from the pipe shoulder and springline regions in a pattern approximate to that observed around a practical tunnel (Figure 3.35b). Beneath and adjacent to the pipe haunches, negative arching strains
were induced as the soil adjacent to the pipe invert settled relative the pipe invert. Due to the use of a concentric tail void loss, a similar pattern was noted (though not recordable) for a narrow band of soil adjacent to the haunches during the tunnel tests. The absence of the resulting negative arching strains from the shear strain distribution around a practical tunnel in Figure 3.35b indicates that due to the use of smaller tail voids than that of the model, this type of pattern of behaviour is typically not observed in practice.

Summarising the respective soil behaviour under static loading conditions, it is clear that the buried flexible pipe and tunnel tests behaved similarly in terms of the establishment of positive arching stresses above the crown. Beneath and adjacent to the haunches however, negative arching strains were recorded during the buried flexible pipe tests, whilst an extension of the positive arching strain pattern is shown around practical tunnels. The use of a flexible lining and the resulting interaction between it and the surround would for the interaction part only (on the basis of the proportional relationships of Gumbel, 1981), generate a similar pattern of shear strain to that shown in the buried flexible pipe tests. The complete shear strain pattern would therefore be that induced by the tail void and the resulting lining-soil interaction. Due to the magnitude of displacements induced by the tail void and the low magnitudes of applied soil stress, decreasing the lining stiffness in these tests would not, however, have had a significant influence on the resulting settlement pattern. By contrast, during the buried pipe tests the stiffness of the structure was shown to be a very significant factor. The positive arching strains induced above the pipe haunches during the 250mm WP PVC-U tests (for example, Figure 7.44 for Test 250WPL1) were shown to be removed by the use of a clay pipe, as the equal settlement contours in Figure 6.12 for Test 120WPL1 demonstrate.

Pender (1980) suggested solutions based on elastic theory to predict stresses and displacements around two main types of loading arrangement (or Load Cases) for buried structures (see Section 2.3). Load case 1 related to a buried structure embedded in an elastic medium subjected to uniform stresses at a distant boundary, typically a C/D ratio greater than 1. This condition is applicable to a buried pipe subjected to a
surface surcharge (as described in this study), or to an existing tunnel subjected to the same loading arrangement. Load Case 2 related to a buried structure that was bored through a pre-stressed soil medium (i.e. a bored tunnel). By adding the change in soil stresses due to the excavation of the tunnel to those of the in-situ soil stresses prior to tunnelling, the stresses applied at the tunnel boundary could be calculating. Gumbel (1981) demonstrated that for all other factors being equal, the response of a buried structure loaded at a distant boundary (Load Case 1) only differed from that loaded at the tunnel interface (Load Case 2) in terms of relative magnitudes of uniform and distortional loading. A practical application of this was demonstrated during the development of the TRL method (see Section 3.2.3), in which a magnification factor was used to convert the uniform load components applied at the tunnel interface (Load Case 2) to a distant boundary solution (Load Case 1).

The comparison of test data outlined above described the behaviour of soil around a buried structure subjected to a high loading at a distant boundary and a buried structure subjected to a soil loading, altered by the excavational process. To assess the elastic theories described by Pender (1980) and hence the relationship between them, it would be necessary to perform a set of tunnel tests using a reduced stiffness lining (i.e. a PVC-U pipe similar to that used for the buried pipe tests) at varying C/D ratios by use of surface stress and the strain response of the lining would need to be measured. The use of a surface stress during the tunnel tests would importantly increase the significance of the tunnel-soil interaction. The initial soil stresses would need to account for the influence of the rigid shield, as the induced negative arching strains increase the stiffness of the soil above the lining crown relative to that adjacent to it, thereby reducing the magnitude of settlement above the crown compared to that with a weaker shield, and conversely altering the settlements at the sides. The change in stress during the withdrawal process would be determined by measuring the soil and tunnel lining displacements as described herein. Adding the incremental to the in-situ stresses would enable the stresses acting on the tunnel interface to be determined. A comparison could then be made between the measured and predicted final structural response after determining the interaction between the lining and soil. A repeat buried pipe test could then be performed using the lining subjected to a surface stress based
on the conversion factors described by Gumbel (1981) and the stress regime applied at the lining. The structural and soil responses from these two test methods could then be compared to assess the elastic theory for a bored tunnel, the conversion relationships between the two test methods, thereby improving the understanding of the behaviour of buried structures.
Figure 7.1 - Comparison of the influence of filling to the Pipe Springline on PVC-U pipe in WPL surrounds (i.e. Step 3 - Step 1)
Figure 7.2 - Comparison of the influence of filling to the Pipe Crown on PVC-U pipe in WPL surrounds (i.e. Step 4 - Step 1)
Figure 7.3 - Comparison of front section internal strain distributions in Tests 250WPL1 and 250WPL1*, Datum Step 1
Figure 7.4 - Comparison of front section external strain distributions in Tests 250WPL1 and 250WPL1*, Datum Step 1
Figure 7.5 - Graph of imposed versus applied load during Tests 250WPL1 and 250WPL1*, Datum Step 1
Figure 7.6  PIPE TEST No 250WPL1  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 7.7  PIPE TEST No 250WPL1  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 7.8  PIPE TEST No 250WPL1*  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 7.9  PIPE TEST No 250WPL1*
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 7.10 - External circumferential pipe wall strain distribution during Test 250PPLA1, Datum Step 1
Figure 7.11  Effect of raising the sidefill to the springline (Step 3), in Test 250WPD1, Datum Step 2
Figure 7.12 - Internal circumferential pipe wall strain distribution during Test 250WPD1, Datum Step 1
Figure 7.13  Effect of raising the sidefill from the springline (Step 3) to the crown (Step 4) in Test 250WPD1
Figure 7.14a - Internal circumferential pipe wall strain distribution during installation of Test 250WPD1, Datum Step 1
Figure 7.14b - Internal circumferential pipe wall strain distribution during static loading of Test 250WPD1, Datum Step 1

- Full, 250mm soil cover (Step 5)
- 50kPa (Step 6)
- 100kPa (Step 15)
- 150kPa (Step 17)
- Unload, 250mm soil cover (Step 18)
VERTICAL MOVEMENT
+ UP  - DOWN

Figure 7.15  PIPE TEST No 250WPDI  STEP 5 - 6
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

Figure 7.16  PIPE TEST No 250WPDI  STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
Figure 7.17 - Graph of imposed versus applied load during Tests 250WPD1 and 250WPD1\(^{\wedge}\), Datum Step 1
Figure 7.18 - Graph of imposed versus applied load under soil loading during Tests 250WPL1, 250WPL1*,
250WPL2, 250WPL1, 250PPL1, 168WPL2.5 and 168WPL3.5, Datum Step 1
Figure 7.19 - Graph of imposed versus applied load during Tests 250WPD1^, 250WPL1*, 250PPL1 and 250WPL2, Datum Step 5
Figure 7.20 - Graph of diametral strain versus applied load during Tests 250WPD1, 250WPL1, 250PPL1 and 250WPL2, Datum Step 5
Figure 7.21a - Soil settlement profiles at crown under 150kPa static surface stress (Step 17) during Tests 250WPL1, 250WPL1*, 250WPD1^*, 250PPL1 and 250PPLA1, Datum Step 5
Figure 7.21b - Soil settlement profiles at invert under 150kPa static surface stress (Step 17) during Tests 250WPL1, 250WPL1*, 250WPD1^, 250PPL1 and 250PPLA1, Datum Step 5
Figure 7.22  Shear strain distribution around Test 250WPDI\textsuperscript{^1} under 150kPa surface stress (Step 17), Datum Step 5

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Figure 7.23  Shear strain distribution around Test 250WPL1* under 150kPa surface stress (Step 17), Datum Step 5
Figure 7.24  Shear strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), Datum Step 5
Figure 7.25 - Graph of internal circumferential strain distributions under 150kPa static surface stress (Step 17) in Tests 250WPDI, 250WPL1, 250WPL2 and 250PPL1, datum Step 5
Figure 7.26 Volumetric strain distribution around Test 250WPD1 under 150kPa surface stress (Step 17), Datum Step 5
-VE DENOTES COMPRESSION

Figure 7.27 Volumetric strain distribution around Test 250WPL1* under 150kPa surface stress (Step 17), Datum Step 5
Figure 7.28  Volumetric strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), Datum Step 5
Figure 7.29  Shear strain distribution around Test 250WPL2 under 70kPa surface stress (Step 7), Datum Step 5.

RHS  +VE DENOTES NEGATIVE ARCHING
LHS  +VE DENOTES POSITIVE ARCHING

100 mm
Figure 7.30  Shear strain distribution around Test 168WPL2.5 under 70kPa surface stress (1st application, Step 7), Datum Step 5
RHS  +VE DENOTES NEGATIVE ARCHING
LHS  +VE DENOTES POSITIVE ARCHING

Figure 7.31  Shear strain distribution around Test 120WPL2.5 under 1" cycle of 70kPa surface stress (Step 7), Datum Step 5
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RHS  +VE DENOTES NEGATIVE ARCHING
LHS  +VE DENOTES POSITIVE ARCHING

Figure 7.33  Shear strain distribution around Test 250WPL2 under 150kPa surface stress (Step 17), Datum Step 5.
Figure 7.34  Volumetric strain distribution around Test 250WPL2 under 150kPa surface stress (Step 17), Datum Step 5.
Figure 7.35  PIPE TEST No 250WPL2  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
Figure 7.36 - Graph of imposed versus applied load during Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
Figure 7.37 - Graph of diametral strain versus applied load during Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
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Figure 7.39 - Internal strain distribution with uniform hoop compression removed (value) under 250mm soil loading plus 50 to 150kPa static surface stress in Test 250WPL4, Step 4 as datum
Figure 7.40 - Internal strain distribution with uniform hoop compression removed (value) under 100 to 150 kPa static surface stress in Test 168WPL2.5, Step 11 datum
Figure 7.41 - Internal strain distribution with uniform hoop compression removed (value) under 100 to 150kPa static surface stress in Test 168WPL3.5, Step 11 datum
RHS +VE DENOTES NEGATIVE ARCHING
LHS +VE DENOTES POSITIVE ARCHING

Figure 7.42 Shear strain distribution around Test 250WPL1 under 50kPa surface stress (Step 6), Datum Step 5
Figure 7.43 Shear strain distribution around Test 250WPL1 under 100kPa surface stress (Step 15), Datum Step 5
Figure 7.44  Shear strain distribution around Test 250WPL1 under 150kPa surface stress (Step 17), Datum Step 5
Figure 7.45  Volumetric strain distribution around Test 250WPL1 under 50kPa surface stress (Step 6), Datum Step 5

-VE DENOTES COMPRESSION
-VE DENOTES COMPRESSION

Figure 7.46  Volumetric strain distribution around Test 250WPL1 under 100kPa surface stress (Step 15), Datum Step 5
Figure 7.47 Volumetric strain distribution around Test 250WPL1 under 150kPa surface stress (Step 17), Datum Step 5
Number of cycles

Figure 7.48 - Graph of imposed load during and after cyclic loading in Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
Figure 7.49 - Graph of diametral strain during and after cyclic loading in Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
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Figure 7.50b - Soil settlement profiles at invert during and after cyclic loading in Test 168WPL2.5, Datum Step 5
Figure 7.51a - Soil settlement profiles at crown due to cyclic loading in Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
Figure 7.51b - Soil settlement profiles at invert due to cyclic loading in Tests 168WPL2.5 and 168WPL3.5, Datum Step 5
Figure 7.52  Volumetric strain distribution around Test 168WPL2.5 under 70kPa surface stress (1st application, Step 7), Datum Step 5
Figure 7.53  PIPE TEST No 168WPL2.5
DESCRIPTION: Removal of Surface Stress, 1st cycle
Figure 7.54  Shear strain distribution around Test 168WPL2.5 on removal of 70kPa surface stress (1st application, Step 8), Datum Step 5
Figure 7.55  Volumetric strain distribution around Test 168WPL2.5 on removal of 70kPa surface stress (Step 8), Datum Step 5
Figure 7.56  Shear strain distribution around Test 168WPL2.5 under 100th cycle of 70kPa surface stress (Step 9) Datum Step 8
Figure 7.57  Shear strain distribution around Test 168WPL2.5 under 500th cycle of 70kPa surface stress (Step 11), Datum Step 8
Figure 7.58 Volumetric strain distribution around Test 168WPL2.5 under 100\textsuperscript{th} cycle of 70kPa surface stress (Step 9), Datum Step 8
Figure 7.59  Volumetric strain distribution around Test 168WPL2.5 under 500th cycle of 70kPa surface stress (Step 11), Datum Step 8
RHS +VE DENOTES NEGATIVE ARCHING
LHS +VE DENOTES POSITIVE ARCHING

Figure 7.60 Shear strain distribution around test 120WPL2.5 under 100th cycle of 70kPa surface stress (Step 9), Datum Step 5
Figure 7.61  Shear strain distribution around test 120WPL2.5 under 500th cycle of 70kPa surface stress (Step 11), Datum Step 5

RHS  +VE DENOTES NEGATIVE ARCHING
LHS  +VE DENOTES POSITIVE ARCHING
-VE DENOTES COMPRESSION

Figure 7.62  Volumetric strain distribution around Test 120WPL2.5 under 100th cycle of 70kPa surface stress (Step 9), Datum Step 5
Figure 7.63 Volumetric strain distribution around Test 120WPL2.5 under 500th cycle of 70kPa surface stress (Step 11), Datum Step 5
VERTICAL MOVEMENT
+ UP - DOWN

Figure 7.64  PIPE TEST No 120WPL2.5  STEP 5 - 9
DESCRIPTION: 70kPa Surface Stress, 100th Cycle
VERTICAL MOVEMENT
  + UP
  - DOWN

Figure 7.65  PIPE TEST No 120WPL2.5
DESCRIPTION: 70kPa Surface Stress, 500th Cycle
VERTICAL MOVEMENT
+ UP - DOWN

Figure 7.66  PIPE TEST No 120WPL2.5
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle
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Figure 7.68 - Values of positive projection fill load coefficient (Cc) from calculated imposed load during cyclic stress (C/D reduced due to glass friction)
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Figure 7.72 - Measured and predicted springline hoop thrust using TRL method and back-calculated $E_s^*$ in Test 250WPDI, Datum Step 5
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Figure 7.75 - Graph of lateral extent of vertical sand movement during tunnel tests (T1 and T2), (Ref: Figure 6.29 for dimensions)
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(Ref: Figure 6.29 for dimensions)
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Figure 7.79  Vertical movements due to withdrawal of 10mm (Steps 1 to 4) in Tunnel Test 1 (C/D=4.5)
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Figure 7.81  Vertical movements due to 10mm pipejack in Test OPJ7 (C/D=4.5) from Chapman, 1992
Table 7.1 Comparison of PVC-U pipe displacements for Well Placed Loose (WPL) sand installations.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>250WPL1</th>
<th>250WPL2</th>
<th>250WPL1*</th>
<th>168WPL2.5</th>
<th>168WPL3.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Step</td>
<td>VD</td>
<td>HD</td>
<td>VDS</td>
<td>HDS</td>
<td>VD</td>
</tr>
<tr>
<td>Bedding to</td>
<td>0</td>
<td>-0.14</td>
<td>0.12</td>
<td>-0.11</td>
<td>0</td>
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<tr>
<td>Haunch (Step 1-2)</td>
<td>0.04</td>
<td>-0.22</td>
<td>0.08</td>
<td>-0.05</td>
<td>0.04</td>
</tr>
<tr>
<td>Bedding to</td>
<td>0.79</td>
<td>0.44</td>
<td>0.11</td>
<td>-0.03</td>
<td>0.79</td>
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<td>Springline (Step 1-3)</td>
<td>-0.60</td>
<td>-0.34</td>
<td>-0.07</td>
<td>-0.14</td>
<td>-0.60</td>
</tr>
<tr>
<td>Bedding to</td>
<td>0.86</td>
<td>1.11</td>
<td>0.21</td>
<td>0</td>
<td>0.86</td>
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<td>Crown (Step 1-4)</td>
<td>-0.50</td>
<td>N/A</td>
<td>0.28</td>
<td>0</td>
<td>-0.50</td>
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<td>VDS</td>
<td>-0.54</td>
<td>-0.57</td>
<td>-0.02</td>
<td>-0.34</td>
<td>-0.54</td>
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<td>HDS</td>
<td>0.57</td>
<td>1.74</td>
<td>0.33</td>
<td>-0.02</td>
<td>0.57</td>
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Note for Table 7.1:-
Bedding (Step 1) is used as datum reading.

Notes for Tables 7.1 to 7.4
VD = Overall vertical pipe deflection (mm).
HD = Overall horizontal pipe deflection (mm).
VDS = Vertical Diametral Strain (%).
HDS = Horizontal Diametral Strain (%).
Positive diametral strain denotes shortening of pipe diameter.
Table 7.2  Comparison of springline circumferential pipe wall strain and VDS at Step 3 for the WPL PVC-U pipe tests, and test 250WPD1* using Step 1 as datum.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Springline circumferential strain (με)</th>
<th>VDS (%)</th>
<th>Comment</th>
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<tbody>
<tr>
<td></td>
<td>Internal (90 and 270°)</td>
<td>External (90 and 270°)</td>
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</tr>
<tr>
<td>250WPL1</td>
<td>151 &amp; 167</td>
<td>-202</td>
<td>External &gt; -0.60</td>
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<td></td>
<td></td>
<td></td>
<td>Internal</td>
</tr>
<tr>
<td>250WPL2</td>
<td>70 &amp; 67</td>
<td>-148 &amp; -162</td>
<td>External &gt; -0.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Internal</td>
</tr>
<tr>
<td>250WPL1*</td>
<td>24 &amp; 42</td>
<td>-33 &amp; -51</td>
<td>External &gt; -0.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Internal</td>
</tr>
<tr>
<td>168WPL2.5</td>
<td>76 &amp; 88</td>
<td>-43 &amp; -62</td>
<td>Internal &gt; -0.14</td>
</tr>
<tr>
<td>168WPL3.5</td>
<td>10 &amp; 47</td>
<td>-30 &amp; -49</td>
<td>Internal &gt; -0.04</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>(Step 4)</td>
</tr>
<tr>
<td>250WPD1*</td>
<td>540 &amp; 552</td>
<td>-202 &amp; -272</td>
<td>Internal &gt; -1.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>External</td>
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Table 7.3  Comparison of diametral strains for tests 250WPL1 and 250WPL1*, using Step 4 (full sidefill) as datum readings.

<table>
<thead>
<tr>
<th>Stage of test</th>
<th>VDS (%)</th>
<th>HDS (%)</th>
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<tr>
<td></td>
<td>250WPL1</td>
<td>250WPL1*</td>
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<tr>
<td>Step 5 - 250mm cover</td>
<td>0.34</td>
<td>-0.01</td>
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<tr>
<td>Step 6 - 50kPa</td>
<td>0.44</td>
<td>0.23</td>
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<tr>
<td>Step 15 - 100kPa</td>
<td>0.58</td>
<td>0.49</td>
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<tr>
<td>Step 17 - 150kPa</td>
<td>0.66</td>
<td>0.54</td>
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See Notes for Table 7.1
Table 7.4 Imposed load, diametral strain and vertical displacement for Tests 168WPL2.5, 168WPL3.5 and 120WPL2.5 during cyclic loading using Step 5 (Full cover) as datum readings.

<table>
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<th>Test Number</th>
<th>Position</th>
<th>Cycle Number</th>
<th>1&lt;sup&gt;st&lt;/sup&gt;</th>
<th>100&lt;sup&gt;th&lt;/sup&gt;</th>
<th>500&lt;sup&gt;th&lt;/sup&gt;</th>
<th>1000&lt;sup&gt;th&lt;/sup&gt;</th>
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<tbody>
<tr>
<td>168WPL2.5</td>
<td>Imposed</td>
<td>90°</td>
<td>-1.929</td>
<td>-2.015</td>
<td>-1.905</td>
<td>N/A</td>
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<td></td>
<td>Load</td>
<td>270°</td>
<td>-2.505</td>
<td>-2.221</td>
<td>-1.880</td>
<td>N/A</td>
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<tr>
<td></td>
<td>(kN/m)</td>
<td>Average</td>
<td>-2.216</td>
<td>-2.116</td>
<td>-1.891</td>
<td>N/A</td>
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<td>Degree of Loading (%)</td>
<td>Average</td>
<td>58</td>
<td>55</td>
<td>49</td>
<td>N/A</td>
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<td>Vertical</td>
<td>Diametral Strain (%)</td>
<td>0.31</td>
<td>0.74</td>
<td>0.87</td>
<td>N/A</td>
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<td>Horizontal</td>
<td>Diametral Strain (%)</td>
<td>-0.14</td>
<td>-0.63</td>
<td>-0.67</td>
<td>N/A</td>
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<tr>
<td>168WPL3.5</td>
<td>Imposed</td>
<td>90°</td>
<td>-2.040</td>
<td>-1.991</td>
<td>-2.144</td>
<td>-1.916</td>
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<td>Load</td>
<td>270°</td>
<td>-2.539</td>
<td>-1.652</td>
<td>-1.208</td>
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<td>(kN/m)</td>
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<td>Degree of Loading (%)</td>
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<td>Vertical</td>
<td>Diametral Strain (%)</td>
<td>0.32</td>
<td>0.75</td>
<td>0.87</td>
<td>1.27</td>
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<td>Horizontal</td>
<td>Diametral Strain (%)</td>
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<td>-0.74</td>
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<td>120WPL2.5</td>
<td>Vertical Displacement (mm)</td>
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See Notes for Table 7.4
<table>
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<tr>
<th>Test Number</th>
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<th>Bedding Constant (kN)</th>
<th>HDS</th>
<th>VDS</th>
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<th>HDS</th>
<th>VDS</th>
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<td>250PPL1</td>
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<td>Test Number</td>
<td>Measurement</td>
<td>Bedding Constant (kN)</td>
<td>HDS</td>
<td>VDS</td>
<td>Bedding Constant (kN)</td>
<td>HDS</td>
<td>VDS</td>
<td>Bedding Constant (kN)</td>
<td>HDS</td>
<td>VDS</td>
</tr>
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<td>250WPL1</td>
<td>Stress</td>
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<td>HDS</td>
<td>VDS</td>
<td>Bedding Constant (kN)</td>
<td>HDS</td>
<td>VDS</td>
<td>Bedding Constant (kN)</td>
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<td>VDS</td>
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<td>250WPL1*</td>
<td>Stress</td>
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<td>Bedding Constant (kN)</td>
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<td>VDS</td>
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<td>168WPL2.5</td>
<td>Stress</td>
<td>70 (1st)</td>
<td>53</td>
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Table 7.5: Back calculated values of E (MPa) from VOS and HDS deflection method readings for Iowa
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<th>250PPLA1</th>
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<th>250WP01*</th>
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*HDS deflection method
CHAPTER EIGHT
8. CONCLUSIONS

The aim of this study was to investigate the behaviour of buried pipes and bored tunnels in sand. This was achieved by undertaking thirteen closely controlled laboratory tests using a glass-sided, 1.5 x 1.5 x 1.0m wide test tank. Eleven buried pipe tests were conducted using two PVC-U pipes and a vitrified clay pipe of differing stiffness installed in an embankment condition and subjected to a static surface stress of up to 150kPa (to simulate the application of an approximate 9m high soil surcharge), or up to 1000 applications of a cyclic surface stress of 70kPa (to simulate traffic loading). The two tunnel tests were performed using a concentric tube arrangement to simulate a 'tail void' loss in which a 194mm diameter steel tube represented the lining. Throughout the installation and subsequent surface loading of the buried pipes and the withdrawal of the outer (shield) tube in the tunnel tests, a set of photographs were taken. These enabled sand particle and buried structure displacements to be determined and additionally for the buried pipe tests, soil strain distributions to be calculated. In addition, strain gauge measurements were recorded during the buried pipe tests to determine the shape of the pipe during the application of applied stresses and allow load acting (or imposed) on the pipe at the springline to be inferred.

The vertical displacement contour plots showed the relative displacement of the sand surround to the structural response of the buried rigid and flexible pipes under static and cyclic surface loading. For the flexible (PVC-U) pipe tests, these plots clearly showed that the pipe deformed relative to the surrounding soil under both static and cyclic surface loading inducing positive arching within the surround. This redistribution of applied stress away from the pipe was also shown by the inferred imposed load measurements at the pipe springline, as indicated by the lower values of imposed, compared to applied, load. The corresponding soil shear strain distributions typically showed a region of positive arching strains extending from between the pipe shoulder and springline regions, extending to form an 'ear' shape in the sidefill material. The reduction of imposed stress on the pipe resulted in the pipe invert settling less than the soil adjacent to it, resulting in the formation of negative arching strains from the region between the pipe haunch and invert. Under static surface loading, the shear strain
distributions were shown to be associated with the pipe deformation pattern, which were in turn associated with the installation procedure. Applying cyclic surface loading resulted in the same general settlement pattern as observed during static loading, but the weaker soil beneath the pipe haunches resulted in soil adjacent to it settling relative to the pipe, thereby inducing more significant negative arching strains.

By contrast, the negative arching settlement pattern was only recorded during the application of cyclic loading to a buried rigid, vitrified clay pipe. As the number of cycles increased, the settlement of the pipe increased at a decreasing rate inducing relative soil settlements as the soil flowed around the pipe into the weaker soil beneath the pipe haunches. The pattern of soil settlement beneath the pipe haunches under cyclic loading mirrored the pattern observed during the flexible, PVC-V, pipe tests. Throughout the load regime, the settlement of the soil above the pipe crown was less than that adjacent to the pipe indicating the occurrence of internal and external soil prisms as described in Marston's (1930) load theory for rigid pipes. The associated shear strain distribution showed the generation of negative strains of up to 15% at the pipe-soil boundary after the application of 500 cycles, forming an approximate triangular shape above the pipe crown. Application of a static surface stress of up to 150kPa was not, however, sufficient to generate measurable negative arching strains around the clay pipe. Throughout the vitrified clay pipe tests the measured strain gauge readings proved unreliable and consequently could not be used to infer imposed load calculations and provide a comparison to the shear strain distributions.

The effect of the installation procedure on the behaviour of the PVC-U pipes was clearly shown in these tests and was assessed by installing the pipes according to Well-Placed (WP) and Poorly-Placed (PP) methods. The aim of the WP surround tests was to ensure good contact existed between the complete circumference of the pipe wall and the surround material by hand feeding material beneath the pipe haunches and raising the sidefill uniformly on either side of the pipe in horizontal layers. By contrast, during the PP surrounded tests the sidefill material was placed either uniformly or asymmetrically in horizontal layers. The absence of hand fed material beneath the pipe haunches resulted in the formation of voids beneath these locations, effectively
reducing the pipe stiffness. This reduction of pipe stiffness was highlighted by a reduction of load imposed on the pipe compared to it being installed in a WP surround. The vertical applied stress was resisted by the bedding solely at the invert causing the pipe to deform to an inverted heart shaped pattern, leading to an internal circumferential strain of $2639\mu\varepsilon$ at the invert and a VDS of 0.63% under a static surface stress of 150kPa. This reduction of imposed load on the pipe was associated with the creation of positive arching strains of up to 8% at the haunches, which formed around the complete circumference of the pipe extending vertically from the pipe shoulders. Mirroring the behaviour of Terzaghi’s (1936) trapdoor experiments and practical tunnels, a region of dilation was evidenced above the pipe crown between the two vertical zones of positive arching strain.

Providing haunch support by installing the pipe in a WP surround reduced the comparable VDS and internal circumferential pipe wall strains (as the degree of distortional bending reduced), but there was an increase in load imposed on the pipe as the stiffness of the pipe was effectively improved. Placed in a loose surround and subjected to static loading the PVC-U pipes deformed according to the heart and elliptical pipe shapes. The associated shear strain distribution showed reduced magnitudes (of up to 2%) and extent of positive shear strains extending from above the pipe haunches in comparison to that observed with the PP tests. Increasing the density of the sidefill by compaction resulted in the most competent soil surround and consequently least structural duress. This was evidenced by the pipe deforming solely by hoop compression resulting in a decrease in imposed load and a decrease in initial magnitude and incremental increase (thus ultimate) pipe deflection, in comparison to the same pipe installed in a WP loose surround. The mobilisation of greater positive arching stresses was, as expected for the dense state, associated with a reduction in the extent of maximum positive shear strains (of 2%), which were observed predominantly adjacent to the pipe springline. This contrasted with the larger extents of positive arching strain from between the pipe haunches and shoulders in the comparable loose surround test.
Increasing the pipe stiffness by using a 168mm diameter (3.5mm wall thickness) rather than a 250mm diameter (2.7mm wall thickness) PVC-U pipe, as expected, increased the degree of load imposed on the pipe under the same applied surface stress. The increase of imposed load on the pipe was associated with a reduction of pipe deflection and a reduction in the positive arching strains pattern observed above the pipe haunches during the weaker PVC-U pipe tests. These results highlighted that the behaviour of buried pipes is clearly influenced by the relative pipe-soil stiffness.

Installing the PVC-U pipes in a WP loose surround condition and increasing the depth of cover from a C/D ratio of 1 to 2 did not significantly alter the behaviour of the pipe. The increased confining stress did, however, lead to a more vertical positive arching strain extending from the pipe shoulders as the springline displaced less laterally into the relatively stiffer surround. The water bag arrangement used in these experiments can only act as an equivalent soil surcharge when it (i.e. the surface) does not interfere with the resulting shear strain distribution. The magnitudes of the shear strains (up to 2%) during these WP loose surround tests indicate that in addition to the pipe, the soil behaved elastically. These observations support the assumptions by Hoeg (1968) and Gumbel (1983) that the C/D ratio must exceed 1 for the elastic theory based upon external stresses applied at distant boundaries to be applicable.

During the PVC-U pipe tests, the increase of vertical deflection and particularly horizontal deflection decreased exponentially as the magnitude of static surface stress increased. A similar pattern was observed during the application of cyclic loading as the increase of deflection decreased exponentially with the number of applications. These relationships indicate that the soil stiffness increases during the application of static and cyclic loading to a buried flexible pipe. The implication of the increase of soil stiffness was evidenced by the measurements of imposed load on the pipe. During increasing static surface loading the imposed load on the pipe increased linearly, indicating a constant degree of loading on the pipe (i.e. a constant degree of applied stress being transferred to the sidefill). By contrast, under the same magnitude of peak cyclic loading the increase of sidefill stiffness reduced the imposed load on the pipe. Subjecting a rigid clay pipe to cyclic loading was shown to increase the settlement of
the sidefill relative to that above the pipe, and of the pipe, indicating that the degree of
loading on the pipe increases.

The results of the buried flexible and rigid pipe tests were shown to correlate
successfully with Marston's load theory under static loading as similar values of relative
settlement were determined from both the vertical displacement and imposed load
measurements. During cyclic loading, however, the reduction of imposed load
recorded during the PVC-U pipe tests was not predicted by the vertical displacement
data. This was due mainly to the greater relative soil to pipe displacements at the pipe
haunches (indicating negative arching) exceeding the relative pipe to soil displacements
(indicating positive arching) at the pipe crown. Correlations between the TRL method
and the measured imposed load at the springline were successful in the WP PVC-U
pipe tests when the pipe deformed approximately to an ellipse and the Free-Slip
condition was assumed. Variations to this occurred when the pipe deformation (thus
the arching pattern) varied significantly from that of an ellipse. This occurred for
example during the PP tests, as the greater positive arching strains extending from the
pipe shoulders reduced the imposed load compared to that predicted. A further
variation was observed during the application of static loading after the cyclic load
regime had been applied to the pipe. The increase of negative strains induced by the
settlement of the soil into the regions beneath the pipe haunches and shoulders
increased the imposed load compared to that predicted, thus approaching the No-Slip
prediction.

The variation of pipe deflection during both static and cyclic loading was evidenced
from the back-calculated values of soil modulus ($E'$ and $E_5$) using the Iowa and TRL
deflection methods. The value of soil modulus was therefore shown to be dependent
upon the relative density of the surround, the installation procedure and applied stress
regime. Measuring the response of a pipe after cyclic loading showed that a higher
value of soil modulus should be used. Comparing cyclic to static loading cases at the
same stress level indicated that a lower value of soil modulus should be used due to the
irrecoverable pipe deformations caused by the cyclic loading.
The use of a tail void to simulate tunnel construction resulted in a narrow width convergent settlement pattern typical of a practical tunnel constructed in granular material. The only variation was the recording of maximum settlements adjacent to the lining shoulders, rather than above the crown. This was due to the use of a concentric volume loss arrangement inducing settlements beneath the pipe springline.

Comparisons between the settlement patterns indicated that, as expected, those recorded during the buried pipe tests were associated with the application of a high surface stress whilst those above the tunnel where confined to a narrow width above the tunnel crown, induced by the volume loss. As Section 8.1 describes in greater detail, further tunnelling testing is recommended with the additional measurement of lining wall strains. This would enable the response of the lining and the surrounding soil to be compared to the elastic theory for tunnels.

8.1 FUTURE WORK

The main recommendation for future work is that the results of the buried pipe tests be compared with additional theoretical methods than described herein. This principally relates to the prediction of both buried pipe and surround behaviour under static and cyclic surface loading by finite element methods. In addition to assessing the influence of the loading regime, use of these numerical methods would allow the influence of cover depth, pipe stiffness and installation procedure on the pipe and soil measurements presented herein to be investigated.

Furthering the experimental approach adopted in this study it is recommended that the accuracy and speed of the digitising method be improved by utilising the scanning method to create a digital image. This method requires the use of reflective markers that would need to be placed adjacent to the glass face whilst the level of the sidefill was raised in horizontal layers (i.e. simulating construction practice).

Future testing should attempt to cause failure of the pipe-soil system by applying greater surface stresses than achievable with the current equipment to a very low stiffness pipe. The behaviour of the progressively buckled pipe and the potential soil failure planes could then by compared with elastic (i.e. TRL) and plastic (i.e. Marston)
pipe design methods and buckling theories. The range of factors influencing buried pipe design could also be expanded from those described herein. The long-term behaviour of a pipe-soil system and the influence of other granular materials could be investigated with minimal future experimental development. Assessing the influence of a submerged pipe beneath a fluctuating water level would, however, require development of the seals used at either end of the pipe. The buried pipe tests described herein were placed in an embankment condition, whilst more commonly for highway drainage applications they are placed in a narrow trench. The influence of the trench geometry and in-situ soil on the behaviour of pipe and surround trench sand could be assessed in both the short and long-terms. Using a comparatively large test tank it would be recommended that a well-graded sand be used as the in-situ soil, dug out to form a trench by wooden supports, that are withdrawn during/after the placement of the pipe surround material. Adopting the approach described by Rogers (1985) clay bricks could be used to form a cohesive in-situ soil.

It is recommended that the tunnel equipment be further developed to ensure that the lining wall strains be measurable, thus allowing imposed loads to be inferred. This would entail the use of a lower stiffness pipe, whilst ensuring that permanent axial strains were not induced during the placement of the axial restraint force required when the outer tube is withdrawn. The distribution of volume loss could be adapted by lowering the level of the lining tube to induce a single maximum settlement convergent pattern above the lining. The aim of future tests with this modified arrangement would be to relate the soil movement pattern (thus arching strains) to the imposed load and consequently to the establishment of an ultimate soil load on the lining shown by the trapdoor experiments. After assessing the influence of the stiffness of the shield on the in-situ soil stresses the results could be compared to elastic theories for bored tunnels. A more thorough comparison could then be made to assess the proportional relationships for uniform and distortional stresses applied to a buried structure-soil system at the tunnel interface and at a distant boundary.
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APPENDIX 1
ERROR IN DISPLACEMENT MEASUREMENT
DUE TO GLASS REFRACTION

Chapman (1992) provided the derivation and the formulae stated below for the calculation of the likely error in the measurement of the displacements due to glass refraction.

From Figure A.1 the following relationships are obtained.

\[
\tan \alpha_1 = \frac{x_1}{z} \quad (A 1.1)
\]
\[
\tan \alpha_2 = \frac{x_2}{z} \quad (A 1.2)
\]
\[
\tan \alpha_1 = \frac{y_1}{(t + z)} \quad (A 1.3)
\]
\[
\tan \alpha_2 = \frac{y_2}{(t + z)} \quad (A 1.4)
\]
\[
\sin \beta_1 = \sin \alpha_1 / M \quad (A 1.5)
\]
\[
\sin \beta_2 = \sin \alpha_2 / M \quad (A 1.6)
\]
\[
\tan \beta_1 = \frac{(y_1 - x_1) / t - d_1 / t}{(A 1.7)}
\]
\[
\tan \beta_2 = \frac{(y_2 - x_2) / t - d_2 / t}{(A 1.8)}
\]

where M is the refractive index. The error in the observed displacement, \(W\), is equal to \((d_2 - d_1)\), therefore taking equation A 1.7 from A 1.8 and rearranging gives:

\[
(d_2 - d_1) = (y_2 - x_2) - (y_1 - x_1) - (\tan \beta_2 - \tan \beta_1) t
\]

Using typical values for the photographs used in this project, a table is presented below for the errors due to glass refraction based on the equations stated above.

Values:

\[
M = 1.4 \quad \text{(refractive index of toughened glass)}
\]
\[
t = 30 \text{ mm}
\]
\[
z = 700 \text{ mm}
\]

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<th>Example 1</th>
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<td>Error (mm)</td>
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Schematic representation of the error in particle movements due to glass refraction

Fig. A.1
Potts (1977) outlined the derivation to determine moment and thrust from strain gauge readings. This is used as the basis for the description of the derivation stated below:

From simple beam theory

\[ \sigma = \frac{My}{I} \]  \hspace{1cm} (A 2.1)

where \( \sigma \) is the normal stress on the cross section at a distance \( y \) from the point of zero strain and \( I \) is the second moment of area. Hooke's law is

\[ \sigma = \epsilon \cdot E \]  \hspace{1cm} (A 2.2)

where \( \epsilon \) is strain and \( E \) is the Young's modulus of the material.

For thin walled tubes it is assumed that a linear distribution of strain occurs across the wall thickness and that simple beam theory holds. Therefore from the following diagram,

\[ \frac{\epsilon}{y} = \frac{\epsilon_0}{A} \]  \hspace{1cm} (A 2.3)

\[ \frac{\epsilon_0}{(A + t)} = \frac{\epsilon_1}{A} \]  \hspace{1cm} (A 2.4)

Rearranging equation A 2.4 for \( A \) gives:

\[ A = \frac{\epsilon_1 t}{(\epsilon_0 - \epsilon_1)} \]  \hspace{1cm} (A2.5)

Substituting equation A 2.5 into equation A 2.3 and rearranging gives:

\[ \frac{\epsilon}{y} = \frac{(\epsilon_0 - \epsilon_1)}{t} \]  \hspace{1cm} (A 2.6)

Substituting equation A 2.2 into equation A 2.1, and rearranging gives:
\[
\frac{\varepsilon}{y} = \frac{M}{EI} \tag{A 2.7}
\]

As \(I = \frac{t^4}{12}\), substituting equation A 2.6 into equation A 2.7 and rearranging for \(M\) yields:

\[
M = \frac{(\varepsilon_o - \varepsilon_i)Et^2}{12} \tag{A 2.8}
\]

The hoop stress is defined as:-

\[
\sigma_h = \frac{\sigma_o + \sigma_i}{2} \tag{A 2.9}
\]

where \(\sigma_o\) and \(\sigma_i\) are the direct stresses in the extreme fibres of the structure wall. Using equation A 2.2, the hoop stress, \(\sigma_h\), and hoop thrust, \(T\) become:-

\[
\sigma_h = \frac{(\varepsilon_o + \varepsilon_i)E}{2} \tag{A 2.10}
\]

\[
T = \frac{(\varepsilon_o + \varepsilon_i)Et}{2} \tag{A 2.11}
\]

Equations A 2.8 and A 2.11 express the Moment (\(M\)) and Thrust (\(T\)) in the structure wall in terms of strains recorded by the strain gauges attached to the inside and outside of the structure wall, the material constants and wall thickness. The derivation assumes thin-walled theory and plane stress conditions apply. To convert equations A 2.8 and A 2.11 for plane strain conditions they must be multiplied by:-

\[
\frac{1}{(1 - v^2)} \tag{A 2.12}
\]
APPENDIX 3  
CALCULATIONS FOR STRESS AND STRAIN FOR A CIRCULAR PIPE UNDER A LINE LOAD

Roark's formulas for calculating stress and strain (Young, 1989) can be used to calculate the circumferential stress, bending moment and deflection of a circular pipe subjected to a single line load. The equations defined below are stated for either the crown or springline positions, denoted as Point A and B, respectively:

**Circumferential Stress**

At springline

\[ \sigma_0 = \frac{N}{A} + \frac{Md}{I} \]  
(A 3.1)

**Circumferential Strain**

At springline

\[ \varepsilon_0 = \frac{\sigma_0}{E} \]  
(A 3.2)

**Bending Moment**

At crown

\[ M_A = 0.3183WR \]  
(A 3.3)

At springline

\[ M_B = -(0.5 - 0.3183k_2)WR \]  
(A 3.4)

**Vertical Deflection**

\[ D_v = -0.1488 \frac{WR^3}{EI} \] (Decrease)  
(A 3.5)

**Horizontal Deflection**

\[ D_H = 0.1366 \frac{WR^3}{EI} \] (Increase)  
(A 3.6)

where:

- \( W \) = Line load (kN/m)
- \( N = \frac{W}{2} \)
- \( R \) = radius to the centroid of the cross-section denoted.
- \( d \) = distance from the neutral axis to the extreme fibre.

When the ratio of \( R/t \) exceeds eight, the difference in the radius to the extreme fibres becomes negligible, and a linear variation in strain across the structure wall can be assumed. Adopting this thin wall theory, it can assumed that the neutral axis passes through the centroid, so that \( d = R - r \), where \( r \) is the radial distance from centre of pipe to extreme fibre.

\( A \) = cross-sectional area of the wall, equal to the wall thickness, \( t \).
\[
\begin{align*}
M & = \text{internal moments at denoted points} \\
E & = \text{modulus of Elasticity} \\
I & = \text{area moment of inertia of the ring cross section about the centroid axis, equal to } t^3/12. \\
k_2 & = 1 \text{ for thin wall theory.} \\
D_V, D_H & = \text{changes in the vertical and horizontal diameters, respectively}
\end{align*}
\]

From equations A 3.5 and A 3.6, it is found that \(D_H = 0.913D_V\), and that this is consistent with the resulting elliptical shape.
APPENDIX 4

CIRCUMFERENTIAL AND AXIAL WALL STRAINS MEASURED DURING BURIED PIPE TESTS
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External pipe wall strains (values x 10^-6)

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## Internal pipe wall strains (values x 10^-6)

| Orientation | Location | Step | Haunch  | 50 mm | 100 mm | 150 mm | 200 mm | Crown  | 50 mm | 100 mm | 150 mm | 200 mm | 250 mm | Static | 50 kPa | Static | 100 kPa | Static | 150 kPa | 250 mm | Static | 250 mm |
|-------------|----------|------|---------|-------|--------|--------|--------|--------|-------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Hoop 0      |          | 2    | -53     | -7    | -88    | -31    | -593   | -916   | -948   | -945   | -960   | -942   | -955   | -1046  | -1123  | -1152  | -1075  |        |        |
| Hoop 45     |          | 3    | 8       | 17    | -11    | -110   | 31     | 49     | 126    | 81     | 57     | 45     | 38     | -104   | -240   | -263   | -69    |        |        |
| Hoop 90     |          | 4    | 36      | 108   | 263    | 540    | 854    | 948    | 855    | 893    | 886    | 873    | 607    | 383    | 337    | 684    |        |        |
| Hoop 180    |          | 6    | 31      | 233   | 334    | 354    | 330    | 163    | 166    | 158    | 148    | 151    | 160    | 8      | -158   | -203   | 60     |        |        |
| Hoop 225    |          | 7    | 68      | 124   | 310    | 552    | 928    | 1051   | 1015   | 1006   | 983    | 977    | 979    | 743    | 557    | 526    | 806    |        |        |
| Hoop 315    |          |      |         |       |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |

## External pipe wall strains (values x 10^-6)

| Orientation | Location | Step | Haunch  | 50 mm | 100 mm | 150 mm | 200 mm | Crown  | 50 mm | 100 mm | 150 mm | 200 mm | 250 mm | Static | 50 kPa | Static | 100 kPa | Static | 150 kPa | 250 mm | Static | 250 mm |
|-------------|----------|------|---------|-------|--------|--------|--------|--------|-------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Hoop 0      |          | 2    | 106     | 184   | 252    | 144    | 780    | 1097   | 1126  | 1095   | 1087   | 1054   | 1078   | 1035   | 975    | 910    | 1060   |        |        |
| Hoop 45     |          | 3    | -23     | 103   | 133    | 227    | 87     | -13    | -122   | -59    | -48    | -30    | -22    | -49    | -104   | -177   | 3      |        |        |
| Hoop 135    |          | 5    | 174     | 784   | 1265   | 1469   | 1658   | 1604   | 1549   | 1543   | 1525   | 1529   | 1556   | 1429   | 1279   | 1117   | 1371   |        |        |
| Hoop 225    |          | 7    | -20     | 30    | 35     | 53     | 77     | 74     | 55     | 63     | 60     | 64     | 76     | 100    | 98     | 99     | 105    |        |        |
| Hoop 270    |          | 8    | -18     | 38    | 48     | 55     | 89     | 85     | 77     | 84     | 84     | 89     | 106    | 148    | 153    | 162    | 137    |        |        |
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## VITRIFIED CLAY PIPE

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### External Pipe Wall Strains (values x 10^-6)

| Step | Orientation | Location | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |
| Haunch | Springline | Crown | 50 mm Cover | 120 mm Cover | 160 mm Cover | 215 mm Cover | 300 mm Cover | Static | 1st | 2nd | 3rd | 4th | 5th | 6th | 7th | 8th | 9th | 10th | 11th | 12th | 13th | 14th | 15th | 16th | 17th | 18th |
| Hoop | 0 | 2 | 4 | 7 | 11 | 10 | 12 | 7 | 5 | -12 | -16 | 4 | -23 | -5 | -48 | -40 | -11 | -2 | -13 | -12 | -27 | -8 |
| Hoop | 45 | 3 | 5 | 3 | 3 | 0 | -1 | -6 | -9 | -10 | -15 | -12 | -10 | -7 | -27 | -33 | 3 | 2 | 0 | -2 | -4 | 2 |
| Hoop | 90 | 4 | 5 | 4 | 6 | 5 | 4 | 0 | 1 | 28 | 31 | 6 | 27 | 8 | 7 | -11 | 27 | 10 | 32 | 39 | 45 | 11 |
| Hoop | 135 | 5 | 7 | 5 | 9 | 12 | 19 | 20 | 22 | 83 | 104 | 43 | 104 | 25 | 83 | -39 | 97 | -8 | 128 | 171 | 204 | 33 |
| Hoop | 180 | 5 | 5 | 4 | 5 | 2 | 1 | -5 | -9 | -42 | -54 | -11 | -22 | 5 | -27 | -24 | 2 | 15 | -6 | -49 | -28 | 22 |
| Hoop | 225 | 6 | 10 | 7 | 7 | 7 | 10 | 8 | 11 | 31 | 36 | 8 | 31 | -1 | 6 | -36 | 18 | -30 | -33 | 46 | 57 | -12 |
| Axial | 90 | 4 | 6 | 4 | 7 | 6 | 8 | 5 | 8 | 20 | 22 | 8 | 26 | 12 | 0 | -14 | 26 | 15 | 28 | 36 | 40 | 22 |
| Axial | 270 | 5 | 5 | 8 | 10 | 11 | 6 | 9 | 14 | 14 | 10 | 15 | 14 | -10 | -16 | 25 | 24 | 28 | 30 | 32 | 31 |
APPENDIX 5

APPLIED LOAD, IMPOSED LOAD AND DEGREE OF LOADING MEASURED DURING BURIED PIPE TESTS
Test 250WPD1, 250mm diameter PVC-U WPD C/D=1

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Springline</td>
<td>0</td>
<td>-0.875</td>
<td>N/A</td>
<td>-0.875</td>
</tr>
<tr>
<td>4</td>
<td>Crown</td>
<td>0</td>
<td>-0.771</td>
<td>N/A</td>
<td>-0.771</td>
</tr>
<tr>
<td>5</td>
<td>250mm, Full Cover</td>
<td>-0.374</td>
<td>-0.472</td>
<td>N/A</td>
<td>-0.472</td>
</tr>
<tr>
<td>6</td>
<td>50s</td>
<td>-4.789</td>
<td>-1.823</td>
<td>N/A</td>
<td>-1.823</td>
</tr>
<tr>
<td>15</td>
<td>100s</td>
<td>-9.204</td>
<td>-3.043</td>
<td>N/A</td>
<td>-3.043</td>
</tr>
<tr>
<td>17</td>
<td>150s</td>
<td>-13.619</td>
<td>-4.109</td>
<td>N/A</td>
<td>-4.109</td>
</tr>
<tr>
<td>18</td>
<td>250mm, Full Cover</td>
<td>-0.374</td>
<td>-1.655</td>
<td>N/A</td>
<td>-1.655</td>
</tr>
</tbody>
</table>

Notes for Appendix 5
- The applied load was calculated using the method described in Section 6.1.3, using measured in-situ unit weight of dense and loose state Leighton Buzzard sand (see Section 5.5) of 16 and 15kN/m\(^3\) respectively.
- The degree of loading was calculated by dividing the average imposed load at both springlines (or a single springline for the first three tests) by the applied load at a single springline (i.e. applied load/2), using Step 4 as datum readings.
**Test 250WPL1, 250mm diameter PVC-U WPL C/D=1**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Springline</td>
<td>0</td>
<td>-0.192</td>
<td>N/A</td>
<td>-0.192</td>
</tr>
<tr>
<td>4</td>
<td>Crown</td>
<td>0</td>
<td>-0.843</td>
<td>N/A</td>
<td>-0.843</td>
</tr>
<tr>
<td>5</td>
<td>250mm, Full cover</td>
<td>-0.350</td>
<td>-1.289</td>
<td>N/A</td>
<td>-1.289</td>
</tr>
<tr>
<td>6</td>
<td>50s</td>
<td>-4.765</td>
<td>-1.777</td>
<td>N/A</td>
<td>-1.777</td>
</tr>
<tr>
<td>15</td>
<td>100s</td>
<td>-9.180</td>
<td>-2.806</td>
<td>N/A</td>
<td>-2.806</td>
</tr>
<tr>
<td>17</td>
<td>150s</td>
<td>-13.596</td>
<td>-4.237</td>
<td>N/A</td>
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<tr>
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<td>-0.350</td>
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### Test 250PPL1. 250mm diameter PVC-U PPL C/D=1

<table>
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<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Crown</td>
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<td>-0.950</td>
<td>N/A</td>
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<tr>
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<td>250mm, Full cover</td>
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<td>-1.987</td>
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<tr>
<td>15</td>
<td>100s</td>
<td>-9.180</td>
<td>-2.632</td>
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<td>150s</td>
<td>-13.596</td>
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<td>N/A</td>
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</tr>
<tr>
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<td>250mm, Full Cover</td>
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### Test 120WPL1, 120mm diameter Clay WPL C/D=1

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<th>Average Degree of Loading (%)</th>
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<tbody>
<tr>
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<tr>
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<td>Crown</td>
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<td>4.596, 7.222</td>
<td>5.909</td>
<td>N/A</td>
</tr>
<tr>
<td>5</td>
<td>120mm, Full Cover</td>
<td>-0.080</td>
<td>-2.701, -5.314</td>
<td>-4.007</td>
<td>11937</td>
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<tr>
<td>6</td>
<td>50s</td>
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<td>-5.407, -12.315</td>
<td>-8.861</td>
<td>634</td>
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<tr>
<td>15</td>
<td>100s</td>
<td>-4.575</td>
<td>-6.632, -12.662</td>
<td>-9.647</td>
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<td>150s</td>
<td>-6.821</td>
<td>-8.267, -7.056</td>
<td>-7.661</td>
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<td>4.824, -0.029</td>
<td>2.398</td>
<td>4227</td>
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## Test 250WPD1^, 250mm diameter PVC-U WP D C/D=1

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<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
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<tbody>
<tr>
<td></td>
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<tr>
<td>≈3</td>
<td>25mm above Springline</td>
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<td>-0.399</td>
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<td>-0.526</td>
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<td>-0.491</td>
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<tr>
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<td>-0.526</td>
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<td>200mm Cover</td>
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<td>-0.440</td>
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<td>Imposed Load (kN/m) - Plane Stress</td>
<td>Average Imposed Load (kN/m)</td>
<td>Average Degree of Loading (%)</td>
</tr>
<tr>
<td>------</td>
<td>---------------------------------</td>
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<td>----------------------------------</td>
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</tr>
<tr>
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<td>-0.555</td>
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<tr>
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<td>-1.135</td>
<td>87</td>
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<tr>
<td>7</td>
<td>70s</td>
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<td>-3.117</td>
<td>41</td>
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<tr>
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<td>-6.002</td>
<td>-5.695</td>
<td>41</td>
</tr>
<tr>
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<td>-0.350</td>
<td>-2.312</td>
<td>-1.840</td>
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</table>
Test 250WPL1*, 250mm diameter PVC-U WPL C/D=1

Front Section

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>3</td>
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<td>-0.036</td>
<td>-0.036</td>
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<td>-0.135</td>
<td>-0.149</td>
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</tr>
<tr>
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</tr>
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<td>-2.913</td>
<td>-2.633</td>
</tr>
<tr>
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<td>100s</td>
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<td>-3.322</td>
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<td>-3.723</td>
</tr>
<tr>
<td>16</td>
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<td>-3.914</td>
<td>-4.819</td>
<td>-4.366</td>
</tr>
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<td>-0.350</td>
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<td>-0.965</td>
<td>-0.925</td>
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Test 250WPL1*, 250mm diameter PVC-U WPL C/D=1

Mid-Way Section

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Springline</td>
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<td>-0.021</td>
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<tr>
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<td>Crown</td>
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<tr>
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<td>-6.542</td>
</tr>
<tr>
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<td>-0.406</td>
<td>-0.633</td>
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<td>-1.312</td>
</tr>
<tr>
<td>Step</td>
<td>Description</td>
<td>Applied Load / 2 (kN/m)</td>
<td>Imposed Load (kN/m) - Plane Stress</td>
<td>Average Imposed Load (kN/m)</td>
<td>Average Degree of Loading (%)</td>
</tr>
<tr>
<td>------</td>
<td>------------------------</td>
<td>-------------------------</td>
<td>-----------------------------------</td>
<td>-----------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Springline</td>
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<td>0.143</td>
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</tr>
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<td>0.033</td>
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<td>0.001</td>
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</tr>
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<td>-0.161</td>
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Test 168WPL2.5, 168mm diameter PVC-U WPL C/D=2.5 (Contd.)

<table>
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<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
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<td>-3.247</td>
<td>-3.127</td>
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<tr>
<td>16</td>
<td>125s</td>
<td>-7.256</td>
<td>-3.531</td>
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</tr>
<tr>
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<td>-8.630</td>
<td>-4.035</td>
<td>-4.486</td>
<td>-4.260</td>
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<tr>
<td>18</td>
<td>250mm, Full cover</td>
<td>-0.381</td>
<td>-1.210</td>
<td>-0.892</td>
<td>-1.051</td>
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## Test 168WPL3.5, 168mm diameter PVC-U WPL C/D=3.5

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td>90 Springline</td>
<td>270 Springline</td>
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</tr>
<tr>
<td>3</td>
<td>Springline</td>
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<td>-0.098</td>
<td>-0.009</td>
<td>-0.053</td>
</tr>
<tr>
<td>4</td>
<td>Crown</td>
<td>0</td>
<td>-0.147</td>
<td>0.040</td>
<td>-0.054</td>
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<tr>
<td></td>
<td>70mm Cover</td>
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<td>-0.103</td>
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<td>180mm Cover</td>
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<td>-0.257</td>
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<tr>
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<td>250mm Cover</td>
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<td>-0.341</td>
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<td>340mm Cover</td>
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<td>-0.725</td>
<td>-0.485</td>
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<tr>
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<td>415mm Cover</td>
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<td>470mm Cover</td>
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<td>-0.482</td>
<td>-0.633</td>
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<td>5</td>
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<td>-0.851</td>
<td>-0.559</td>
<td>-0.705</td>
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<tr>
<td>6</td>
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<td>-2.443</td>
<td>-2.519</td>
<td>-2.481</td>
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<tr>
<td>7</td>
<td>70s</td>
<td>-4.086</td>
<td>-2.891</td>
<td>-3.098</td>
<td>-2.995</td>
</tr>
<tr>
<td>8</td>
<td>590mm, Full Cover</td>
<td>-0.515</td>
<td>-2.060</td>
<td>-1.938</td>
<td>-1.999</td>
</tr>
<tr>
<td>9</td>
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<td>-2.842</td>
<td>-2.211</td>
<td>-2.523</td>
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### Test 168WPL3.5, 168mm diameter PVC-U WPL C/D=3.5 (Contd.)

<table>
<thead>
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<th>Step</th>
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<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
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</thead>
<tbody>
<tr>
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<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>590mm, Full Cover</td>
<td>-0.515</td>
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<td>-1.284</td>
<td>-1.727</td>
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<td>11</td>
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<td>-2.995</td>
<td>-1.767</td>
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<tr>
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<td>-2.470</td>
<td>-1.156</td>
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<tr>
<td>13</td>
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<td>-2.767</td>
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<td>-2.067</td>
</tr>
<tr>
<td>14</td>
<td>590mm, Full Cover</td>
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<td>-2.329</td>
<td>-0.761</td>
<td>-1.545</td>
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<tr>
<td>15</td>
<td>100s</td>
<td>-5.616</td>
<td>-3.791</td>
<td>-2.282</td>
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<tr>
<td>16</td>
<td>125s</td>
<td>-6.891</td>
<td>-4.327</td>
<td>-3.075</td>
<td>-3.701</td>
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<tr>
<td>17</td>
<td>150s</td>
<td>-8.167</td>
<td>-4.871</td>
<td>-3.685</td>
<td>-4.278</td>
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<tr>
<td>18</td>
<td>590mm, Full Cover</td>
<td>-0.515</td>
<td>-2.713</td>
<td>-0.966</td>
<td>-1.840</td>
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</table>
Test 120WPL2.5, 120mm diameter Clay WPL C/D=3.5

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Load / 2 (kN/m)</th>
<th>Applied Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Springline</td>
<td>0</td>
<td>4.889 6.076</td>
<td>5.482</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>Crown</td>
<td>0</td>
<td>2.972 3.455</td>
<td>3.214</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>50mm Cover</td>
<td>-0.035</td>
<td>3.600 3.396</td>
<td>3.498</td>
<td>-808</td>
</tr>
<tr>
<td></td>
<td>120mm Cover</td>
<td>-0.083</td>
<td>3.215 3.148</td>
<td>3.182</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>160mm Cover</td>
<td>-0.110</td>
<td>2.573 3.673</td>
<td>3.123</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>215mm Cover</td>
<td>-0.146</td>
<td>-0.444 1.394</td>
<td>0.475</td>
<td>1879</td>
</tr>
<tr>
<td>5</td>
<td>300mm, Full Cover</td>
<td>-0.200</td>
<td>-0.818 1.472</td>
<td>0.327</td>
<td>1446</td>
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<tr>
<td>6</td>
<td>50s</td>
<td>-2.272</td>
<td>1.405 2.575</td>
<td>1.990</td>
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<tr>
<td>7</td>
<td>70s</td>
<td>-3.101</td>
<td>-0.023 2.362</td>
<td>1.170</td>
<td>66</td>
</tr>
<tr>
<td>8</td>
<td>300mm, Full Cover</td>
<td>-0.200</td>
<td>1.076 -0.461</td>
<td>0.308</td>
<td>1456</td>
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<tr>
<td>9</td>
<td>70c100</td>
<td>-3.101</td>
<td>3.115 2.342</td>
<td>2.729</td>
<td>16</td>
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<tr>
<td>10</td>
<td>300mm, Full Cover</td>
<td>-0.200</td>
<td>4.313 -3.040</td>
<td>0.637</td>
<td>1291</td>
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</table>
### Test 120WPL2.5, 120mm diameter Clay WPL C/D=3.5 (Contd.)

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Applied Load / 2 (kN/m)</th>
<th>Imposed Load (kN/m) - Plane Stress</th>
<th>Average Imposed Load (kN/m)</th>
<th>Average Degree of Loading (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 Springline</td>
<td>270 Springline</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>70c500</td>
<td>-3.101</td>
<td>-11.237</td>
<td>-12.165</td>
<td>-11.701</td>
</tr>
<tr>
<td>12</td>
<td>300mm, Full Cover</td>
<td>-0.200</td>
<td>-13.801</td>
<td>-24.368</td>
<td>-19.084</td>
</tr>
<tr>
<td>13</td>
<td>70c1000</td>
<td>-3.101</td>
<td>2.906</td>
<td>-3.811</td>
<td>-0.453</td>
</tr>
<tr>
<td>14</td>
<td>300mm, Full Cover</td>
<td>-0.200</td>
<td>2.615</td>
<td>-16.294</td>
<td>-6.839</td>
</tr>
<tr>
<td>15</td>
<td>100s</td>
<td>-4.344</td>
<td>2.210</td>
<td>-0.023</td>
<td>1.094</td>
</tr>
<tr>
<td>16</td>
<td>125s</td>
<td>-5.381</td>
<td>1.367</td>
<td>2.080</td>
<td>1.723</td>
</tr>
<tr>
<td>17</td>
<td>150s</td>
<td>-6.417</td>
<td>2.469</td>
<td>4.481</td>
<td>3.475</td>
</tr>
<tr>
<td>18</td>
<td>590mm, Full Cover</td>
<td>-0.200</td>
<td>5.527</td>
<td>-8.166</td>
<td>-1.312</td>
</tr>
</tbody>
</table>
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 250PPLA1
DESCRIPTON: 50kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250PLA1
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)

HORIZONTAL MOVEMENT
+ RIGHT - LEFT

STEP 5 - 15

100 mm
HORIZONTAL MOVEMENT
+ RIGHT   - LEFT

PIPE TEST No 250PPLA1
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250PPLA1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT

+ UP  - DOWN

100 mm

PIPE TEST No 250PPLA1

DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP       - DOWN

PIPE TEST No 250PPLA1       STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT - LEFT 100 mm

PIPE TEST No 250WPD1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 250WPD1
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT — LEFT

PIPE TEST No 250WPD1
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPD1
STEP 5 - 6
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP    - DOWN

PIPE TEST No 250WPD1
STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP \quad - DOWN

PIPE TEST No 250WPDI
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPL1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)

HORIZONTAL MOVEMENT
+ RIGHT    - LEFT
PIPE TEST No 250WPL1

DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPL1

DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPL1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPL1
STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP         - DOWN

PIPE TEST No 250WPL1
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250PPL1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250PPL1

DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250PPL1
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250PPL1

DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
  + UP - DOWN

PIPE TEST No 250PPL1
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)

STEP 5 - 15
PIPE TEST No. 250PPL1
DESCRIPTION: 150 kPa Static Surface Stress (No Cycle)

VERTICAL MOVEMENT
+ UP
- DOWN

STEP 5 - 17

100 mm
PIPE TEST No 120WPL1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
PIPE TEST No 120WPL1
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT
- LEFT

PIPE TEST No 120WPL1
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP        - DOWN

PIPE TEST No 120WPL1
DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)

STEP 5 - 6
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 120WPL1
STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 120WPL1
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPD1

DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 250WP1^ STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPD1

DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPD1

DESCRIPTION: 50kPa Static Surface Stress (No Cyclic)

STEP 5 - 6
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPD1^ STEP 5 - 15
DESCRIPTION: 100kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPDI^  STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 250WPL2
STEP 5 - 7
DESCRIPTION: 70kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPL2
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 250WPL2
STEP 5 - 7
DESCRIPTION: 70kPa Static Surface Stress (No Cyclic)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 250WPL2
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)

STEP 5 - 17
PIPE TEST No 250WPL1*
STEP 5 - 17
DESCRIPTION: 150kPa Static Surface Stress (No Cyclic)
PIPE TEST No 168WPL2.5

DESCRIPTION: 50kPa Static Surface Stress (Before Cyclic)
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 168WPL2.5

DESCRIPTION: 70kPa Static Surface Stress, 1st cycle
DESCRIPTION: Removal of Surface Stress, 1st cycle
PIPE TEST No 168WPL2.5

DESCRIPTION: 70kPa Surface Stress, 100th cycle
PIPE TEST No 168WPL2.5

DESCRIPTION: Removal of Surface Stress (after 100, 70kPa Cycles)
PIPE TEST No 168WPL2.5

DESCRIPTION: 70kPa Surface Stress, 500th cycle
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 168WPL2.5

DESCRIPTION: Removal of Surface Stress (after 500, 70kPa Cycles)
PIPE TEST No 168WPL2.5

STEP 8 - 15

DESCRIPTION: 100kPa Static Surface Stress (after 500, 70kPa Cycles)
PIPE TEST No 168WPL2.5

DESCRIPTION: 125kPa Static Surface Stress (after 500, 70kPa Cycles)
PIPE TEST No 168WPL2.5

DESCRIPTION: 150kPa Static Surface Stress (after 500, 70kPa Cycles)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 168WPL2.5

DESCRIPTION: 70kPa Static Surface Stress, 1st cycle
PIPE TEST No 168WPL2.5

DESCRIPTION: Removal of Surface Stress, 1st cycle
PIPE TEST No 168WPL2.5
DESCRIPTION: 70kPa Surface Stress, 100th cycle

STEP 8 - 9

VERTICAL MOVEMENT
+ UP - DOWN

100 mm
VERTICAL MOVEMENT
+ UP  - DOWN

PIPE TEST No 168WPL2.5

DESCRIPTION: Removal of Surface Stress (after 100, 70kPa Cycles)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 168WPL2.5
DESCRIPTION: 70kPa Surface Stress, 500th cycle
DESCRIPTION: Removal of Surface Stress (after 500, 70kPa Cycles)
VERTICAL MOVEMENT
+ UP       - DOWN

PIPE TEST No 168WPL2.5

DESCRIPTION: 100kPa Static Surface Stress (after 500, 70kPa Cycles)
PIPE TEST No 168WPL2.5

DESCRIPTION: 125kPa Static Surface Stress (after 500, 70kPa Cycles)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 168WPL2.5
DESCRIPTION: 150kPa Static Surface Stress (after 500, 70kPa Cycles)
PIPE TEST No 168WPL3.5
DESCRIPTION: 70kPa Surface Stress, 1st Cycle
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 168WPL3.5
STEP 5 - 9
DESCRIPTION: 70kPa Surface Stress, 100th Cycle
PIPE TEST No 168WPL3.5

DESCRIPTION: 70kPa Surface Stress, 500th Cycle
PIPE TEST No 168WPL3.5

DESCRIPTION: 70kPa Surface Stress, 1000th Cycle
DESCRIPTION: 150kPa Static Surface Stress (After 1000 Cycles)
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 168WPL3.5
DESCRIPTION: 70kPa Surface Stress, 1st Cycle
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 168WPL3.5
STEP 5 - 9
DESCRIPTION: 70kPa Surface Stress, 100th Cycle
VERTICAL MOVEMENT
+ UP   - DOWN

PIPE TEST No 168WPL3.5
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle

STEP 5 - 13
PIPE TEST No 168WPL3.5

DESCRIPTION: 150kPa Static Surface Stress (After 1000 Cycles)
HORIZONTAL MOVEMENT
+ RIGHT  - LEFT

PIPE TEST No 120WPL2.5
DESCRIPTION: 70kPa Surface Stress, 1st Cycle
DESCRIPTION: 70kPa Surface Stress, 100th Cycle

HORIZONTAL MOVEMENT
+ RIGHT - LEFT

PIPE TEST No 120WPL2.5
STEP 5 - 9
PIPE TEST No 120WPL2.5

DESCRIPTION: 70kPa Surface Stress, 500th Cycle
HORIZONTAL MOVEMENT
+ RIGHT    - LEFT

PIPE TEST No 120WPL2.5
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle
PIPE TEST No 120WPL2.5
DESCRIPTION: 150kPa Static Surface Stress (After 1000 Cycles)
PIPE TEST No 120WPL2.5

DESCRIPTION: 70kPa Surface Stress, 1st Cycle
VERTICAL MOVEMENT

+ UP  - DOWN

PIPE TEST No 120WPL2.5

DESCRIPTION: 70kPa Surface Stress, 100th Cycle

STEP 5 - 9
VERTICAL MOVEMENT
+ UP        - DOWN

PIPE TEST No 120WPL2.5

DESCRIPTION: 70kPa Surface Stress, 500th Cycle
VERTICAL MOVEMENT
+ UP - DOWN

PIPE TEST No 120WPL2.5
DESCRIPTION: 70kPa Surface Stress, 1000th Cycle

STEP 5 - 13
VERTICAL MOVEMENT
+ UP  - DOWN

PIPE TEST No 120WPL2.5
DESCRIPTION: 150kPa Static Surface Stress (After 1000 Cycles)
APPENDIX 7  BURIED PIPE DISPLACEMENTS
## Test 250PPLA, 250mm diameter PVC-U PPLA C/D=1

### Pipe Displacements

<table>
<thead>
<tr>
<th>Steps</th>
<th>Vertical Displacements</th>
<th>Vertical Diametral Strain (%)</th>
<th>Horizontal Displacements</th>
<th>Horizontal Diametral Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Crown (mm)</td>
<td>Invert (mm)</td>
<td>Overall (mm)</td>
<td>LHS (mm)</td>
</tr>
<tr>
<td>1-4</td>
<td>0.66</td>
<td>0.66</td>
<td>-0.58</td>
<td>-0.58</td>
</tr>
<tr>
<td>4-5</td>
<td>-0.54</td>
<td>0.12</td>
<td>-0.34</td>
<td>-0.92</td>
</tr>
<tr>
<td>5-6</td>
<td>-1.75</td>
<td>-1.63</td>
<td>-0.83</td>
<td>-1.75</td>
</tr>
<tr>
<td>5-15</td>
<td>-2.73</td>
<td>-2.61</td>
<td>-1.75</td>
<td>-2.67</td>
</tr>
<tr>
<td>5-17</td>
<td>-3.55</td>
<td>-3.43</td>
<td>-2.10</td>
<td>-3.02</td>
</tr>
<tr>
<td>5-18</td>
<td>-2.93</td>
<td>-2.83</td>
<td>-1.37</td>
<td>-2.29</td>
</tr>
<tr>
<td>17-18</td>
<td>0.60</td>
<td>-2.83</td>
<td>0.73</td>
<td>-2.29</td>
</tr>
</tbody>
</table>

### Notes for Appendix 7

A positive diametral strain denotes a decrease in pipe diameter.
### Test 250WPDI, 250mm diameter PVC-U WPD C/D=1 Pipe Displacement

<table>
<thead>
<tr>
<th>Steps</th>
<th>Vertical Displacements</th>
<th></th>
<th>Vertical Diametral Strain (%)</th>
<th></th>
<th>Horizontal Displacements</th>
<th></th>
<th>Horizontal Diametral Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Crown (mm)</td>
<td>Invert (mm)</td>
<td>Overall (mm)</td>
<td>Increment / Σ</td>
<td>Increment / Σ</td>
<td>LHS (mm)</td>
</tr>
<tr>
<td>1-2</td>
<td>-0.39 -0.39</td>
<td>-0.14</td>
<td>-0.14</td>
<td>-0.27</td>
<td>0.10</td>
<td>0.10</td>
<td>-0.72</td>
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Test 250PPL1, 250mm diameter PVC-U PPL C/D=1
**Test 120WPL1, 120mm diameter Clay WPL C/D=1 Pipe Displacements**

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### Test 250WPΔ1^<sup>c</sup>, 250mm diameter PVC-U WPD C/D=1 Pipe Displacements

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<th>Invert (mm) Increment / Σ</th>
<th>Overall (mm)</th>
<th>Vertical Displacements</th>
<th>LHS (mm) Increment / Σ</th>
<th>RHS (mm) Increment / Σ</th>
<th>Overall (mm)</th>
<th>Horizontal Displacements</th>
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### Test 250WPD1, 250mm diameter PVC-U WPD C/D=1

Pipe Displacements (Contd.)

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<td>Increment / Σ</td>
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**Test 250WPL1*, 250mm diameter PVC-U WPL C/D=1**

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### Pipe Displacements

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Note: The values in the table represent increments in millimeters for the specified measurements.
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<td>Overall (mm)</td>
<td>Increment / Σ</td>
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<td>-0.19</td>
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Test 168WPL2.5, 168mm diameter PVC-U WPL C/D=2.5 ——— Pipe Displacements (Contd.)
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<tr>
<th>Steps</th>
<th>Vertical Displacements</th>
<th>Vertical Diametral Strain (%)</th>
<th>Horizontal Displacements</th>
<th>Horizontal Diametral Strain (%)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Increment / Σ</td>
<td>Crown (mm)</td>
<td>Invert (mm)</td>
<td>Overall (mm)</td>
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Test 168WPL.3.5, 168mm diameter PVC-U WPL C/D=3.5  

Pipe Displacements

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<th>Vertical Strain (%)</th>
<th>Diametral LHS (mm) Increment / Σ</th>
<th>Diametral RHS (mm) Increment / Σ</th>
<th>Diametral Overall (mm) Increment / Σ</th>
<th>Diametral Strain (%)</th>
<th>Horizontal Invert (mm) Increment / Σ</th>
<th>Horizontal Overall (mm) Increment / Σ</th>
<th>Horizontal Strain (%)</th>
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<td>0.54</td>
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17-18 corresponds to the final step in the test sequence.
**Test 120WPL2.5, 120mm diameter Clay WPL C/D=2.5**  
Pipe Displacements

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<th>Horizontal Increment (mm)</th>
<th>Horizontal Σ</th>
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</tr>
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<td>0</td>
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APPENDIX 8
DESCRIPTON OF COMPUTER PROGRAM TO DETERMINE SOIL STRAINS

Section 6.1.8 described that a computer program was developed to calculate soil shear and volumetric strains. Roscoe et al (1963) outlined a method for calculating strains in soils based upon the displacement of discrete points. The notation used by Roscoe et al is shown below and considers the displacements of four points with components of displacement u and v in the x and y directions respectively. The displacements are assumed to be small compared to the diagonal lengths, I_x and I_y.

\[ S_{Ex} = \frac{(u_1 - u_3)}{I_x}; \]
\[ S_{Ey} = \frac{(v_2 - v_4)}{I_y}; \]
The volumetric strain, \( \delta v \) is given by:-
\[ \delta v = \delta e_x + \delta e_y; \]
Defining shear strain (\( \delta e_{xy} \)) as a reduction in the angle YOX gives:-
\[ \delta e_{xy} = \frac{(v_1 - v_3)}{I_x} + \frac{(u_2 - u_4)}{I_y}; \]
The notation used during this project was the same as that described above for the horizontal displacements but was the reverse in the vertical direction. The resulting notation used during this computer program for the determination of a strain between
a selected point (point 1) and points to the right and above it (points 2 and 3 respectively) is:

![Diagram showing points 1, 2, 3, and vectors](image)

Points 2 and 3 used to calculate soil strains at Point 1
Lengths 1 - 2 and 1 - 3 are positive

The four resulting components of strain are given by:

\[ \frac{\Delta u}{\Delta x} = \frac{u_2 - u_1}{l_x} \quad (A 8.5) \]
\[ \frac{\Delta v}{\Delta x} = \frac{v_1 - v_2}{l_x} \quad (A 8.6) \]
\[ \frac{\Delta u}{\Delta y} = \frac{u_3 - u_2}{l_y} \quad (A 8.7) \]
\[ \frac{\Delta v}{\Delta y} = \frac{v_3 - v_1}{l_y} \quad (A 8.8) \]

When the selected point (point 1) is compared to one to its left or below it (points 2 and 3 respectively), the numerical order of the calculating of relative displacement is reversed with respect to those stated above in order to maintain the definition of strain, leading to:

\[ \frac{\Delta u}{\Delta x} = \frac{u_1 - u_2}{l_x} \quad (A 8.9) \]
\[ \frac{\Delta v}{\Delta x} = \frac{v_2 - v_1}{l_x} \quad (A 8.10) \]
\[ \frac{\Delta u}{\Delta y} = \frac{u_2 - u_1}{l_y} \quad (A 8.11) \]
\[ \frac{\Delta v}{\Delta y} = \frac{v_3 - v_2}{l_y} \quad (A 8.12) \]

From these components of strain the volumetric, \( \delta V \), and shear strain \( \delta \varepsilon_{xy} \) are found at each point from:

\[ \delta V = \frac{\Delta u}{\Delta x} + \frac{\Delta v}{\Delta y} \quad (A 8.13) \]

and

\[ \delta \varepsilon_{xy} = \frac{\Delta v}{\Delta x} + \frac{\Delta u}{\Delta y} \quad (A 8.14) \]

Applying this notation results in a consistent definition of volumetric strain throughout the soil medium of an extension being positive, whereas the sign of shear strain is reversed between either side of the vertical plane to produce a mirror imaged pattern. Whilst this notation complicates the presentation of the shear strain...
distribution it acts as a check for the computer program. For this reason, on the LHS of the vertical pipe axis the occurrence of positive and negative arching strains is shown by positive and negative signs, whilst on the RHS the notation of the shear strains is reversed.

The philosophy behind the computer program is to determine these four components of strain for each digitised point. For each point, a search is conducted using the remaining digitised points to find the nearest point with a magnitude of relative settlement (du or dv) greater than 0.5mm in both the x and y planes. This value of relative settlement was selected with respect to the precision of the digitising method and the need to identify accurately a significant change in displacement. In addition to the criteria of relative settlement, the sought point is not considered if it is the same as the selected point or if the two points are both located on the pipe wall.

With these conditions, the search itself involves selecting each digitised point and, for example, in calculating the components of strain with an \(l_x\) term, by finding if this point is either to the left or right of the vertical plane. To ensure that the change in displacement is in the same plane the y co-ordinate of the sought point must lie within ±25mm of that of point 1. With these criteria the nearest point is sought lying a defined distance of \(l_x\) away from the selected point to ensure that it is large relative to the change in displacement. In the case of no points being found, for example at the outer edges of the measured points, the search is repeated in the reverse direction using the opposite equations.

An alternative approach was initially considered in which a strain was calculated between a point and its nearest neighbour, in the same plane, based merely upon a minimum common distance of \(l_x\) and \(l_y\). The problem with this approach was its susceptibility to the inaccuracies of the measuring technique. This is highlighted by considering the calculation of a 1% strain using a common minimum value of \(l_x\) and \(l_y\) of 15mm. A change in displacement of only 0.15mm is required which is below the precision of a displacement with a confidence interval of 95% of ±0.39mm. As the
comparison outlined below demonstrates, increasing the common value of $l_x$ and $l_y$ results in a loss of precision.

To check the precision and accuracy of the computer program calculations of shear and volumetric strain were determined manually from the vertical displacement contour plot during Test 250PPLA1, Step 5-17. Due to a lack of clarity, the strain contributions from the horizontal displacements have been omitted. This test was chosen due to the rapid variations in contours, particularly beneath the pipe haunches. The only differences between the approach of the manual method compared to that of the computer program was the calculation of gradients in the desired plane, so removing the ±25mm criteria, and use of random values of $l_x$ or $l_y$ to represent the gradient between two contour lines.

Three trials of the program were undertaken to determine the minimum common value of $l_x$ and $l_y$. This exercise therefore examined the precision of the program. Figures A.8.1 to A.8.3 show the shear strain for common values of $l_x$ and $l_y$ of 25, 15 and 10mm respectively and Figures A.8.4 to A.8.6 the corresponding volumetric strain pattern for common values of $l_x$ and $l_y$ of 25, 15 and 10mm respectively. These each show that the same distributions of 0 and 1% shear and volumetric strain were defined by each of the three criteria. With the extents of strain being consistently identified, the effect of decreasing the common value of $l_x$ and $l_y$ was seen to increase the precision of the strain distributions within these boundaries. This is highlighted in Figures A.8.1 and A.8.2 by examining the 2% shear strain distribution extending from the RHS pipe shoulder to a distance of 100mm above the pipe crown for the 15mm criteria compared to a distance of 40mm with the 25mm criteria. Decreasing the criteria further to 10mm is seen (Figure A.8.3) to result in similar distributions, but leads to a loss of clarity.

To assess the accuracy of the distribution based upon a common minimum length of $l_x$ and $l_y$ of 15mm, Table A.8 shows the values of shear and volumetric strain calculated manually and by the computer method at a number of random points. The first eleven points listed in the table are located on the pipe wall and so have been compared
directly. As sand grains do not necessarily exist at the co-ordinates of the remaining points, the values stated from the computer program have been measured from the contour plots of shear and volumetric strain.

Comparing the two sets of values it is clear that there is good agreement as the values of strain typically differ by ±0.5%. Increased correlation between the results of the two methods is shown when the manual calculation of strain is based upon a distance greater than 15mm, thereby exceeding the common minimum lengths $l_x$ and $l_y$. The only major discrepancies between the results occurred when the manual calculation of strain was based on a length less than 15mm, for example at point (100,200). Decreasing the minimum common lengths of $l_x$ and $l_y$ would resolve this problem, but as described above, would result in a loss of clarity in defining the overall strain patterns. In view of the rarity of these discrepancies between the two methods, the soil strain distributions presented in this study are based upon a minimum common length of $l_x$ and $l_y$ of 15mm.
Table A.8 Comparison of manual and computer calculated values of shear and volumetric strain based upon vertical displacements only, for Test 250PPLA1, Step 5-17

<table>
<thead>
<tr>
<th>Co-ordinates</th>
<th>Manual Calculations</th>
<th>Computer calculations (Min. $l_x$ &amp; $l_y = 15\text{mm}$)</th>
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Table A.8 (Contd)

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<td>36</td>
<td>-0.4</td>
<td>120</td>
<td>1.0</td>
</tr>
<tr>
<td>200</td>
<td>400</td>
<td>1.0</td>
<td>50</td>
<td>0.5</td>
<td>100</td>
<td>1.0</td>
</tr>
<tr>
<td>250</td>
<td>400</td>
<td>-0.5</td>
<td>94</td>
<td>0</td>
<td>110</td>
<td>0</td>
</tr>
<tr>
<td>300</td>
<td>400</td>
<td>-0.5</td>
<td>94</td>
<td>1.7</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>350</td>
<td>400</td>
<td>-0.8</td>
<td>62</td>
<td>0.5</td>
<td>110</td>
<td>-1.5</td>
</tr>
</tbody>
</table>
Figure A8.1  Shear strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 25mm
RHS +VE DENOTES NEGATIVE ARCHING
LHS +VE DENOTES POSITIVE ARCHING

Figure A8.2 Shear strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 15mm
Figure A8.3  Shear strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 10mm
-VE DENOTES COMPRESSION

Figure A8.4  Volumetric strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 25mm
-VE DENOTES COMPRESSION

Figure A8.5 Volumetric strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 15mm.
Figure A8.6 Volumetric strain distribution around Test 250PPL1 under 150kPa surface stress (Step 17), using common length of 10mm
APPENDIX 9

HORIZONTAL AND VERTICAL DISPLACEMENT CONTOUR PLOTS DURING TUNNEL TESTS
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

TUNNEL TEST No 1 (C/D=4.5)

WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 0 - 4mm
TUNNEL TEST No 1 (C/D=4.5) STEP 2 - 3
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 4 - 7mm

HORIZONTAL MOVEMENT
+ RIGHT - LEFT

100 mm
TUNNEL TEST No 1 (C/D=4.5)  
STEP 3 - 4  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 7 - 10mm
TUNNEL TEST No 1 (C/D=4.5)  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 10 - 15mm
TUNNEL TEST No 1 (C/D=4.5)
STEP 5 - 6
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 15 - 20mm
HORIZONTAL MOVEMENT
+ RIGHT  - LEFT

TUNNEL TEST No 1 (C/D=4.5)  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 20 - 25mm
TUNNEL TEST No 1 (C/D=4.5)  
STEP 7 - 8  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 25 - 29mm
TUNNEL TEST No 1 (C/D=4.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 29 - 32mm
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

TUNNEL TEST No 1 (C/D=4.5) STEP 9 - 10
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 32 - 37mm
TUNNEL TEST No 1 (C/D=4.5)

WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 37 - 54mm
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

TUNNEL TEST No 1 (C/D=4.5)
STEP 11 - 12
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 54 - 72mm
TUNNEL TEST No 1 (C/D=4.5)

WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 72 - 91mm
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 1 (C/D=4.5)  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 0 - 4mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 1 (C/D=4.5)  STEP 2 - 3
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 4 - 7mm
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 1 (C/D=4.5)
STEP 3 - 4
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 7 - 10mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 1 (C/D=4.5)
STEP 4-5
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 10 - 15mm
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 1 (C/D=4.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 15 - 20mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 1 (C/D=4.5)
STEP 6 - 7
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 20 - 25mm
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 1 (C/D=4.5)

WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 25 - 29mm
VERTICAL MOVEMENT
+ UP        - DOWN

TUNNEL TEST No 1 (C/D=4.5)               STEP 8 - 9
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 29 - 32mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 1 (C/D=4.5)
STEP 9 - 10
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 32 - 37mm
TUNNEL TEST No 1 (C/D=4.5)  
STEP 10 - 11  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 37 - 54mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 1 (C/D=4.5) STEP 11 - 12
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 54 - 72mm
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 1 (C/D=4.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 72 - 91mm
HORIZONTAL MOVEMENT
+ RIGHT     - LEFT

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 0 - 4mm
HORIZONTAL MOVEMENT
+ RIGHT  - LEFT

TUNNEL TEST No 2 (C/D=2.5)
STEP 2 - 3
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 4 - 7mm
TUNNEL TEST No 2 (C/D=2.5)  
STEP 3 - 4  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 7 - 10mm
TUNNEL TEST No 2 (C/D=2.5)  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 10 - 15mm
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

TUNNEL TEST No 2 (C/D=2.5)  
STEP 5 - 6
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 15 - 20mm
TUNNEL TEST No 2 (C/D=2.5) STEP 6-7
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 20 - 25mm
HORIZONTAL MOVEMENT
+ RIGHT       - LEFT

TUNNEL TEST No 2 (C/D=2.5)  STEP 7 - 8
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 25 - 29mm
TUNNEL TEST No 2 (C/D=2.5)  
STEP 8-9  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 29 - 32mm
HORIZONTAL MOVEMENT
+ RIGHT - LEFT

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 32 - 37mm

STEP 9 - 10
TUNNEL TEST No 2 (C/D=2.5) 
STEP 10 - 11 
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 37 - 54mm
HORIZONTAL MOVEMENT
+ RIGHT    - LEFT

TUNNEL TEST No 2 (C/D=2.5)  
STEP 11 - 12
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 54 - 72mm
HORIZONTAL MOVEMENT  
+ RIGHT - LEFT  

TUNNEL TEST No 2 (C/D=2.5)  
STEP 12 - 13  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 72 - 91mm
HORIZONTAL MOVEMENT
+ RIGHT
- LEFT

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 91 - 120mm

STEP 13 - 14
HORIZONTAL MOVEMENT
+ RIGHT
- LEFT

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 120 - 150

STEP 14 - 15
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 0 - 4mm

STEP 1 - 2
VERTICAL MOVEMENT
+ UP  - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 4 - 7mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5) STEP 3 - 4
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 7 - 10mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 10 - 15mm
VERTICAL MOVEMENT
+ UP          - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 15 - 20mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5)
STEP 6 - 7
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 20 - 25mm
VERTICAL MOVEMENT
+ UP     - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 25 - 29mm
TUNNEL TEST No 2 (C/D=2.5)  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 29 - 32mm
VERTICAL MOVEMENT
+ UP
- DOWN

100 mm

TUNNEL TEST No 2 (C/D=2.5)
STEP 9-10
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 32 - 37mm
VERTICAL MOVEMENT
+ UP   - DOWN

TUNNEL TEST No 2 (C/D=2.5)
STEP 10 - 11
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 37 - 54mm
TUNNEL TEST No 2 (C/D=2.5)  
STEP 11 - 12  
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 54 - 72mm
VERTICAL MOVEMENT
+ UP    - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 72 - 91mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5)
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 91 - 120mm
VERTICAL MOVEMENT
+ UP - DOWN

TUNNEL TEST No 2 (C/D=2.5)  STEP 14 - 15
WITHDRAWAL OF 245mm DIAMETER OUTER TUBE: 120 - 150mm