Abstract

Soil nailing is a method of earth reinforcement used to create excavations and steepen existing slopes. It has been developed during the past two decades and so is a relatively new technique: adopted design methods are generally based on limit equilibrium analyses. Such design methods are adequate but their efficiency is uncertain due to a lack of understanding of the complex behaviour that occurs at the interface between the soil and the nails and in the soil in the immediate vicinity of the nails.

A fundamental study has been carried out to investigate the processes that occur along the interface of a soil-nail used to form an excavation. The main emphasis of the work has been experimental element testing; numerical and microfabric analyses have also been undertaken as part of the study.

Triaxial apparatus were designed for performing single element tests with a model nail in clay and sands, allowing simplified boundary conditions to be imposed to simulate those occurring in practice. Numerical analyses were performed prior to the final selection of model nail and sample dimensions to assess the stress conditions within the sample. Different numerical techniques were used to model the interface boundary.

Instrumented model nails were developed to examine the stress conditions at the interface during the experimental work. The final prototype nail is able to measure axial force and radial stress at three points along its length using strain gauged mechanisms.

A series of tests has been performed using instrumented and dummy nails in kaolin. Some of the tests were stopped at different stages and thin-sections of the clay prepared to study the microfabric at the interface. Relationships between changes in stress and microfabric during the tests are investigated. Results are also compared to the numerical analyses carried out prior to final equipment design.

Series of tests were also performed on three sands representing fine, medium and coarse grain sizes at loose, medium dense and dense states. The sands were tested dry and parallel reference tests with no nail were also carried out for comparison. The results are discussed in terms of stress-strain behaviour and stiffness, with particular emphasis on the effects of grain size and relative density on the particulate nature of the sands.

The clay and the sand tests made possible the measurement of the magnitudes and the development of interface shear stress and radial stress along the length of the nail. They have also provided evidence of phenomena such as rigid-body inclusion effects, arching and restrained dilation. Very high interface shear stresses have been observed in both sand and clay at the free end of the nail which represents conditions in the vicinity of the face of an excavation.

The results are discussed in relation to current practice, and suggestions are made regarding correlations with pull-out tests, construction and aspects of analysis and design.
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Glossary of terms

α  slope of the tangent to creep curves from controlled force pull-out tests at time, \( t = 60 \)  
α'  interface sliding factor (essentially the same as \( f_s' \) )  
γ  soil unit weight  
δ  displacement vector  
\( Δδ_d' \)  incremental displacement resulting from dilation  
\( Δσ_{d'} \)  incremental dilational stress  
\( Δσ_{app} \)  increment of applied radial stress (at sample boundary)  
\( Δσ_{int} \)  increment of radial stress acting on the nail-soil interface  
\( Δσ_{nail} \)  increment of radial stress acting on nail  
\( Δσ_{soil} \)  increment of radial stress acting on soil  
\( Δw_{ave} \)  average axial displacement at the top of the sample  
\( Δw_{core} \)  approximate axial movements at the top of the core of soil immediately surrounding the nail  
\( δ_{int} \)  deformation induced at the nail-soil interface  
\( δ_{nail} \)  radial displacement of nail  
\( δ_{soil} \)  radial displacement of soil  
\( Δ_{soil} \)  soil displacement relative to the nail  
\( δW \)  incremental work expended during shearing  
\( δ' \)  angle of interface shearing resistance  
\( δ'_{cv} \)  angle of interface shearing resistance at constant volume  
\( δ'_{max} \)  maximum value of angle of interface shearing resistance  
\( ε_a \)  axial strain  
\( ε_a^* \)  axial strain measured in nailed sample  
\( ε_p \)  volumetric strain  
\( ε_q \)  shear strain  
\( n \)  stress ratio, \( q/p' \)  
\( n_n \)  stress ratio at degree \( n \) of shearing  
\( λ \)  characteristic of the system of four expressions defining the roots of the equation with units of \((\text{length})^1 - 1/λ_n \) denoted \( l_n \) in this study, has units of length and is referred to as the characteristic length  
\( λ_p \)  non-dimensional pull-out factor dependent on the type of reinforcement - HA68/94  
\( µ \)  frictional constant  
\( µ' \)  apparent coefficient of friction induced by restrained dilation  
\( ν \)  Poisson’s ratio  
\( ρ_d \)  dry density of soil  
\( ρ_w \)  bulk density of water  
\( σ \)  domain of admissible stress - in homogenisation method
Σ quantity representing all admissible macroscopic states of stress, for the intermittent stress -
used in homogenisation method

σₘₐₜ circumferential stress in r, θ, z coordinates
σₙₐ ′ soil resistance per unit area
σₙₐ ′ₚₚₚ ultimate bearing capacity
σₙₐ ′ component of normal stress induced by restrained dilation
σₙₐ ′ in situ effective stress acting laterally on the nail and parallel to the excavated face
σₙₐ normal stress acting on shearbox sample (effective as sample dry)
σₙₐ ′₂Critical initial normal stress (re restrained dilation)
σₙₐ ′ normal stress (normal to the reinforcement)
σₙₐ ′ interface radial stress measured on instrumented nail
σₙₐ ′ₚₚₚₚ radial stress applied at boundary of sample (re arching)
σₙₐ ′ radial stress in r, θ, z coordinates
σₙₐ ′₂ vertical effective overburden pressure
σₙₐ, σₙₐ and σₙₐ components of total stress in x, y, z directions
σₙₐ limiting or yield axial stress
σₙₐ ′ effective stress
τ shear stress
τₙₐ interface shear stress
τₙₐ shear stress allowing for additional component from restrained dilation
φₚₚ angle of interparticle friction
φₘₚₚ angle of interface resistance
φₚₚ ′angle of internal shearing resistance in extension
φₚₚ ′ the internal angle of shearing resistance of the soil
φₚₚ ′ₚₚₚₚ angle of interparticle friction at constant volume
φₚₚ ′ₚₚₚₚ and cₚₚₚₚ as usually defined but where the subscripts int and des relate to interface and
design values
φₚₚ ′ₚₚₚₚ an apparent externally mobilised angle of friction acting on a horizontal plane
φₚₚ ′ₚₚₚₚ maximum angle of interparticle friction
χ reinforcement density
ψ the angle of dilation
B bulk modulus
C constant relating Vₘₚₚₚₚ, Mₘₚₚₚₚ and lₘₚₚₚₚ which varies depending on the type of analysis used
D borehole diameter for grouted nails or equivalent diameter for driven nails
Dₘ relative density
e voids ratio
EI flexural rigidity of the pile
Eₚₚₚₚ equivalent Young’s modulus at interface
eₘₚₚₚₚ maximum value of voids ratio
eₘₚₚₚₚ minimum value of voids ratio
$E_{\text{m}}$, Young's modulus of nail
$E_s$, elastic spring modulus representing the soil stiffness
$E_{\text{sec}}$, secant Young's modulus
$F$, global factor of safety
$F_{\text{ax}}$, axial force measured in the instrumented nail
$f_b$, bond coefficient
$G$, shear modulus
$G_{\text{hom}}$, domain representing the macroscopic bounding surface - homogenisation method
$G_s$, specific gravity
$G_s$ and $G_t$ functions representing the characteristics of the soil and reinforcements which describe their bounding surfaces - homogenisation method
$HRS$, Ham River Sand
$k_g$, initial part of the load-displacement curve (i.e. initial tangent modulus)
$K_d$, ratio of incremental dilational stress $\Delta \sigma_d'$ to the corresponding displacement $\Delta \delta_d$
$K_0$, coefficient of earth pressure at rest
$K_a$, earth pressure coefficient in active state
$K_t$, coefficient of lateral earth pressure
$K_p$, earth pressure coefficient in passive state
$k_r$, modulus of subgrade reaction
$L$, total length of each nail
$L_0$, the anchored length of the nail
$l_0$, distance to which the reinforcement length must extend to satisfy equilibrium
$L_s$, length of reinforcement which extends beyond the critical failure surface
$l_c$, characteristic length
$l_s$, length between points of maximum moment in a bar, referred to as the shear width
$M$, stress ratio at critical state
$M_p$, fully plastic moment for a bar in pure bending
$N_q$, $N_q$ and $N_q$, bearing capacity factors
$P_{\text{des}}$, design pull-out force which relates to each layer rather than each nail
$PI$, plasticity index
$p_l$, limiting pressure, as measured in a pressuremeter test
$p''$, mean effective stress for case with nail in sample
$p_u$, ultimate bearing capacity of the soil
$p'$, mean effective stress $(\sigma, + \sigma, + \sigma,)/3$
$q$, deviator stress $(\sigma, - \sigma,)$
$q''$, deviator stress for case with nail in sample
$Q$, single load-related parameter (extreme value represented by $Q_{\text{hom}}$ - homogenisation method)
$q_s$, skin friction
$r$, width of the shear surface (re restrained dilation)
$r_p$ pore pressure ratio
$S_h$ and $S_v$ horizontal and vertical spacing of nails
$T$ axial force in the element (field conditions)
$T_{\text{max}}$ maximum force in nail (field conditions)
$T_C$ critical creep tension ($n.b. T_C$ often used in French literature for $V$)
$T_C'$ nail force at the intersection of the two straight portions of the creep curve
$T_G$ the elastic limiting force of the reinforcement
$T_L$ maximum pull-out force at the head of the nail
$T_{TE}$ estimated limiting pull-out force (introduced for controlled force tests)
$T_{\text{lim}}$ limiting axial force
$T_0$ mechanism - limiting mechanism requiring no reinforcement (HA68/94 design method)
$T_{ob}$ mechanism - $T_0$-mechanism considered when selecting the length of the lowest nail (i.e. $T_0$ at the base)
$T_p$ fully plastic axial force for a bar in pure tension
$u$ pore pressure
$V$ sample volume
$V$ shear force across a section
$V_{\text{lim}}$ limiting shear force
$V_o$ the shear force in the nail at the position of the slip surface
$W$ resultant of the soil reaction on one side of the slip surface
$W^{\text{nal}}$ work expended in shearing nailed sample
$W^{ref} - W^{\text{nal}}$ work difference
$W^{ref}$ work expended in shearing reference sample
$y$ lateral deflection of the pile
$y$ axial movement of nail during pull-out test
$y$ displacement of top cap of shearbox
$y_l$ limiting lateral deflection of a pile
$y_o$ $y$ at the head of the nail or pile
$z$ length along the length of a pile
CHAPTER 1

Introduction to the research subject

For millennia man has improved the condition of the ground by reinforcing it: stone layers were placed to increase surface ground strength and load capacity; wooden staves and piles were driven to transmit loads to deeper strata; mats of wicker and reed were placed to provide drained strengthened layers to cross wetlands. The first known attempts to improve the lateral strength of an earth structure date back to the Sumerians in Mesopotamia around 2100 BC. They constructed ziggurats, high platforms on which they built their temples, from many courses of bricks made from sun-baked clay. Their earlier structures tended to spread and settle as the underlying soft alluvial ground yielded. This problem was overcome by incorporating thick layers of woven reeds between every six to eight courses. Thus the horizontal thrust that tended to split the mass of the ziggurat was taken by these mats (see Kerisel, 1987 for further details); the structures remained intact. Further improvements were made by placing the mats within thin layers of sand which acted as drains inducing suctions in the bricks, thus further holding them together.

During subsequent centuries, similar systems to that of the Sumerians have been applied to overcome difficulties of earthwork construction. Two examples that Jones (1985) mentions in his summary of the historical development of reinforced earth structures are the Great Wall of China (c. 200 BC) and a Roman reinforced earth wharf in London (c. 100 AD). Most of the applications since, have been for military earthworks, consisting typically of alternating layers of timber and earthfill.

In the 1960s the construction method that became known as Reinforced Earth was introduced and patented (Terre Armée: see Vidal, 1966). In this system, embankments and retaining walls are constructed by placing layers of selected fill with reinforcement between them. Initially the inclusions were metal strips placed horizontally and perpendicular to the sides of the structure, more recently these have been largely superseded by grids or meshes of synthetic material such as polymers.

The above methods of improving the lateral strength of an earth structure apply to cases where the mass material and its method of placement are chosen by the designer. Greater understanding is required for excavations in natural deposits. The research described in this thesis relates to the lateral strengthening of natural in situ materials so that an excavation can be made safely with minimal external support to the faces or a slope cut at a very steep angle. In these instances reinforcing elements are installed within the in situ soil as the excavation or
cutting proceeds.

This study concentrates on the method of soil nailing. In this first chapter, other methods of lateral earth support are also described for comparison and to show that although there are similarities between them, the operating mechanisms of each are often quite different. The section has relevance when discussing issues relating to soil nailing design. Following from these observations, definitions for soil nailing and soil dowelling are proposed. An overview of soil nailing is then given, including its history, method of construction and typical behaviour under working conditions. The development of the research and the contents of the thesis are summarised in the final section of the chapter.

1.1 METHODS OF IMPROVING THE LATERAL STRENGTH OF EARTH STRUCTURES

In this section, internal reinforcement of the ground itself is dealt with, rather than cases where an external structure provides support (e.g. diaphragm walls or contiguous, secant, or sheet pile walls). The main objective of the section is to define the principal methods of lateral earth reinforcement.

The five systems described are shown in Figure 1.1.

1.1.1 Ground anchors

These are used in in situ soils in combination with some external facing against the excavation face (e.g. a diaphragm wall). Usually the wall is constructed before any excavation takes place and then anchors are installed through it sequentially as layers of soil are removed. Anchors are relatively long compared with soil nails and are grouted into intact competent material, over the ‘fixed’ or bond length of the anchor, some distance back from the face. A length immediately behind the wall is sleeved and is known as the ‘free length’ of the anchor. The main function of an anchor is to tie the wall back to ground beyond the vicinity of the excavation, to prevent structural movements. The anchors are tensioned at the face of the wall and thus are loaded directly. In the case of smooth-bored anchors, this load is carried by the shaft friction generated over the bonded length, in the same manner that a pile is loaded. For reamed anchors, where the bore is increased in diameter locally, the load is taken in part by bearing.

1.1.2 Soil nails

These are installed sequentially as an excavation progresses in a similar way to ground anchors. However, there are some important differences. Soil nails are much shorter, typically 5 to 10 m in length and their whole length is in contact with the soil. A light shotcrete facing is applied immediately after excavation. Nails are generally not tensioned against the facing
other than to seat the head of the nail. The support mechanism of soil nails is the friction that is built up along their length as the soil moves outwards as excavation proceeds. They are therefore loaded *indirectly* by the soil as it expands from the removal of support and swells from the lateral stress release (*i.e.* without an externally applied axial force). A more detailed description of the mode of operation of soil nails is given later in this chapter in Section 1.5.

### 1.1.3 Reinforced earthfill

Although this technique is not relevant to *in situ* soils, it is worth considering because of certain similarities with soil nailing. The word earthfill has been adopted so that a general term can be referred to rather than the tradename Reinforced Earth. It is a bottom-up, staged, earthfill construction technique where the layers of reinforcement are loaded indirectly and go into tension as the embankment is being built. Comparisons between reinforced earthfill and soil nailing behaviour and design have been covered in detail by Schlosser (1982, 1983) and Bruce and Jewell (1986) and are discussed where relevant in Section 1.5.

### 1.1.4 Reticulated micro-piles

This system (also known as *pali radice*) operates by forming a dense forest of piles orientated both parallel and angled to a proposed excavation face, entering the reinforced zone from the surface prior to excavation. A reinforced block of soil is formed by these piles which retains the mass behind. The piles resist the lateral forces acting on the block both in bending and shear.

### 1.1.5 Soil dowels

These are usually installed vertically in shallow slopes where additional shear resistance is required along either a potential or existing slip surface. They therefore act primarily in shear and because of this they tend to have very much larger cross sections than soil nails. Lengths vary depending on the depth of the slip surface: they must extend to an adequate depth below the shear surface so that bending moments can be fully developed in the sections.

Further discussion on the actions operating around soil dowels and their possible relation to the behaviour and capacity of soil nails are given in Chapter 2.

### 1.1.6 Comparisons between the methods

In Figure 1.1, some differences in operating mode and mechanism between the systems are evident. In the cases of ground anchors, micro-piles and soil dowels, reliance is placed predominantly on soil-structure interaction within soil which is being stressed in the close vicinity of the reinforcements (*i.e.* most of the soil remains in its original undisturbed state). There will be local swelling from stress relief at the front of the excavation in the cases of ground anchors and reticulated micro-piles but the principal load-bearing sections of the reinforcements are in intact ground. However, in the cases of soil nailing and reinforced
earthfill, and particularly the former, the whole soil mass around the inclusions is changing state as a result of stress changes occurring (from unloading in the soil nailing situation). Stress changes trying to take place within the soil are transmitted to the inclusions. As a consequence the lateral ground expansion is arrested in part and a strengthened mass with active reinforcements is provided.

1.2 DEFINITIONS OF SOIL NAILING AND SOIL DOWELLING

The current BSI (1993) Glossary of building and civil engineering terms (based on a collation of all parts of BS 6100) does not give a specific definition for soil nailing. The DoT HA68/94 document, which is an Advice Note covering design methods for the reinforcement of highway slopes using reinforced soil and soil nailing, gives the following definition.

Soil nailing is the technique whereby in situ ground (virgin soil or existing fill material) is reinforced by the insertion of tension-carrying soil nails. Soil nails may be of either metallic or polymeric material, and either grouted into a predrilled hole or inserted using a displacement technique. They will normally be installed at a slight downward inclination to the horizontal.

As the Advice Note relates to slopes there is perhaps no necessity to be more explicit about the specific application of soil nails. However, no mention is given to soil dowels or the reinforcing of an unstable slope where there is an existing or potential slip surface. As the modes of operation of these two methods of reinforcement are quite different, the following definitions are proposed for use in this study.

Soil nails are inclusions installed into soil experiencing lateral stress relief from either the forming of an excavation or the steepening of a slope. They are typically placed horizontally and operate by going into tension as ground movements occur during removal of further successive layers of soil from in front of the excavated face below them.

Soil dowels are inclusions installed into soil where there is no stress relief locally, but where it is necessary to reinforce a ground mass, e.g. in a slope with an existing or potential shear surface. They are usually placed at an orientation perpendicular to existing or potential shear surfaces and operate by withstanding bending and shear stresses applied across their sections. ¹

¹The word dowel does usually refer to an element operating in shear (see definitions for dowel and dowel bar given in BSI glossary 1993) and the term soil dowelling is commonly understood to have this meaning. However, this meaning is not clear with the term rock dowel. In the BSI glossary a ground anchorage is defined as 'an installation to transmit an applied tensile load to loadbearing strata' and then rock dowel as 'a ground anchorage in which a bar of steel or other appropriate material is fixed without tensioning in rock'.
It should be noted that some types of soil nails can also develop bending and shear stresses across their sections but that these only become significant at large displacements when a nailed-soil structure starts to approach collapse.

1.3 THE HISTORY OF SOIL NAILING

An instance of inclusions being used to provide additional lateral strength to an excavation undergoing stress relief dates back to 1835 and the construction of the Thames Tunnel (Skempton and Chrimes, 1994). It was necessary to dismantle the original tunnelling 'shield' (comprising an iron frame of three levels with twelve bays in each) so that it could be replaced after the flooding of 1828 and consequent halting of works for seven years. Marc Brunel, in charge of the tunnel construction, specified that 'pins' be driven into the soil in front of the shield prior to removing any sections of it (Beamish, 1862). These pins were strips of iron about eight feet long and four inches wide: they undoubtedly would have gone into tension as the supporting framework was removed and thus acted in exactly the same manner as soil nails. They proved effective in as much as it was possible to replace the shield without major loss of ground from the face.

The evolution of the present system started from mining applications where reinforcements were fully bonded by resin into rocks. The New Austrian Tunnelling Method (NATM) developed from this in the 1960s; in this system, fully bonded reinforcements with a shotcrete facing are used to provide temporary support to excavations. Initially NATM was used only in hard rocks, but later it was extended to weaker formations such as shales and marls (see Rabcewicz, 1964-1965). The same idea also began to be used and gain acceptance in the stabilisation of steep rock slopes. By the late 1960s trials were being carried out in soils such as silts, sands and gravels, leading in 1970 to the construction of sections of the Frankfurt Metro tunnel by successful experimentation using the method. The first major nailed-soil structure was constructed in 1972 near Versailles, France: a 70° cut slope was formed in cemented Fontainebleau Sand as part of a railway widening scheme (Rabejac and Toudic, 1974).

Following the success of this project, the method was increasingly used in France for forming both steep slopes and excavations. The system was also being developed, seemingly independently, in W. Germany and the U.S.A..

In Germany the system was developed under a research-and-development project Bodenvernagelung started in 1975 at the Institut für Bodenmechanik und Felsmechanik (IBF) of the University of Karlsruhe under Stocker (1976). It was the literal translation of that German word that resulted in the term 'clouage des sols' or in English, soil nailing.

The first reported major project using soil nailing in N. America was the excavation
works carried out in 1976 for the foundations of an extension to the Good Samaritan Hospital, Portland, Oregon, (Shen et al., 1981a). The term used then for the technique was ‘lateral earth support system’; soil nailing is now more common.

The above references mark the principal initial project in France, Germany and the U.S.A.. A comprehensive review of major works carried out since then is given by Bruce and Jewell (1986, 1987) and some more recent case histories are given in Recommandations Clouterre (see below). Developments and refinements to the technique itself are described in Section 1.4, which covers the construction of nailed-soil structures.

An extensive research programme into soil nailing was carried out at CERMES in France (Centre d’Enseignement et de Recherche en Mécanique des Sols, the research group at the École Nationale des Ponts et Chaussées) under Schlosser. The results of this work are given in Recommandations Clouterre (1991), which is probably the most authoritative work on the subject at this time. An English translation of the above work was made in 1993 under the direction of the United States Federal Highway Administration (see under Recommandations Clouterre 1991).

During the past decade the technique has been increasingly used in these countries and elsewhere in the world. There are several references to soil nailing in Japan (e.g. see Kakurai and Hori, 1990; Tateyama et al., 1993), Hong Kong (e.g. see Powell and Watkins, 1990; Shui et al., 1997), Brazil (Ortigao et al., 1995) and South Africa (Schwartz and Friedlaender, 1989; Heymann et al., 1992). However, it should be noted that some of these cases relate to the strengthening of existing slopes, and so according to the proposed definitions, the reinforcements act as soil dowels. In the U.K. there are an increasing number of cases: the largest known reported project was for a 6 to 11-m high 70° slope cut into weakly cemented silty fine sand at Rye, Sussex, (Anon., 1991). A number of cases are given by Barley (1992a). More recently, the design and construction of an extensive temporary nailed-soil wall in Keuper Marl have been described (Baker, 1994) and an 8-m nailed-soil wall in sand and gravel near Bournemouth is also reported (Anon., 1995). A soil nailing project in Jersey has been cited, but this appears to be more a case of rock bolting (Wheeler, 1994, Warner and Barley, 1997).

The reasons for the slow acceptance of soil nailing in the U.K. were suggested by Bruce and Jewell (1986) as ranging from lack of suitable applications (e.g. unsuitable soils) to lack of knowledge and even to protectionism by those trading in alternative techniques. Another reason could be that there was, until recently, still no accepted design method for nailed-soil structures in the U.K. as there is in France and Germany (see for example the article by Wheeler, 1993). However, there are now two documents available giving guidance on the design of nailed-soil structures, although the main emphasis in both is on the steepening of slopes. First, an Advice
Note covering the design of reinforced slopes incorporated into the DoT Design manual for roads and bridges (see HA 68/94, 1994). More recently a code of practice has been produced for Strengthened/reinforced soils and other fills (BS8006:1995). The controversy in recent years over whether to take factors such as bending stiffness into account in the analysis will have further undermined confidence in the technique.

1.4 CONSTRUCTION STAGES

Soil nailing is a staged set of construction techniques that follows a similar sequence to that shown in Figure 1.2.

1.4.1 Stage 1: excavation

This is usually carried out in steps of 1 to 2 m depth depending on the ability of the soil to remain stable for a few hours without support. This time is required to enable a shotcrete facing to be applied prior to the nails being installed. Gässler (1990) suggests some typical unsupported excavation depths for different soils as given in Table 1.1.

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Cut depth</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>(sandy) gravel</td>
<td>0.5 to 1.5 m</td>
<td>some capillary cohesion required at lower limit: upper limit for cemented material</td>
</tr>
<tr>
<td>sand</td>
<td>1.2 to 1.5 m</td>
<td>medium dense to dense with capillary cohesion</td>
</tr>
<tr>
<td></td>
<td>1.2 to 2 m</td>
<td>if cemented</td>
</tr>
<tr>
<td>silt</td>
<td>1.2 to 2 m</td>
<td>depending on structure, stability of grain skeleton and water content</td>
</tr>
<tr>
<td>clay</td>
<td>1.5 to 2.5 m</td>
<td>normally to overconsolidated (i.e. all states)</td>
</tr>
</tbody>
</table>

Table 1.1. Unsupported excavation depths of nearly vertical cuts (after Gässler, 1990)

Surface water should be removed at all times to prevent local erosion of the excavated face, this is usually achieved with a drainage trench along the crest of the excavation.

As can be seen from Table 1.1, some capillary or natural cohesion or cementation is usually necessary to excavate to the depths shown. Depths will be reduced or ground treatment may be necessary when there are: zones of low density in sands; dry uncemented sands with water content less than 10%; external vibrations; inclusions of larger stones or rock pieces; or seepage pressures from water-bearing layers. The economic viability of the method of soil nailing is partly controlled by the ability to be able to leave reasonable heights of free unsupported face exposed at each step. Sometimes a thin layer of shotcrete (20 to 30 mm thick)
8

is sprayed on the face immediately after excavation to improve temporary stability.

Often it is the first two steps that are most critical as the upper ground consists of weathered weaker soil and in urban areas is often of loose fill with voids (Gässler, 1990).

1.4.2 Stage 2: shotcreting

The thickness of shotcrete, which is applied as soon as possible after excavation, is usually in the range of 80 to 200 mm depending on soil properties, height of wall and expected service life (temporary or permanent). The final appearance of shotcrete is generally not aesthetically acceptable for permanent structures and so sometimes vegetation is encouraged to grow on the shotcrete surface.

The shotcrete is usually reinforced with one or two layers of mesh depending on the permanency of the structure: for temporary works only one layer is generally used either near the soil surface or in the mid-section of the shotcrete. If nails are installed before shotcreting, it is good practice to embed the nail plate (at its head) into the wet concrete. If, as is the usual situation, nails are to be installed after shotcreting, temporary blocks can be placed at the nail positions and then removed before drilling.

Variations to standard reinforced shotcrete are (i) the use of prefabricated metal panels, applied as reinforcement to wet concrete (see Louis, 1987, p.67) and (ii) shotcrete reinforced with steel fibres. This latter method is useful in cases where the wall is temporary and will be demolished.

Alternatively, precast panels can be used. These have some advantages in terms of appearance, and that they are constructed off site, but they can only be used where the excavation steps will remain stable without support for some period; their use also means that wall geometry cannot readily be changed to suit ground conditions during construction, as it can with shotcrete.

1.4.3 Stage 3: installation of nails

There are various types of nail and methods of installation which are usually chosen with regard to the soil type, the permanency of the wall and its geometry.

The traditional method, which is probably still the most widely used, is predrilling a hole, inserting a reinforcing bar and grouting it in place. Depending on the soil conditions and length and diameter of nail, the hole is drilled open or cased. Open-hole drilling is faster and cheaper but can only be achieved in clays, soft rocks and well-cemented sands. It is carried out by rotary boring with or without a flushing medium. If a flushing medium is employed, it is usually air or foam (drilling mud should be avoided) the hole is sometimes additionally advanced by percussive means; alternatively the hole is dry augered. Cased-hole drilling is not often used in soil nailing applications as generally it is not commercially viable. When it is adopted,
drilling proceeds using either an auger, rotary methods or a down-the-hole hammer, the casing being advanced in all cases as the hole is bored; usually it is necessary to flush with air or water in the latter two methods.

In some applications, nails, commonly comprising open tubes or angles, are driven directly; this method has become known as the Hurpinoise system, after M. Hurpin of Bouygues who pioneered the technique in the Versailles-Chantiers job (Rabejac and Toudic, 1974; see also Goulesco 1984). The ‘pins’ driven in the Thames Tunnel under the direction of Marc Brunel (Beamish, 1862) fall into this category. This system is usually only adopted for temporary works because of the possible corrosion of the reinforcement: there are also limitations on the length of nails that can be driven. Nails are installed at a much higher density (say one nail per 0.5 m$^2$) than the predrilled type (one nail per 2.5 m$^2$) as their cross-sectional area is much smaller. The system has the advantages of speed, economy and avoiding temporary casing for granular soils. It is suited more to granular than cohesive soils, although care should be taken to ensure that loose deposits are not destabilised by driving.

Another dynamic installation technique without grouting developed in the U.K. involves firing reinforcing bars into the ground using a converted missile launcher (see Myles and Bridle, 1991). Although the method has been used to a limited extent for shallow slope stabilisation, the effects of the high speed penetration are unknown. It is probable that the compression wave propagated along the nail on entering the ground would tend to create an oversized hole, with a consequent decrease in frictional capacity that would restrict the nail’s effectiveness. A useful comparative study would be of pull-out tests (this being the standard test for assessing the frictional capacity of in situ inclusions) on nails either driven Hurpinoise style or fired into the same stratum.

The German company Bauer developed another alternative nail during the Bodenvernagelung research in the 1970s, where a tip of greater diameter than the reinforcing bar attached to it creates an oversized hole which is simultaneously grouted as it is advanced.

Another German group, Ischebeck GmbH, developed a type of self-boring nail for ground anchoring (first used in 1983). The nail has a ribbed outer surface and a hole running down its centre through which initially air or water is flushed and in later stages grout pumped as the nail is rotated into the ground. Add-on sections can be fitted as drilling proceeds. The nails may be zinc coated to provide corrosion resistance.

One of the latest and most effective techniques of soil nailing is the jet bolting system, described by Louis (1987). This involves driving a specially manufactured nail into the ground using a vibro-percussive hammer. The nail has a hole down its centre and slots in its tip for pressure grouting. Very high grout pressures of greater than 20 MPa (Gässler, 1990) are
applied, which lubricate the end and enable very rapid installation.

Apart from the above principal types of nails and recent developments, there are some other variants such as injection nails (see for example Guilloux and Schlosser, 1982; Wichter, 1984). Materials such as fibreglass have also been tried for nailing applications, mainly to reduce potential corrosion problems.

1.4.4 Durability of nails

The importance of a nail's resistance to corrosion depends on whether it is part of a temporary or permanent structure. German codes define the former as relating to a structure that has a working life of less than two years. Concern arose because metal strips used in early Reinforced Earth structures were found to seriously corrode within a short period. Circular bars used in soil-nailing (usually of minimum diameter 20 mm) are less prone to corrosion because of their smaller exposed area compared to that of thin, flat strips. Nevertheless, this is of considerable importance as the working life of a structure is generally several decades. The steel of the bar itself affects the overall life of a nail: high tensile strength bars (generally used in anchors) have less resistance to corrosion than those of mild steel which in most cases are adequate for nailing applications (and cheaper too).

Nails driven or fired into the ground without grouting have little protection against corrosion; even if they are coated, it is unlikely that the surface will remain intact during the course of installation. These are therefore only suitable for temporary structures, sometimes a sacrificial increase in diameter is provided to accommodate corrosion. A nail grouted into the ground cannot be guaranteed against corrosion as microcracking can occur when the nail goes into tension under operational conditions. Even with temporary nails, precautions such as adequate centralising and provision of sufficient cover (usually 15 mm minimum) should be taken.

Full protection for permanent nails can be achieved (as based on German procedures) by preparing the nail in two stages, the first off site. The steel bar is centralised within a protective corrugated plastic tube (sheath) so that there is an annulus of at least 5 mm which is then grouted from the base up so as not to leave any air pockets. This operation is performed with the sheaths lying in steel channels so that the nail ‘formers’ can be transported to site without cracking. This former (with centralisers placed at 1.5 to 2 m intervals) is then set in the nail hole, which is either filled with grout beforehand if the hole is unstable, or grouted afterwards using some form of tremie pipe: the latter method is preferred and more commonly used. The inner core is thus protected against corrosion by the plastic sheath while stresses are fully transmitted from the surrounding soil to the nail itself by virtue of the ribbed surface of the sheath.
1.4.5 Provision of drainage

Another important consideration regarding the permanency of a nailed-soil structure is the provision of adequate drainage from behind the face of the excavation. This has two purposes: first to ensure that pore water pressures do not build up behind the wall and, second, to minimise effects of frost. Frost action can increase force at the head of the nail and reduce frictional resistance along the affected length; for these reasons, the effect of frost is one of the main current research themes at CERMES (Unterreiner et al., 1994).

Drainage can be provided simply by installing short perforated tubes (less than 1 m long) between the nails at a spacing to suit the ground-water conditions and passing through the facing to drain freely. However, these can leave unsightly stains on the wall and the drained water is not efficiently removed from the base of the excavation. A better alternative is to place geomembrane strips or perforated pipes, vertically on the soil face before shotcreting. These can be overlapped and extended as the excavation steps are made. It is then necessary to provide a horizontal drainage channel along the base of the excavation to collect the water.

The above methods are for draining water from the soil face; for deeper excavations with longer nails, long drains extending into the soil mass beyond the length of the nails are sometimes used. These comprise slotted or perforated tubes (often wrapped in a filter geotextile) which are inserted in holes angled upwards at a slight incline to ease flow, collecting tubes are connected to the drains to lead off the water.

1.5 THE BEHAVIOUR OF A NAILED-SOIL EXCAVATION UNDER WORKING CONDITIONS

The main points discussed in this section concern current understanding of the overall behaviour of a nailed-soil structure. Reinforced earthfill has been studied for longer than soil-nailing and there is considerably more known about this method of soil reinforcement. Consequently, similarities between the two systems are sometimes used to propose working mechanisms in nailed-soil structures.

The sequence of construction of a nailed-soil excavation is explained in Section 1.4 and shown in Figure 1.2. The self-tensioning of the nails occurs progressively as excavation proceeds. During the excavation for the first full-scale experimental wall, as part of the Clouterre research programme, it was found that about 20% of the working tension load in the nail developed during its initial installation. In practice, the initial amount mobilised depends on how soon after excavation the nail is installed. Most of the tensile force in a nail is mobilised during the three subsequent phases of excavation (i.e. cutting for and installation of the following three layers of nails), there being negligible increase thereafter. This behaviour
is shown in Figure 1.3 where, for illustration, the load at the head of a nail installed in the third row of a typical excavation is considered during the subsequent excavation phases. As a consequence of this effect, the lowest rows of nails are subjected to the smallest tensile forces, although these do continue to increase after construction with long-term deformation. In the case of reinforced earthfill the reverse occurs and the reinforcing layers are increasingly stressed as successive layers of fill are placed above them, i.e. the uppermost layers of reinforcement are subjected to the smallest tensile forces.

Tensile force develops in a nail as a result of stress relief at the face of the excavation and consequent outward deformation of the soil mass. Deformations are greatest at the top of a nailed-soil structure and decrease with depth as illustrated in Figure 1.4, which shows lateral movements resulting from excavation. Conversely, with reinforced earthfill structures, it is the base of the wall that tends to bulge out as construction proceeds, as this is where the greatest tensile forces occur.

The distribution of tensile force along the nails is not uniform and has a maximum within the reinforced soil mass rather than at the head of the nail, see Figure 1.5. This latter characteristic has been observed in other types of earth reinforcement where there is a continuous interaction with the soil along the whole length of the inclusion (e.g. again reinforced earthfill). The position of the point of maximum tensile force along each row of nails is significant as the line formed by joining these points separates two zones within the reinforced mass. There is: (i) an ‘active’ zone immediately behind the facing where the soil exerts shear stresses along the nail-soil interface which act outwards (towards the facing) and (ii) a ‘passive’ or resistant zone where the shear stresses exerted along the nail-soil interface act inwards (towards the soil mass).

The interface shear stresses in the active zone result from the soil responding to the progressive stress relief taking place as excavation proceeds. The soil deformations are outward and are restrained to some degree by the reinforcements: soil movements are greater than those of the reinforcements which are anchored further back in the mass (i.e. in the passive zone). Thus there is greater relative movement of the soil than the nail in the active zone. The interface shear stresses developed in the passive zone are induced by the reinforcing elements being pulled from within the active zone. The effect of the stress relief at the face is much reduced in this zone, consequently soil deformations are much smaller than those in the active zone. Thus in the passive zone, the relative movement of the soil is less than that of the nail. These ideas are shown schematically in Figure 1.5.

It follows from the above argument that there is no relative movement between soil and reinforcement at the boundary of the two zones, which is the line of maximum tensile force in the nails. This is an important factor when considering the experimental boundary conditions
of the apparatus used for the research described in this thesis.

The actual location of the line of maximum tension is not easy to ascertain because of the complexities of the nail-soil interaction and the overall behaviour of a nailed structure (which is dependent on various parameters relating to the soil, water conditions and geometry). Bassett and Last (1978) argue theoretically that the line becomes vertical towards the top of the excavation. In their approach the line of maximum tension is considered as a potential failure surface; they show that the presence of horizontal reinforcements working in tension behind a retaining wall completely modifies the field of deformations as shown in Figure 1.6. This is based on the concept, described by Roscoe (1970), that failure lines in a material correspond to zero-extension lines. Full-scale nailed-soil walls constructed with instrumented nails have also shown this effect, where failure occurs along the line of maximum tensile force in the nails (e.g. Cartier and Gigan, 1983).

During excavation there will also be downward vertical movements, mostly immediately behind the facing. These, during construction, are usually of similar magnitude to the lateral movements and vary from $H/1000$ for rocky soils to $4H/1000$ for clays. Bending moments would be induced in the reinforcements as a consequence of such movements, and also in initial stages from the weight of the facing on the heads of the nails until facing construction is complete.

Materials used for reinforced earthfill structures are usually flexible and unable to sustain any bending moments; for this reason there is no case to be made for these providing additional restraint in shear. On the other hand, soil nail reinforcing elements have more rigidity (admittedly weak in the case of driven rods, but quite strong for grouted nails) and so have some capacity for resisting shear across their section. Under typical soil nailing working conditions the capability to sustain shear stress across the nail sections is not utilised or only to an insignificant degree (Jewell and Pedley, 1990a). Significant deformations across the sections would be required to mobilise such shear stresses and at this point the nails would be close to a limiting equilibrium.

1.6 THESIS CONTENT

The preceding sections of this introductory chapter give an overview of soil nailing. The study described in this thesis investigates and examines the behaviour of soil nails, in particular the stress transfer mechanism that takes place as deformation of the soil mass occurs during excavation. This subject is then discussed in relation to current field practice and analysis and design. The way that the research has been approached is reflected in the following description of the chapters.
Chapter 2 summarises the methods available for analysis and design of nailed-soil structures and the main influencing factors controlling them. One of the main components of design is determining the tensile capacity of soil nails, the approaches available are discussed. One of the 'fundamental' indices commonly required for this aspect of design is the 'pull-out resistance' of a soil nail. This is usually obtained by performing pull-out tests on site either before and/or during construction; test methods and the relevance of their results are examined. The relevance of incorporating reinforcement bending stiffness into the design is also discussed. Most design methods are based on limit equilibrium analyses. The methods adopted by different countries and some of the several computer programs for analysing and designing nailed-soil structures are discussed. The chapter concludes by summarising some different and more recent approaches to analysis and design.

Chapter 3 reviews the different approaches to studying soil-reinforcement interaction that have been taken in the past. The purpose of the chapter is provide an overview of the different approaches to the problem that have been made. The intention is to consider the reasoning behind the types of research rather than to give a detailed account of the findings. Much of the laboratory research into soil nailing, where soil nail element tests have been performed, has been carried out using variously modified forms of shear box. Most of the information relating to the role of bending stiffness comes from such tests. The chapter continues with a summary of scaled tests on walls, from models constructed in the laboratory through centrifuge testing, to full-scale prototype tests. On the basis of the conclusions drawn about the previous research work, the approach that has been adopted for this study was developed.

The main conclusion is that the tensile axial forces generated in nails from shear stresses mobilised along the nail-soil boundary give additional strength and rigidity to a soil mass undergoing lateral stress relief, these aspects of interaction are the focus of this research thesis. A fundamental approach is taken, using laboratory element tests on single model (very small scale) nails. In order to control the boundary stresses and deformations accurately, the tests were performed in a specially modified triaxial apparatus. At the end of Chapter 3, the experimental boundary conditions necessary to simulate conditions in situ are summarised and the stress paths to be followed during the testing explained.

It was decided initially to perform tests to investigate nail interaction with clay, for which purpose, kaolin was chosen. It was necessary to select sample and nail dimensions to satisfy the boundary and apparatus conditions. Two main criteria to be achieved were that the nail should be of such a size to be small enough to ensure that a substantial mobilisation of stress could take place along its interface (if too large in diameter then failure could occur across one of the sample boundaries), but large enough for instrumentation to be practicable. Another
target to attain was reasonably uniform shear stresses at the interface where they were to be measured; the development of stress with progressive displacement being under investigation, as well as studies of stress distribution along the nail.

The test conditions were investigated using parametric numerical analyses based on the finite element method; their numerical simulation is described in Chapter 4. One of the most difficult boundaries to model is the interface between the nail and the soil (this after all being the main theme of the research). The methods available for numerically modelling this boundary and those adopted are explained in the chapter. The results from the parametric studies, where nail and sample sizes were varied, are presented with conclusions regarding the final selection of dimensions.

Once the sample dimensions were established, it was necessary to design the apparatus and experimental testing system that would achieve the specified boundary conditions during testing. The triaxial apparatus modifications and testing procedures for clay are outlined in Chapter 5 along with notes of trials performed to establish procedures for operations such as installing the nail in the sample.

After the initial clay tests had been performed, the behaviour of nails in granular materials was investigated. It was necessary to develop another testing system as the triaxial apparatus used for clay samples was not suitable for setting-up sands. The apparatus was designed so that sand samples could be tested under similar triaxial boundary conditions as for the clay. This equipment, the testing system and setting-up procedure are described in Chapter 6.

One of the early tasks of the research was the design and fabrication of an instrumented model soil nail. In order to investigate the changes of shear stress and normal stress on the nail interface the model nail had to be instrumented to measure axial force and radial stress on its surface. Development of the model nail involved extensive trials and experimentation. A full description of the final design of the instrumented model nail is given in Chapter 7, which includes coverage of cell-action effects. A record of the development of the instrumentation from which much was learnt that could benefit other researchers is given in appendices.

The series of tests on sands ended up being far more comprehensive than that for clay, for this reason the sand tests are presented first. This allows a development of a series of idealised models and phenomena such as rigid-body inclusion effects, arching and restrained dilation to be introduced.

The results from the sand tests are presented and analysed in the context of the experiments themselves in Chapter 8. The investigation into the behaviour of a single nail in granular soil involved carrying out a series of tests on three sands of fine, medium and coarse
grain size at three densities from loose to dense. Reference tests without any nail inclusions were also performed for comparison. Different methods for assessing the benefit of the nail are discussed. Additional interpretation of the results is also made using the results from a series of shearbox tests performed both as interface and sand-sand tests.

The clay tests are presented and analysed with respect to the experimental conditions in Chapter 9. When the clay test programme was carried out, the instrumented model nail had not been developed sufficiently to enable measurements of radial stress to be made. However, values are estimated using the results from interface shearbox tests.

Previous studies on soil-structure interaction, carried out at Imperial College (e.g. Martins, 1983, and Mahmoud, 1989) have shown the importance of examining changes in the clay microfabric during the mobilisation of shear stress in the soil at the interface. The opportunity was therefore taken to prepare samples from some of the tests for microfabric analysis. Observations from this supplementary investigation are also presented in Chapter 9.

Chapter 10 draws together information from all aspects of the study and interprets the results of the tests in both clay and sand in relation to field practice and the analysis and design of soil nailing works. Assessments are made of the possibility of some of the effects observed in the tests being witnessed under field conditions.

Chapter 11 presents the main conclusions from the study along with suggestions for further research.
In many advanced papers on soil mechanics the mathematical refinement is out of proportion to the importance of the errors due to simplifying assumption. If these assumptions are clearly and completely stated, such papers have at least the merit of honest mental experiments and the reader may be in a position to judge for himself to what extent the experiment can be considered successful. However, statements covering all the vital assumptions are rather exceptional. Since few readers know the subject well enough to recognise a gap in the set of assumptions, a theoretical paper with an incomplete set may do more harm than good.

Occasionally investigators make assumptions of grave import without suspecting it themselves. On closer scrutiny of their papers one may even find that their attempts to solve old problems by apparently more rigorous methods have increased the error owing to the fact that a set of conspicuous but tolerable assumptions was replaced by less conspicuous but far more detrimental ones. The most instructive examples of which can be found among the advanced theories dealing with the stability of slopes and with the bearing capacity of piles and pile groups.

Karl Terzaghi, Theoretical Soil Mechanics, 1943 (p65).
CHAPTER 2

The analysis and design of nailed-soil structures

Most of the several methods available for the design of nailed-soil structures, are based on a limit equilibrium approach. These methods follow the same principles used in slope or wall analyses where activating and resisting forces are calculated for different potential failure surfaces. The ratio of resisting to activating forces, the factor of safety, can then be established to assess the stability of the earth mass. In such analyses, the lengths of the nails projecting beyond the potential failure surface are considered to contribute to the resisting forces. Assigning values to these forces is not straightforward, as pointed out by Jewell (1990).

Uncertainty in the evaluation of the force that can develop in reinforcement through bond stress still remains, a decade after the debate started.

The principal objective of the research reported in this thesis is to understand better how shear stresses are mobilised in the vicinity of the interface of the nail and hence generate forces within it. The intention is to be able to quantify the bond stresses that operate under working conditions, considering factors such as the effects of arching and restrained dilation and the displacements required to mobilise them within the context of limiting serviceability deformations.

The last point is particularly important: in practice, deformations are ‘controlled’ using various approaches. A sufficiently high factor of safety could be adopted, the value of which is usually based on previous experience where nailed-soil structures have had tolerable movements when constructed in similar ground conditions. More specifically, the first row of nails could be overdesigned, reducing deformations at the head of the structure. If pull-out tests are being used to establish the bond strength, the strength available up to a certain magnitude of displacement can be adopted in the design. Construction techniques could also help minimise movements, e.g. by restricting excavation to smaller steps or by excavating the face at a slight batter.

The purpose of this chapter is to review current soil nailing design, paying particular attention to how bond strengths are determined and how deformations are considered. Initially, the methods available for determining the allowable tensile force for reinforcing elements are described, with particular emphasis on the pull-out test. The subject of reinforcement bending stiffness and its relevance to soil nailing design is also discussed.

The design methods as used in France, Germany, U.S.A. and U.K. are summarised, drawing attention to common similarities and relevant differences. Commercially available
computer programs exist for most of the limit equilibrium methods and these are briefly
described and their main features discussed.

The final part of the chapter considers alternative methods of design which are either
still under development (e.g. methods based on kinematics or deformation criteria) or are not
readily applicable to general use (e.g. the finite element method).

2.1 THE DETERMINATION OF BOND STRENGTH FOR DESIGN PURPOSES

This section is about the bond strength resulting purely from the nail-soil friction. The
limiting tensile strength of the reinforcement itself is not be considered here nor cracking of the
grout which is discussed in Thompson and Miller, (1990). Three methods of determining the
bond strength or skin friction are described. These include approaches where values are obtained
solely from past experience, and from \textit{in situ} tests where the skin friction (that generated from
the imposed conditions of the test) is measured directly. The end of this section is a short
discussion of the relevant merits of each method.

Beneficial effects from effects such as restrained dilation, induced during the
mobilisation of shear stress in the soil surrounding the nails, are not usually taken into account
in the determination of bond strength (except in the case of pull-out tests where its contribution
is included in the total force measured). However, the subject which has been studied by others
(for example as part of the Clouterre project), is covered in this thesis in Chapters 3, 8 and 10
where previous work is reviewed and discussed in relation to the results from the experimental
tests carried out. The phenomena of rigid-body inclusion effects and arching that were observed
in this study are also covered.

2.1.1 Design charts

The use of design charts for determining the skin friction of reinforcing elements is
possible as a result of the large database of pull-out tests in different soil types collated during
the work carried out as part of the Clouterre project. Charts for five soil types are given in the
\textit{'Recommandations'} (1991, pp 150-152, French edition), although it is made very clear that these
should only be used for preliminary design. Skin friction ($q$) is related to a limiting soil
resistance ($p$) measured with the pressuremeter, a site investigation tool that has more
widespread use in France than in the U.K.. In this way variations of \textit{in situ} stress due to
different overburden pressures and lateral stresses can be taken into account when considering
the structure to be designed.

Most of the pull-out tests in the database were carried out under conditions of controlled
force and the remainder by controlled displacement (these different types of pull-out test are
described in Section 2.1.3). Not all of the tests were taken to failure as in some cases the
service load was being assessed and in others the elastic limit of the reinforcing bar was less than the failure load of the grout or the grout-soil interface.

Some of the charts provide individual \( q - p_i \) curves for different methods of nail installation. These curves provide a useful means of obtaining an initial idea of the nail capacity for preliminary design (discussed in Section 2.3.1). The total limiting bond strength or resistance, expressed as a nail force, is then given by the expression:

\[
T_L = \pi D L_0 q_i
\]  
(2.1)

where \( T_L \) is the maximum pull-out force at the head of the nail, either a peak value or that after a certain displacement depending on the type of test being performed. \( L_0 \) is the anchored length of the nail defined either as the working length of the nail in a pull-out test, or for design considerations, the length of the nail extending into the resistant zone, in which case \( T_L \) relates to the nail force at the boundary of the zone (see Figure 1.5). \( D \) corresponds either to the borehole diameter for grouted nails or equivalent diameter for driven nails.

Note that Recommendations Clouterre (1991) state that design on the basis of such charts ought to be verified by compulsory \textit{in situ} pull-out tests.

2.1.2 Consideration of \textit{in situ} stresses and a bond coefficient

\textit{Approach used in NAIL-Solver}

The method described below is incorporated in the computer program NAIL-Solver. Details of the program and the following approach to bond strength are given by Jewell (1990). The approach is similar to that sometimes used in the design of reinforced earthfill structures and forms the basis of the DoT HA68/94 document which deals with the design of nailed-soil walls. The average normal effective stress acting around the nail is taken to be the mean of the vertical effective overburden pressure, \( \sigma_v' \) and the \textit{in situ} effective stress acting laterally on the nail and parallel to the excavated face, denoted by \( \sigma_l' \). These stresses are given by the following expressions:

\[
\sigma_l' = \gamma z - u
\]  
(2.2)

where \( \gamma \) = bulk unit weight of the overlying soil,

\( z \) = depth of soil,

\( u \) = pore pressure, and

\[
\sigma_v' = \sigma_l' K_i
\]  
(2.3)

where \( K_i \) is a coefficient of lateral earth pressure, which is used to determine the lateral stress (out-of-plane) acting in a soil deforming under plane-strain conditions. The average normal
stress (normal to the reinforcement), \( \sigma_{\nu}' \), is then given by:

\[
\sigma_{\nu}' = 0.5 \sigma_{\nu}'[1 + K_i]
\]  

(2.4)

In order to determine the skin friction on the nail, an angle of interface resistance needs to be incorporated. This is achieved by using a bond coefficient, \( f_b \), which relates the angle of interface resistance, \( \phi_b' \), to the internal angle of shearing resistance of the soil, \( \phi' \):

\[
\tan \phi_b' = f_b \tan \phi'
\]

(2.5)

The skin friction per unit area of reinforcement is thus given by:

\[
q_s = \sigma_{\nu}' \tan \phi_b' = 0.5 \sigma_{\nu}'[1 + K_i] f_b \tan \phi'
\]

(2.6)

The ultimate bond resistance that can be sustained (if \( \phi' \) is not factored) for each level of nails can then be determined from equation (2.1).

The two unknowns in equation (2.6) are \( K_i \) and \( f_b \). Jewell mentions that laboratory measurements on granular materials suggest a typical range \( 0.5 < K_i < 0.8 \). He then continues: for comparison, if the mean stress around the nail were derived from the vertical and the outward horizontal stress (which could be assumed to be not less than \( \sigma_{\nu}' = K_o \sigma_o' \)), then the coefficient of lateral earth pressure would not be less than \( K_l = (1+K_o)/2 \). This statement is somewhat clarified in the HA6894 document (see next section) where the same expression for \( K_l \) is also used, otherwise it is not obvious in the NAIL-Solver document where this lower limiting value of \( K_l \) is derived from and why the outward horizontal stress is being considered.

It seems reasonable to assume the horizontal effective stress to be not less than \( K_o \sigma_o' \). However, this would imply that the lower bound to \( K_l \) should be \( K_o \). In fact, \( K_o \) might be a more appropriate coefficient for the case where the nails are installed by predrilling a hole and hence causing an active movement of the surrounding soil radially inwards (without considering any restraining effects from arching). In which case a more likely explanation is that a lower limiting value of the normal stress, \( \sigma_{\nu}' \), of \( \sigma_{\nu}'(\text{min}) = \sigma_o'(1+K_o)/2 \) was meant.

The approach of taking the mean of the vertical and lateral (parallel to the slope face and perpendicular to the nail axis) stresses to represent the average normal stress on the nail seems reasonable (equation (2.4)). It is assigning a value \( K_i \) that is problematic. It is not clear in the NAIL-Solver manual which value for \( K_i \) is used, whether a value in the range \( 0.5 < K_i < 0.8 \) or \( K_o \) which is significantly smaller. During soil nailing construction operations, the principal ground movements are within the plane perpendicular to the excavated or steepened face. Lateral movements out-of-plane can occur as a consequence of the changing stress field (rotation of principal stresses) or locally from the installation of the nails. If such movements
are very small, it could be considered that \( K_t \) is approximately equivalent to \( K_o \). \( K_o \) and \( K_0 \) values are therefore compared in Table 2.1 for a range of \( \phi' \) values for a normally-consolidated granular material.

<table>
<thead>
<tr>
<th>( \phi' ) (deg)</th>
<th>( K_o )</th>
<th>( K_o=1-\sin\phi' )</th>
<th>( K_o/K_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.70</td>
<td>0.83</td>
<td>1.19</td>
</tr>
<tr>
<td>20</td>
<td>0.49</td>
<td>0.66</td>
<td>1.35</td>
</tr>
<tr>
<td>30</td>
<td>0.33</td>
<td>0.50</td>
<td>1.52</td>
</tr>
<tr>
<td>40</td>
<td>0.22</td>
<td>0.34</td>
<td>1.55</td>
</tr>
<tr>
<td>50</td>
<td>0.13</td>
<td>0.23</td>
<td>1.77</td>
</tr>
</tbody>
</table>

Table 2.1. Comparison of \( K_o \) and \( K_0 \) values for a range of values of \( \phi' \).

The range of \( K_t \) values from laboratory measurements under conditions of plane strain given by Jewell correspond more closely to the \( K_o \) values although even these generally fall well below the range given. No details are given of the experiments, it is possible that effects from dilation resulted in increased values of lateral stress being determined. The subject is discussed further in the next section and in Appendix 2.1.

The bond coefficient \( J \) is a measure of the inclusion interface roughness and so would not be expected to be greater than unity. Jewell suggests that values of \( J < 0.4 \) would be very unlikely. Values of \( h \) can be estimated for different construction materials (by using angle of interface resistance values for the appropriate material, e.g. a steel pile against sand) or measured in direct shearbox interface tests.

**Approach used in HA68/94**

The DoT document describes a method which is similar to that used in NAIL-Solver. As the method of analysis can be used for both nailed-soil and reinforced earthfill structures more general expressions are given with factors which depend on the application. The design pull-out resistance for each layer/level of reinforcement is given by an expression similar to (2.1):

\[
P_{\text{det}} = \lambda_p L_e (\sigma'_n \tan \phi'_n + c'_n) \tag{2.1a}
\]

where \( \lambda_p \) is a non-dimensional pull-out factor dependent on the type of reinforcement, \( L_e \) is the length of reinforcement which extends beyond the critical failure surface and \( \sigma'_n \) represents the normal effective stress acting on the reinforcement beyond the failure surface. As mentioned in the previous section \( \sigma'_n \) is defined as \( \sigma'_n/(1+K_o)/2 \). It is stated in Appendix D of the
document that *if active conditions are assumed to develop perpendicularly to the slope then it may be shown for a given yield criterion and flow rule* *(Burd, Yu and Houlsby, 1989)* *and conditions of plane strain parallel to the slope and zero dilation (a conservative assumption),* that \( K_i = \frac{1 + K_a}{2} \). It is noted in the text that using the above expression for \( \alpha'^a \) might lead to underestimations of \( \alpha'^a \) for granular soils as it ignores any effects of dilation, and overestimation in soils with appreciable cohesion because of arching of soil around the nail. This statement is reasonable although arching is at least as likely to occur in granular soils, such effects can be verified by pull-out tests. The expression \( K_i = \frac{1 + K_a}{2} \) is not clearly derived by Burd *et al.* It may not be valid, see Appendix 2.1 where a case is made, based on experimental and analytical evidence to show that it is invalid. Also care should be taken to check for the case when \( L_c \) given in equation (2.1a) is greater than the length of the nail in front of the potential failure surface (*i.e.* the plane between active and resistant zones). In such a case pull-out in the front face can occur, this matter is discussed in Section 2.3.4.

The factor \( \lambda_p \) is given by \( \pi Dv_a/S_a \), where \( S_a \) is the horizontal spacing of the nails (necessary as \( P_{des} \) relates to the pull-out of each layer rather than each nail) and \( \alpha \) is an interface sliding factor. This \( \alpha \) value is essentially the same as \( f_h \) mentioned above but allows for the soil having a \( c' \) value, though HA68/94 recommends that \( c' \) is taken as zero for clayey soils with \( PI < 25 \% \). \( \alpha \) is therefore defined by:

\[
\alpha = \frac{c'_t \tan \phi'_m + c'_m}{c'_t \tan \phi'_d + c'_d} \tag{2.5a}
\]

where the subscripts *int* and *des* relate to interface and design values respectively. It is also suggested that \( \phi'_m \) and \( c'_m \) values are chosen so that \( \alpha = \tan \phi'_m / \tan \phi'_d = c'_m/c'_d \) is constant regardless of \( \sigma'_t, \) *i.e.* that the same partial strength factor is applied to \( \tan \phi' \) and \( c' \), this idea is shown in Figure 2.1. Comprehensive guidelines are given regarding which values of \( c' \) and \( \phi' \) to use for the interface and design cases for different soil types, *e.g.* whether to use critical state or residual values or factored peak values.

### 2.1.3 Pull-out tests

This method of determining the bond strength at the nail soil interface is the most likely to give results directly applicable to the site ground conditions. In *Recommandations Clouterre* (1991), it is recommended that such tests be compulsory for every job to verify the nailing design (*i.e.* the skin friction value used which perhaps had been derived from pressuremeter test results) using the relevant type of nail within the soil in which the excavation is to be made. Three types of test are described.

(i) Preliminary tests at the planning stage: these are usually only employed when a soil or
nailing technique different from normal applications is to be used.

(ii) Conformity tests carried out when work starts on site: these are compulsory if no preliminary tests are performed and are to verify the skin friction values adopted in the design. They should be carried out for each soil layer once the appropriate excavation depth has been reached.

(iii) Inspection tests during the construction: these are to check the as-constructed nails during the works, the nail not being taken to failure.

The tests all have the same procedure and are performed primarily to assess the nail skin friction. This is achieved by applying a static tensile load at the head of the nail causing a build-up of friction until movement occurs. Note that tested nails should not be incorporated into the structure. Nail cross-sections are chosen so that tensile failure of the nail would not occur.

The following description of the test is based on the recommendations given following the Cloutere project. The standard pull-out test is described as a controlled displacement test (i.e. at constant rate of movement of the head). When these are carried out as preliminary or conformity tests, they should also be augmented by an equal number of controlled force tests (i.e. with creep stages at constant load).

Full details of the physical set-up of the controlled displacement pull-out test are not given here (those for the controlled force test are essentially the same). However, it is useful to appreciate some of the precautions taken to help provide meaningful and reproducible results. Prior to loading the nail in tension, the free length of the nail should be fully sleeved by a tube which must not come into contact with the reaction plate. If there is no free length, the loading mechanism must be supported by a waling system so that reactions are not applied within a metre of the nail axis and no bending of the nail must occur. The nail is tensioned at a rate of nail head displacement of 1 mm/min ±10 % with clearance for 50 mm of travel. In the controlled displacement test, the maximum pull-out resistance, the residual resistance and the initial slope of the force-displacement curve can be obtained as shown in Figure 2.2. Results from these tests are usually plotted in this form i.e. as force against displacement at the head of the nail. The maximum pull-out force $T_L$ is the maximum force value obtained during the test, defined either at peak or residual as shown in Figure 2.3 (when there is no decrease in force, there are cut-offs in terms of either change in force per unit displacement or a limiting maximum displacement). Having established $T_L$ for the test, $q_s$ can be obtained from equation (2.1) and applied to the appropriate dimensions of the nail to be used.

It is important that both load and displacement are measured as this enables a relationship between them to be assessed, helps estimate the magnitude of wall deformations and in the future will be essential for design at the serviceability limit state. In Recommandations
Clouterre (1991) a theoretical relationship is developed from a skin friction mobilisation law given by Frank and Zhao (1982), this is shown graphically in Figure 2.4. The relationship enables the displacement at the head or tip of the nail to be calculated from the known force at the head of the nail and the slope, $k_p$, of the initial part of the load-displacement curve (i.e. initial tangent modulus) from a representative pull-out test. The derivation of the expressions given and the assumptions made in the approach are detailed in Appendix 2.2.1

It is appropriate to mention here the controlled force pull-out tests; these do allow the maximum pull-out resistance to be determined in most cases, but are mainly used to establish the critical creep tension load. A controlled displacement test should be performed first, to establish the maximum pull-out force, $T_L$. In Recommandations Clouterre (1991) an estimated limiting pull-out force, $T_{LE}$, is introduced for controlled force tests, this must be less than $0.6T_G$, where $T_G$ is the elastic limiting force of the reinforcement and is based on the $T_L$ measured previously. During the test the force is applied to the nail in steps of $0.1T_{LE}$ starting from $0.2T_{LE}$ and at each step the force is held constant for an hour (three hours for the $0.7T_{LE}$ stage). A series of displacement measurements is taken at set times once the load is reached, the load then being held constant to within $0.001T_{LE}$. If the nail does not lose its bond when $T_{LE}$ is reached, the loading steps should be continued to $0.9T_G$, and if the bond then still holds the nail should be unloaded. The nail skin friction can be calculated from $T_L$, for the cases where it is reached, again using equation (2.1).

The results from controlled force pull-out tests are presented as cumulative displacements against log time; an example is given in Figure 2.5. The creep curves are generally straight at the lower loads becoming more curved as $T_L$ is approached. An angle $\alpha$ is defined as the slope of the tangent to these curves at time, $t = 60$ minutes. This angle, $\alpha$, can be plotted against $T/T_{max}$ as shown in Figure 2.6 and from this plot the critical creep tension, $T_C$, can be determined. This is defined as the previous load applied before the curve sharply changes gradient, indicating much greater creep potential; a value $T_C'$. Also shown is the intersection of the two straight portions of the curve. The results from numerous pull-out tests indicate that $T_C' \approx 0.9T_C'$. If $T_L$ is not attained during the test, it can be estimated using an empirical relationship between $T_L$ and $T_C$: $k = T_L / T_C$. Values of $k$ are based on a large number of pull-out test results, e.g. for bored and gravity grouted nails, $k = 1.2$ for sands and 1.3 for clays, (Recommandations Clouterre, 1991, Table III, Chapter 4).

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1In Recommandations Clouterre, displacement $y$ is used both for the lateral movement of the nail from deflections either side of a potential failure surface as well as the axial movement of the nail during pull-out testing. Although this could lead to confusion the symbols have not been altered. In this study usage in the lateral sense is restricted to the discussion on bending stiffness.
2.1.4 Conclusions regarding the current methods of determining bond strength

Three methods have been described, two of which are directly related to in situ pull-out tests. In the first, \( q_l \) is obtained from design charts which are based on the results of many pull-out tests; these have been correlated with the limiting pressure, \( p_l \), from pressuremeter tests. Values thus obtained for design should be verified before construction with appropriate site pull-out tests. The third method involves pull-out testing at the outset, from which the limiting and critical creep tensile forces (\( T_L \) and \( T_C \) respectively) can be obtained. Thus again \( q_s \) can be determined and the final design can be checked so that none of the nails is loaded to a force near \( T_C \), beyond which extensive deformations would be expected.

In the second method, the normal stress acting on the nail is estimated and a bond resistance calculated using an angle of interface resistance between the soil and the nail. The uncertainty with this approach concerns the values to attribute to the normal stress on the nail, in particular the lateral stress in the ground and the interface resistance angle. However, estimates can be made by drawing on experience with pile and reinforced earthfill design, where similar interface and in situ stress conditions also have to be assessed. Any beneficial effects from dilation are not taken into account and there does not seem to be any way of estimating or limiting deformations other than by arbitrarily increasing the factor of safety.

The pull-out test appears to be the best way of estimating nail bond resistances that relate directly to the site ground conditions and nail installation method. Such tests can also provide information about limiting forces to minimise deformations and are more likely to take soil arching and dilation effects into account. Alternative procedures for performing pull-out tests have recently been proposed in order to provide more representative results (Barley et al. 1997a and b). The interpretation of results from pull-out tests are discussed in Chapter 10.

2.2 THE RELEVANCE OF REINFORCEMENT BENDING STIFFNESS IN SOIL NAILING DESIGN

In this section, the lateral loading of a reinforcement is considered. Such loading occurs when a potential slip surface is developing across the nail section: one part of the reinforcement is held within the resistant zone (see Figure 1.5) while the tendency for downward movement in the active zone causes shear and bending stresses to be set up within the nail in the vicinity of the slip surface.

The interactions between the forces and stresses acting in the nail and the soil in such a condition are complex and difficult to quantify. Limiting factors that have to be taken into account include:

- shearing and bending resistance of the nail (its resistance to plastic deformation);
- bearing capacity of the soil (to resist the stresses imposed by the loading of the nail);
- tensile capacity of the nail itself and;
- pull-out resistance of the nail (i.e. its maximum skin friction).

These factors relate to the stability of the nailed-soil system. The magnitude of deformations required to mobilise the different actions requires further more complex analysis. It is not taken into account in current design approaches, except in some numerical analyses. There has been considerable debate and controversy in recent years about which type of analyses to adopt for assessing stability in terms of bending stiffness, or whether to take it into account at all. It is not a subject that has been covered experimentally or analytically in this research, but resolution is necessary if the different design methods for soil nailing are to be assessed.

The subject is discussed under three separate headings which are then interrelated.

(i) The system of actions (e.g. shear forces, bending moments) that are induced along the length of a reinforcement embedded in an elastic or plastic medium as a result of an applied lateral force, (Sections 2.2.1 and 2.2.2).

(ii) The effects of having combinations of actions in a member (i.e. axial force, shear force and bending moment), where the member is examined in isolation without considering pressures or reactions exerted on it from surrounding soil, (Section 2.2.3).

(iii) The influence of the bearing capacity of the soil in terms of its capability to resist the stresses applied to it by the nail and particularly whether the soil or nail reaches a plastic state first, (Section 2.2.4).

(iv) The combination of the above, a multicriteria system, to borrow Schlosser's terminology, (Section 2.2.5).

These subjects were originally considered by Schlosser (1982, 1983) and formed his multicriteria method. The analyses and method have since been refined and are now included within Recommandations Clouterre (1991). Jewell and Pedley (1990a and b) looked in detail at some of the relationships put forward by Schlosser and proposed different analyses and interpretations of their own. For instance, in item (i) above Schlosser considers everything using elastic analyses (Section 2.2.1) while Jewell and Pedley (1990b) suggest some relationships based on a plastic approach (Section 2.2.2).

Jewell and Pedley's approaches have been incorporated into a design method (1990b) which is discussed later. Note though, that effects from the bending resistance of nails are considered by them to have only a negligible contribution to the system of forces maintaining stability within a nailed-soil structure under working conditions. Therefore they are not included in Jewell's NAIL-solver program nor the HA68/94 document.

The components of these two methods are described in detail in the following sections.
Other methods, such as those proposed by Juran et al. (1988 and 1990a and b) and Bridle (1989), either partly or fully utilise the Schlosser method: their interpretation and implementation are discussed below where there is a significant deviation from Schlosser's original approach.

2.2.1 Schlosser's approach to analysing a laterally loaded inclusion

In Schlosser's 1982 and 1983 papers the subject of the lateral loading of a reinforcement by relative movement of two soil masses (active and resistant as shown in Figure 1.5) along a discrete shearing plane is related to the case of a laterally loaded pile. The reinforcement is assumed to bend elastically and the soil is represented by a series of elasto-plastic springs. The assumed mode of deformation is shown schematically in Figure 2.7; in the analysis, conditions on one side of the shearing surface are considered. The same conditions for analysing a pile embedded in a soil modelled by elastic springs and loaded by a horizontal force and moment at its head were considered by Matlock and Reese (1962) in their development of the $p-y$ method. It is useful to refer to the initial introduction to their work in order to see from where the base expression quoted by Schlosser is derived. Figure 2.8 is reproduced from their paper and shows the distribution of different quantities (e.g. moment and shear) along the length of a pile (in direction $x$) being subjected to lateral load and moment at its head. These distributions are based on solutions using traditional elastic beam theory. The expression for the soil reaction (expressed as force per unit length of pile) is given by:

$$p = EI \frac{d^4y}{dx^4} \tag{2.7}$$

where $EI$ represents the flexural rigidity of the pile. Another definition of $p$ is introduced as:

$$p = -Ey \tag{2.8}$$

in which $y$ is the lateral deflection of the pile and $E$ is an elastic spring modulus representing the soil stiffness. The negative sign indicates that the direction of the soil reaction is always opposite to the direction of the pile deflection. Equating these expressions yields an equation similar to that given by Schlosser:

$$EI \frac{d^4y}{dx^4} + Ey = 0 \tag{2.9}$$

In his 1982 paper, the second component is expressed in two ways depending on the magnitude of the deflection $y$. The delineating deflection is given as $y$, which appears (it is not defined) to correspond to the deflection required to cause the lateral earth pressure adjacent to the nail to exceed the limiting pressure, $p_l$ as measured in a pressuremeter test. It would seem reasonable to assume that these ranges relate to elastic and plastic behaviour. In characterising the second
term, recourse is again made to work on laterally loaded piles, so that the modulus of subgrade reaction, \( k \), and reinforcement diameter, \( D \) are introduced. The second term is thus defined as:

\[
E_y y = k_i D y
\]

(2.10a)

for \( y < y_i \) (elastic condition), and

\[
E_y y = p_j D
\]

(2.10b)

for \( y \geq y_i \) (plastic condition).

The subgrade reaction is defined by Hetényi (1946) as the force which distributed over a unit area causes a deflection equal to unity; its units are expressed as force per unit length cubed. The justification for the expressions given by equations (2.10) is based on relationships, developed from experimental observation as well as on theory, between the pressure in front of a pile being laterally loaded and the displacement \( y \) (usually normalised by the pile radius). Details of the studies carried out in this field are beyond the scope of this work, the following expression corresponding to the elastic range is given by Hetényi and is the one used by Schlosser in his 1983 paper.

\[
E l \frac{d^4 y}{dx^4} + k_i D y = 0
\]

(2.11)

The solution of this equation for the boundary conditions relating to a soil nail is given fully in Appendix 2.3. The solution is expressed in terms of a common component \( \lambda \) known as the characteristic of the system of four expressions defining the roots of the equation. \( \lambda \) has units of \((\text{length})^{-1}\) and so the term \( 1/\lambda \), which is denoted \( l_0 \) in this study, has units of length and is referred to as the characteristic length.

\[
l_0 = 4 \sqrt{\frac{4EI}{k_i D}} = \frac{1}{\lambda}
\]

(2.12)

This ‘length’ is significant as it is a function of the flexural rigidity and diameter of the inclusion and the subgrade reaction of the soil, these being the main components controlling the system.

In Figure 2.9, the curves describing the distribution of deflection, slope, bending moment, shear force and soil reaction are given with their corresponding equations (as would be expected the shapes of the curves are similar to those in Figure 2.8). Distances to the different maxima and zero points are defined in terms of the characteristic length, \( l_0 \). Note that the equations contain the term \( W \) which represents the resultant of the soil reaction on one side of the slip surface, which for equilibrium is equal to \( V_0 \), the shear force in the nail at the position
of the slip surface \((x = 0)\).

It is interesting how the characteristic length \(l_c\), (referred to as the transfer length by Schlosser), which is essentially an intrinsic component of the mathematical solution, takes on a physical identity in Schlosser’s soil nailing design methodology. The analysis is developed on the assumption that the reaction forces of the foundation are proportional at every point to the deflection of the member at that point; this was first proposed by Winkler in 1867. Terzaghi (1943, p 345), comments as follows when considering the case of foundation contact pressures. In order to establish agreement between assumption and reality it would be necessary to replace the soil support of the footing by a bed of equally spaced and equally compressible springs each one of which is independent of the others. This is a highly artificial concept. Therefore the results of computations based on this concept must be considered crude estimates. Hetényi (1946) in a similar vein states that while the theory of beams on elastic [spring] foundations holds rigidly for most [structural] problems, its application to soil foundations should be regarded only as a practical approximation. It is its simplicity as a method of analysis that makes it a very useful tool for obtaining an impression of the actions taking place around a soil nail. More detailed and accurate representations can be obtained using numerical analyses (e.g. the FEM, in particular, utilising the work by Ganendra (1994), who developed a pseudo three-dimensional method of FE analysis using Fourier series to analyse laterally loaded piles and pile groups). Usually, because of time and financial restraints, such analyses are restricted to special projects.

In the solution developed in Appendix 2.3 it is assumed that the nail is infinitely long. Schlosser qualifies this by saying that a pile loaded laterally at its head can be assumed to behave as an infinitely long pile if its length is greater than \(3l_c\), and that the same can be applied to the nail case. This length, presumably, was chosen as \(3l_c/2\) roughly corresponds to \(\pi l_c/2\) the distance to the first point at which nail deflections are zero in the distributions either side of the slip surface (refer to Figures 2.8 and 2.9 and equation A2.3xvii). At a distance of \(3l_c\), all actions, resulting from the downward movement of the active zone, have essentially diminished to zero.

The relationships given by Schlosser in both the 1982 and 1983 papers are between moment and soil reaction and between displacement and shear force. They are obtained from equations (A2.3xvii) to (A2.3xxi) given in Appendix 2.3. \(M = M_{\text{max}}\) at \(x = \pi/4\lambda\) and so:

\[
M_{\text{max}} = \frac{4EI\lambda^3W}{k_pD} e^{-\frac{x}{\lambda}} \sin 45 = \frac{W}{\lambda} e^{-\frac{x}{\lambda}} \sin 45
\]  

and \(p = p_{\text{max}}\) at \(x = 0\), giving:
\[ p_{\text{max}} = - \frac{8EI\lambda^4 W}{k_x D} = -2\lambda W \] \hspace{1cm} (2.14)

Rearranging and substituting for \( W \) gives:

\[ M_{\text{max}} = -0.16l_o^2 p_{\text{max}} \] \hspace{1cm} (2.15)

Note that \( p \) as defined above is expressed as force per unit length. It should also be noted that \( W \) is used to represent the resultant force from the mass of the active zone acting on the nail at the position of the slip surface, see Figure 2.9. In Schlosser’s expressions the ‘width of the pile’, \( B \) is included, which means that \( p \) is expressed as force per unit area in his solution. The most likely reason for this is so that \( p \) can be related to different pressure levels obtained from the pressuremeter test. It is evident that care must be taken to define \( p \) clearly. In this work the symbol \( p \) denoting soil resistance per unit length, as assigned by the earlier works (e.g. Hetényi, Matlock and Reese) will be retained; soil resistance per unit area will be denoted \( \sigma_o' \) (as used by Jewell and Pedley, 1990a and b). Continuing with the development of relationships: \( y = y_o \) and \( V = V_{\text{max}} \) at \( x = 0 \) and so:

\[ y_o = \frac{2\lambda W}{k_x D} \] \hspace{1cm} (2.16)

and

\[ V_{\text{max}} = \frac{4EI\lambda^4 W}{k_x D} = W = \frac{M_{\text{max}}}{0.32l_o} = \frac{p_{\text{max}}l_o}{2} \] \hspace{1cm} (2.17)

from which

\[ y_o = \frac{2V_{\text{max}}}{k_x Dl_o} \] \hspace{1cm} (2.18)

The above expressions relating \( y_o, M_{\text{max}}, V_{\text{max}} \) and \( p_{\text{max}} \) are used in the development of the multicriteria method which considers limiting conditions for four states of reinforcement to soil interaction. These states are described in Section 2.3.1. It is important to remember at this stage that the above expressions are developed from an analysis using elastic springs and that they relate to actions resulting from the combined influences of the soil and reinforcement (later in Section 2.2.3, loading conditions relating to the inclusion alone are considered).

2.2.2 Jewell and Pedley’s approach to analysing a laterally loaded inclusion

Jewell and Pedley (1990a and b) cover bending and shear resistance in detail; much of the discussion here arises from their work. The elastic analysis adopted by Schlosser (as described in Section 2.2.1) is considered and then compared with plastic analysis for three
different cases; two they proposed themselves and the other is referenced to Hansen and Lundgren (1960).

Jewell and Pedley prefer to use the length between points of maximum moment in a bar in their analyses. This is denoted $l_s$ and referred to as the shear width (1990a). From Figure 2.9 it can be seen that:

$$l_s = \frac{\pi l_o}{2}$$  \hspace{1cm} (2.19)

Jewell and Pedley (1990a) utilise the ratio $l_s/D$ for assessing the role of bending stiffness in soil nailing design. In considering the elastic analysis used by Schlosser they express equation (2.15) as:

$$\left(\sigma'_b\right)_{\text{max}} = \frac{P_{\text{max}}}{D} = \frac{M_{\text{max}}}{0.16 l_s^2 D} = \frac{15.4 M_{\text{max}}}{l_s^2 D}$$  \hspace{1cm} (2.15a)

and equation (2.17) as:

$$V_{\text{max}} = \frac{4.9 M_{\text{max}}}{l_s}$$  \hspace{1cm} (2.17a)

Note that neither Schlosser nor Jewell and Pedley show $M_{\text{max}}$ as negative in quoting equation (2.15). Equations (2.15a) and (2.17a) have been expressed here using $l_s$ to enable comparisons to be made with Jewell and Pedley's expressions derived from plastic analyses.

The following analyses are described as plastic, presumably to mean perfectly plastic with uniform constant stresses regardless of the deformations taking place (i.e. with no strain hardening). Expressions relating to the three analyses are given below in the required form for comparison with each other and the elastic analysis. The derivations of the expressions are given in Appendix 2.4 for completeness.

The first plastic analysis was derived in their (1990a) paper. The assumed pressure distribution is shown in Figure 2.10 with symmetry satisfied by imposing zero moment where the potential slip surface crosses the reinforcement. The distance $l_b$ is that to which the reinforcement length must extend to satisfy equilibrium, for this case $l_b/l_s = \sqrt{3}/2$ and

$$V_{\text{max}} = \frac{4 M_{\text{max}}}{l_s}$$  \hspace{1cm} (2.20)

The second plastic analysis was given only as a limiting model and was not intended to simulate working conditions. The mechanism is shown in Figure 2.11 and assumes rigid-body shear displacement with encastré reinforcement. In this case, $l_b$ is not a consideration in view
of the latter assumption, while:

\[ V_{\text{max}} = \frac{2M_{\text{max}}}{l_s} \]  

(2.21)

The third plastic analysis, that of Hansen and Lundgren (1960), is shown in Figure 2.12. It is similar to the first shown in Figure 2.10 except that the ratio \( l_v/l_s = 1\sqrt{2} \). As shown in Figures 2.10 and 2.12 the reinforcement length either side of the potential failure plane must be greater than \( (l_v + l_s/2) \) to enable equilibrium to be satisfied.

It can be seen from equations (2.17a), (2.20) and (2.21) that the maximum shear force and bending moment can be related using the expression

\[ V_{\text{max}} = \frac{CM_{\text{max}}}{l_s} \]  

(2.22)

where \( C = \) constant depending on the type of analysis used.

### 2.2.3 Limiting relationships for a reinforcing member subjected to a combination of loads

The previous sections present relationships between bending moment, shear force and earth pressure. Another factor that needs to be considered is that the capacity of an element to resist bending is dependent on the level of axial force acting in it (specifically tension in nailing cases). Also, as the shear force that an element can sustain is related to the bending moment, this too is related to axial force. Expressing these relations in terms of maximum values as were used in the previous sections.

\[ M_{\text{max}} = fn (T) \quad \text{and} \quad V_{\text{max}} = fn (M_{\text{max}}) \]

and so

\[ V_{\text{max}} = fn (T) \]

where \( T \) is the axial force in the element.

The relationship between shear and axial stress is arrived at by Schlosser using the Tresca criterion, which postulates that yielding will occur if the shear stress, \( \tau \), exceeds a limiting shear stress, \( k \). Considering an element of a reinforcement and expressing axial and shear stresses using a Mohr diagram, as shown in Figure 2.13, a limiting inequality can be written:

\[ \sqrt{\tau^2 + \sigma^2/4} = \tau \leq k \]  

(2.23)

At the limit:
\[
\frac{\tau^2}{k^2} + \frac{\sigma^2}{4k^2} = 1
\]

(2.23a)

and in terms of force:

\[
\left(\frac{V}{V_{\text{lim}}}\right)^2 + \left(\frac{T}{T_{\text{lim}}}\right)^2 = 1
\]

(2.24)

where \(V_{\text{lim}}\) and \(T_{\text{lim}}\) are the limiting shear and axial forces and which must be related by \(T_{\text{lim}} = 2V_{\text{lim}}\) to satisfy the expression (2.23a). Note that the limiting forces are those beyond which plastic behaviour occurs.

Jewell and Pedley (1990a) arrive at the same expression (2.23a), by drawing an analogy to the relationship between axial force and torque in a member subjected to these actions, (developed by Taylor and Quinney and described by Calladine, 1985), recognising that \(k = \sigma_y / 2\) or \(k = \sigma_y / \sqrt{3}\) for materials behaving as Tresca or von Mises materials respectively, where \(\sigma_y\) is the limiting or yield axial stress, (refer to Calladine, 1985 p.50 and p.154).

In continuing the development of the relationships between combined actions existing in an element, Jewell and Pedley point out that although the limiting relation between axial and shear force is independent of the element cross-section, this is not so for the relation between axial force and bending moment. It is this relationship that is the next subject to be dealt with and is the one that prompted Jewell and Pedley’s paper (1990a) and has also provoked considerable correspondence and discussion between different parties, (Ground Engineering: June 1991, vol. 24, no. 5, p. 7; July/August 1991, vol. 24, no. 6, p. 7; September 1991, vol. 24, no. 7, p. 6; Discussion in November 1991, vol. 24, no. 9, pp 30-39).

In the original papers by Schlosser, there is mention of a limiting relation associated with the effect of bending, defined by a parabola in \((T,V)\) space, but no details are provided nor specific expression quoted. Blondeau et al. (1984) give an expression relating shear force in the inclusion to a maximum composite bending moment and limiting earth pressure when there is no axial force in the nail. Note that the moment is referred to as composite to indicate that the bending action is influenced by the soil resistance, i.e. it is not a case of pure bending. This relationship is referred to again in Section 2.2.4 as equation (2.33).

The next stage is to develop relationships between axial force, shear force and bending

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\(^2\)Jewell and Pedley in response to a discussion by Schlosser (1991, pp 34-35) of their 1990a paper say that they do not consider that the state of stress in a bar subject to combined axial and shear loading can be represented by a unique Mohr’s circle as stresses across the bar section are non-uniform and shear stresses reduce to zero at the boundaries of the bar.
moment within a reinforcing element. Jewell and Pedley (1990a and b) introduce an expression relating maximum moment and axial force in a bar at the plastic limit:

\[
\frac{M_{\text{max}}}{M_p} + \left( \frac{T}{T_p} \right)^2 = 1 \tag{2.25}
\]

This expression defines a parabola and is only valid for a bar of rectangular cross-section; no such simple form is available for a circular section, but equation (2.25) approximates well and is only slightly conservative. \(M_p\) and \(T_p\) are described as fully plastic values which apply for a bar in pure bending or in pure tension. In practice a more conservative quadrilateral surface is adopted, given by:

\[
\frac{M_{\text{max}}}{M_p} + \frac{T}{T_p} = 1 \tag{2.26}
\]

Both bounding surfaces are shown in Figure 2.14. The expression (2.26) can be given in terms of elastic as well as plastic limiting values by replacing \(M_p\) by \(M_{\text{elm}}\) so that \(M_{\text{max}}\) is the maximum elastic moment permissible. Such relationships linking bending moment and axial force are given in *Recommandations Clouterre* (1991), and by Schlosser and Jewell and Pedley in their discussions (1991) using the more complete forms given by equations (2.27) to (2.29) given below. However, no such relationships in a soil nailing context were published before this time (except equation 2.33). In the meantime a Draft Manual of Practice for Soil Nailing by Elias and Juran (1988) had been written in which bending stiffness is included as a beneficial component in the analysis (as reported by Juran et al. 1991). The consequences of not taking all the factors into account, which are discussed in Section 2.2.5, were what originally concerned Jewell and Pedley (1990a).

Three expressions relating \(T\), \(V\), and \(M\) are given in *Recommandations Clouterre* (1991) which are of a similar form; the first is attributed to Anthoine, (1987) and is described as being a simplified formula:

\[
\left( \frac{T}{T_{\text{lim}}} \right)^2 + \left( \frac{V}{V_{\text{lim}}} \right)^2 + \left| \frac{M}{M_{\text{lim}}} \right|^2 \leq 1 \tag{2.27}
\]

which is said to be conservative compared to Sobotka’s formula (1954):

\[
\left( \frac{T}{T_{\text{lim}}} \right)^2 + \left( \frac{V}{V_{\text{lim}}} \right)^2 + \left| \frac{M}{M_{\text{lim}}} \right|^2 \left[ 1 - \left( \frac{V}{V_{\text{lim}}} \right)^2 \right] \leq 1 \tag{2.28}
\]

and also to the expression given by Neal (1961):
Note that when there is no shear force, $V$, all three expressions revert to the same as that given by Jewell and Pedley (1990a, equation 4) for a bar of rectangular cross-section (equation 2.25). The derivation of these formulae can be obtained from the original references.

Equations (2.27) to (2.29) are not exact, as the relationships depend on factors such as the type of cross-section of the member and the end conditions. Neal states that lower bounds can be found which he believes to be close to the true interaction relations. His equation (2.29) is described as an empirical interaction relation which, although based on the analysis of a cantilever with rectangular section, he suggests can be used for the solution of other problems providing that the length/depth ratio of the beam is not too small.

The main point to note in these relationships is that the load-carrying capacity at a point along an inclusion is very dependent on the other actions acting at that point. It is not possible to approach limiting conditions of shear force, for example, while axial forces exist (equation 2.24); the magnitude of shear that can be developed is then further reduced when there is a bending moment as well (equations 2.27 to 2.29). As mentioned above, these relationships derive from consideration of an elemental 'beam' (inclusion). In practice the ability of a nail to carry such a system of forces is governed by the strength of the soil around it. In Sections 2.2.1 and 2.2.2 earth pressures or reactions (for elastic and plastic cases respectively) were incorporated into the analyses without considering in detail the limitation of the soil strength and its bearing capacity. This is the next influence to be examined.

2.2.4 Influence of the soil's bearing capacity

The soil's strength plays an important role in the development of stresses and moment in an inclusion and also in the mode of deformation and failure of the individual members. Schlosser, in his early (1982, 1983) papers, categorised the soil behaviour as either elastic or plastic, depending on the deformation, $\gamma$, induced by the nail (as described in Section 2.2.1, equations 2.10a and b). Equation (2.17) relates shear force in the nail, $V_{\max}$, and the soil resistance $p_{\text{max}}$ for the elastic case. The limiting point at which the soil starts to behave plastically, assuming that the nail is cutting through the soil without deforming plastically itself, is defined by the same equation, but replacing $p_{\text{max}}$ by $p_{\sigma}$, the ultimate bearing capacity of the soil. At the limit:
\[ V_{\text{max}} \leq \frac{l \sigma_{\text{ult}}}{2} = \frac{D l_{o} \sigma'_{\text{ult}}}{2} \]  

(2.30)

where \( \sigma'_{\text{ult}} \) is the ultimate bearing capacity in units of stress (force per unit area).

The above relation is relevant where the soil deforms plastically before the nail. For this to occur either the nail must deform sufficiently or it must cut through the soil. Another relationship to be considered is the case when the nail deforms plastically first. This would take place at the points of maximum bending moment, occurring at a distance \( \pi/4 \lambda = \pi l_{o}/4 \) either side of the point of intersection of nail with potential slip surface.

In considering the plastification of the nail alone without taking into account any soil resistance, *Recommendations Clouterre* (1991) adopts a simplified approach, where soil and nail behaviour are described by elastic relations. At the point of \( M_{\text{max}} \), where \( V = 0 \), equation (2.27) reduces to:

\[ \left( \frac{T}{T_{\text{lim}}} \right)^{2} + \left| \frac{M}{M_{\text{lim}}} \right| = 1 \]  

(2.27a)

or

\[ M = M_{\text{lim}} \left[ 1 - \left( \frac{T}{T_{\text{lim}}} \right)^{2} \right] \]  

(2.31)

(note that as \( T \rightarrow T_{\text{lim}}, M \rightarrow 0 \)). Now the shear force in the nail at the intersection point with the failure surface is given by equation (2.17), rearranging gives:

\[ M_{\text{max}} = 0.32 l_{o} V_{\text{max}} \]  

(2.17a)

The limiting relationship for \( V_{\text{max}} \) can now be obtained for the point when \( M \rightarrow M_{\text{max}} \), taking into account the axial force acting in the nail *(i.e. after equating expressions 2.31 and 2.17a)*:

\[ V_{\text{max}} = 3.125 \frac{M_{\text{lim}}}{l_{o}} \left[ 1 - \left( \frac{T}{T_{\text{lim}}} \right)^{2} \right] \]  

(2.32)

For this case where the nail deforms plastically first, its deformation is limited by the soil and can only continue once the soil starts to yield as well. The limiting relationship is therefore described by a combination of limiting states of soil and nail, *(i.e. by combining equations (2.30) and (2.32) to give:)*
It should be noted that no derivation for this equation is given in *Recommendations Clouterre* (1991) or the original reference of Blondeau *et al.* (1984).

An alternative, plastic analysis is given by Jewell and Pedley (1990a). The stress distribution considered previously and given in Figure 2.10 is used to obtain the limiting bearing stress acting on the reinforcement (refer to Appendix 2.4 for further details):

\[
\sigma_p = \frac{2V_{\text{max}}}{Dl_s}
\]  

(A2.3 iii)

The limiting bearing stress is then related to the soil strength by considering a modified form of the stress characteristic field originally given by Prandtl (refer to Chen, 1975), to provide a mechanism for an upper bound solution for the bearing capacity of a soil loaded by a strip footing. The bearing capacity of a footing on soil is traditionally calculated using a superposition method with three components (Terzaghi, 1943). The ultimate bearing capacity, \(q_0\) is given by:

\[
q_0 = c N_c + q N_q + 0.5 \gamma B N_y
\]  

(2.34)

where the bearing capacity factors \(N_c\), \(N_q\) and \(N_y\) take into account respectively effects of soil cohesion, \(c\), surface loading, \(q\), and soil unit weight, \(\gamma\). Prandtl's closed-form solution assumes a weightless material which reduces the \(N_y\) term to zero. If a granular material with no cohesion is being considered the equation (2.34) further reduces to:

\[
q_o = q N_q
\]  

(2.34a)

The upper bound \(N_q\) bearing capacity factor is obtained for the Prandtl mechanism (see Figure 2.15a) by equating internal and external energy dissipation taking account of the surcharge \(q\), to give:

\[
N_q = e^{\tan \phi} \tan^2 \left( \frac{\pi}{4} + \frac{\phi}{2} \right)
\]  

(2.35)

The surcharge \(q\) in this case relates to the weight of the soil above the base of the foundation (i.e. \(q = \gamma D\)).

Jewell and Pedley (1990a) make an analogy between the strip footing bearing capacity problem and the case of a soil nail being pushed perpendicular to its axis into the surrounding soil by movement of the active soil mass. The ratio \(q_o/q = N_q\) is replaced by \(\sigma'/\sigma'_n\) to be
consistent with the symbols used, and a modified form of equation (2.35) is given as:

\[
\frac{\sigma'_b}{\sigma'_n} = e^{\left(\frac{\pi + \phi}{4}\right) \tan^{-1}\left(\frac{\pi + \phi}{2}\right)} \tan\left(\frac{\pi + \phi}{2}\right) \tag{2.36}
\]

Equation (2.36) was originally proposed by Jewell et al. (1984) for assessing the lateral bearing stresses on transverse elements of grid reinforcements in earthfill applications. The mechanism that was considered is shown in Figure 2.15b. No derivation of the equation is given in the original paper but it does provide a lower estimate of \( \sigma'_b \) than using equation (2.35). Two points should be noted when using a modified form of equation (2.35). First, the mechanism proposed by Prandtl develops from the loading of a plane area rather than a curved surface as in the case of the nail; kinematically the Hill mechanism is probably more appropriate for the latter (see Figure 2.15c). Secondly, the loading of the soil is not uniform along the length of the nail, this introduces a three-dimensional aspect to the mechanism. The normal effective stress \( \sigma'_n \) for the reinforced earthfill case is taken to be equivalent to the vertical effective overburden pressure, given as \( \sigma'_n = \gamma z \), where \( z \) is the depth to the grid.

The normal stress, \( \sigma'_n \), in Jewell and Pedley (1990a) is taken to be \( \sigma'_n \left(1 + K_n\right)/2 \) which is the same value as that deduced in Section 2.1.2. This value is a lower limit to \( \sigma'_n \), given that a surface across which bending might take would be located at a distance into the soil mass (e.g. approximately one third the excavation height), the definition given by equation (2.4) would seem more appropriate. The ‘normal stresses’ relevant when considering nail-soil bond strength and bearing capacity, are illustrated in Figure 2.16. Jewell and Pedley (1990a) also suggest that a higher estimate for lateral stress is given by \( \sigma'_n = \gamma \).

An expression for \( V_{\text{max}} \) using the plastic analysis can now be derived from equations (A2.4.iii), (2.19) and (2.36):

\[
V_{\text{max}} = \frac{\pi D \sigma'_n}{4} e^{\left(\frac{\pi + \phi}{4}\right) \tan^{-1}\left(\frac{\pi + \phi}{2}\right)} \tan\left(\frac{\pi + \phi}{2}\right) \tag{2.37}
\]

As the components of equation (2.37) are constant for a given level of nails, a single value of \( V_{\text{max}} \) is obtained, (regardless of \( T \) or \( M \)), and which is limited by the bearing capacity of the soil.

Jewell and Pedley (1990a) compare elastic and plastic analyses by determining typical ranges of \( l/D \) values while varying the subgrade reaction, \( k_n \) and angle of internal friction, \( \phi' \). In carrying out the elastic analysis, equation (2.12) is used whilst for the plastic analysis \( M_{\text{max}} \) is taken to be the moment causing plastic bending \( M_p = \sigma_n D^3/6 \). \( l/D \) is then obtained from equations (A2.4.iii) and (2.36) with their adopted value of \( \sigma'_n \). The conclusion from the comparison is that the range of \( l/D \) values is very similar for both analyses when considering
ranges of $k$, and $\phi'$ likely to be encountered in practice (typically $15 \leq l/D \leq 30$).

The agreement of results from the elastic and plastic analyses is encouraging given their very different approaches. However, the complexity of the mechanism leading to bearing capacity failure of the soil and the assumptions made in developing the analyses should not be underestimated.

2.2.5 Combining all limiting conditions which control soil nail behaviour

In the previous sections the forces, stresses and moments acting in a soil nail, the soil stresses and the interaction between nail and soil are discussed. In order to define possible working states for soil nailing design it is necessary to combine these actions. Schlosser (1982) achieved this by representing them on a common set of axes, in $(T, V)$ space. He defined four criteria that should be respected in soil nail design, as follows.

Criterion 1. The bond strength (skin friction) between nail and soil (equation 2.1, a constant value of $T$).

Criterion 2. The bearing capacity of soil for resisting nail deformations normal to their axes, for use in the case where the nail cuts through the soil; this limits the shear resistance that can be provided by the nail (equation 2.30, a constant value of $V$).

Criterion 3. The capacity of the nail itself to resist shear while being subjected to tensile axial force and bending at the same time (equation 2.27, an ellipse).

Criterion 4. The capacity of the nail to resist shear once it has started deforming plastically by bending; a complex interaction problem as the soil must yield before further bending can take place (equation 2.33, a parabola).

The graphical representation of these criteria in terms of axial and shear force given by Schlosser (1982) is shown in Figure 2.17. The area bounded by their intersections represents the domain within which operating states are possible. The boundary of this area is considered to define the limiting state at the point where the nail intersects the potential slip surface. The necessity of combining the criteria is evident; for example, the parabola representing criterion 4 extends beyond the limiting tensile resistance of the nail $T_{im}$ and so it is necessary to limit the parabola by the ellipse of criterion 3. Curiously, in a later figure given in *Recommandations Clouterre* (1991) the line of constant $T$ representing the limiting skin friction (criterion 1) lies beyond $T_{im}$ implying that the nails will break before skin friction is fully mobilised.

For the purposes of design, the limiting force acting in the nails at each construction level has to be determined. Schlosser (1982) uses the bounding surface defined by the above criteria in combination with a *maximum plastic work rule* to determine the resultant nail forces as described below. His method relies on the *principle of normality* and its relationship to maximum plastic work. For completeness and to clarify the graphical method that follows, a
proof of the normality principle by considering the maximum plastic work is given in Appendix 2.5. Normality is a condition that applies to a body at a limiting state, which when acted upon by a load causes an incremental plastic deformation whose resultant vector is perpendicular to the yield surface, i.e. the plastic potential and yield surface are considered to have the same shape. In the appendix, a plastic material is considered with a smooth yield locus; the principle can equally be applied to a plastic structure whose yield surface has corners and flat faces.

The same principle is therefore used, but in this case considering the components of shear and axial force in the nail and the corresponding incremental nail deformations at yield. These quantities, \((T,V)\) and \((\delta_r,\delta_v)\), are superimposed on the same axes as shown in Figure 2.18a. The main assumption made is that the direction of nail deformation at yield is governed by its orientation in relation to the slip surface. The direction of the deformation vector is taken to be that of a tangent to the slip surface at the nail intersection point. Once the direction of the deformation vector is known, the position where a line at this orientation is normal to the boundary surface can be found. A line can then be drawn to this point from the origin, thus defining the load vector causing maximum plastic work. This vector corresponds to the limiting force in the nail; the shear and axial force components can then be readily obtained. In practice the point where the displacement vector, \(\delta\) is normal to the boundary surface is located by drawing a tangent to the surface perpendicular to the direction of \(\delta\) as shown in Figure 2.18b.

This method thus allows the limiting forces in nails at all construction levels and stages to be determined whilst respecting the governing criteria defining the limits of the soil and nail behaviour and their interaction.

2.2.6 Conclusions regarding the relevance of reinforcement bending stiffness to soil nailing design

This section has sought to explain the approaches used in assessing and analysing the contribution that can be obtained from the mobilisation of shear force and bending moment in a soil nail to improve stability. The analyses are based on components of axial and shear force and bending moment within a nail, the bearing capacity of the soil and their interaction. These components and their interaction have been considered individually using a unified set of symbols to assist with the comparison between the methods of Schlosser and Jewell and Pedley. The source papers to reference on the subject are Schlosser (1982, 1983), Jewell and Pedley (1990a and 1990b) and the discussion by Schlosser (1991).

No comparison or assessment of methods has been made here by working through the analyses using typical values. This exercise was done comprehensively by Jewell and Pedley (1990a and 1990b). Their main criticisms of Schlosser's original works were that:

(i) no relationship between axial force, shear force and bending moment in a reinforcement was
given (e.g. as in equation (2.27));
(ii) there was no expression given to describe the parabolic relationship between the bearing capacity of the soil and a nail deforming by plastic bending (e.g. as in equation (2.33));
(iii) the relationship between axial force and shear force in a nail was obtained from the consideration of a unique Mohr’s circle of stress (equations (2.23) and (2.24)).

Without the correct limiting relationships, the proportion of load that can be carried in shear and bending can be seriously overestimated. Jewell and Pedley show this in graphical form with plots of normalised shear force against axial force for different values of $\phi'$ and constant $C$ as given in equation (2.22): their graphs indicate that $V_{\text{max}}$ can be overestimated by a factor of the order of 10 to 20. The relationships lacking in (i) and (ii) above are now provided in *Recommandations Clouterre* (1991) and were given in Schlosser’s discussion (1991). Regardless of this, the overall conclusion from Jewell and Pedley’s work is that the maximum shear force that can be developed in the reinforcement is always a very small proportion of the axial capacity of the bars. This is the case even when the nails are orientated so as to mobilise the maximum shear force and has been verified experimentally by tests carried out in a large shear box (Pedley *et al.* 1990a and b). The same view has also been stated by Gassler (1990) and *Recommandations Clouterre* (1991), both of whom also observe that bending and shear forces are only mobilised at or near failure.

Field tests carried out on nailed-soil structures also indicate that in practice a diffuse pattern of shearing occurs rather than movement across a single shear surface and consequently the explicit deformation required to generate bending moments and hence shear forces does not take place (Gassler, 1990).

For these reasons, most methods of soil nailing design do not take into account any beneficial effects from the mobilisation of shear force or bending stress in the nails. The analysis and design are thus considerably simplified and will tend to be slightly conservative. This seems sensible given the complex and tenuous nature of some of the analyses described, the ambiguity of certain terms and the assumptions necessary.

### 2.3 PRINCIPAL DESIGN METHODS ADOPTED IN PRACTICE

The soil nailing design methods described in this section are all based on ultimate limit state. The method relies on factors of safety so that probability of collapse occurring is acceptably small. Serviceability limit state design requires that a specified threshold deformation is not exceeded or that stresses applied to the construction materials will not affect their durability. As stated in *Recommandations Clouterre* (1991): *the basic application of combining ultimate and serviceability limit state principles is not possible within our present state of*
knowledge. It is for this reason that ultimate limit state design is still the method relied on.

In ultimate limit state design the stability of a proposed structure is assessed using either deformation or limit equilibrium analyses. The former requires the use of techniques such as the FEM which is not practicable for general design at present. Limit equilibrium methods are more commonly employed and take the form of either the determination of disturbing and resisting forces about a potential failure surface (the classic method employed for slope stability analysis) or a kinematic approach combined with limit analysis or yield design theory. In both types of limit equilibrium analysis the resisting forces from the nails are incorporated by considering their resultant vectors at the appropriate points on the relevant potential failure surface.

The classic limit equilibrium method is the more common. Although it is necessary to make some assumptions, the method has been in use for several decades with unreinforced soils (e.g. Bishop's method of slices) and has been used more recently with the design of reinforced earthfill. The kinematic approach is more rigorous than the classic method as no additional assumptions are necessary (for example concerning inter-wedge forces and friction) but there is little experience in using the method which has only been developed for simplified cases.

The implicit assumptions in using the limit equilibrium methods are that: (i) soil and all nails reach their limiting state simultaneously, i.e. there is strain compatibility between soil and nails at all stages and (ii) deformations are small enough such that there are no changes in the geometry of the structure prior to failure.

In any of the design approaches for a nailed-soil structure the stability should be considered at each phase of construction, as often intermediate stages are the most critical.

Assessing the stability of nailed-soil structures by analysing assumed potential failure surfaces is justified in Recommandations Clouterre (1991) by considering cases where failure has occurred under experimental or full-scale working conditions. Four failure modes are identified:
- breaking of nails themselves;
- lack of friction between soil and nails;
- instability during excavation stages;
- overall instability of the reinforced mass.

The first three relate to the internal stability and the fourth, external stability. The mechanisms causing failure, apart from the second relating to lack of interface friction, directly involve a failure surface with the whole nailed-soil mass deforming. The two implicit assumptions given above relating to the simultaneous mobilisation of stresses in nails and soil and the constant geometry of the structure during loading up to failure are also discussed in the
document in the context of observed behaviour. Their justification is more complex but the first assumption is argued by considering mobilisation of the four main resistances:
- tensile resistance of nail;
- pull-out resistance of nail (skin friction);
- shear resistance of soil;
- passive resistance of soil (normal to nail deforming in shear).
These are given in order of increasing deformation required to mobilise each. The deformations required are dependent on the particular material properties being considered. It is noted for example that for mild steel inclusions and sufficiently stiff soil, the tensile strength of the steel and the shear resistance of the soil would be mobilised at similar deformations.

In recent years there has been an increasing use of limit state design methods (e.g. Eurocode 7; BS 8006). At present, in the geotechnical field, only ultimate limit state is considered. Methods for dealing with serviceability limit state have not yet been developed. The arguments for and against the limit state philosophy in geotechnical engineering were discussed when it was first proposed for this field, (e.g. Simpson et al, 1981; Bolton, 1981; and Mortensen, 1982). In the method, partial factors are applied to all influencing components of the design, e.g. loads and soil parameters, the magnitude of which depend on factors such as the method of analysis being used and the economic consequences of failure. Many of the partial factors are determined from parametric 'calibration' analyses where checks are made so that using the limit state approach does not result in lower overall factors of safety obtained from conventional limit equilibrium analysis. The ultimate limit state approach has been adopted for the latest British Standard 8006:1995 for strengthened/reinforced soils and other fills, which is discussed in Section 2.3.4.

Design in a general context has been discussed in this introduction, especially regarding the use of the limit equilibrium method. The specific approaches to design used by the main countries carrying out soil nailing works are now described.

2.3.1 The French method

A preliminary design method is detailed in Recommandations Clouterre which is intended as a tool for the initial estimation of number and length of nails and hence cost for a project. The document stresses that in most cases a full design of the final structure must be carried out subsequently, in France this involves a multicriteria method.

Preliminary design

Rules for preliminary design are based on experience and parametric studies using limit equilibrium methods from which design charts have been produced (see for example Bangratz and Gigan, 1984). Series of charts have been drawn up by Gigan (1986); examples of which
are given in *Recommendations Clouterre*, along with guidelines on how the type of nail used and the geometry of nails and facing will influence the number and length of nails required.

Two dimensionless density indices are defined assuming constant nail dimensions and homogeneous ground conditions. The reinforcement density was the first introduced and is given by $\chi$ where:

$$\chi = \frac{T_R}{\gamma S_h S_v L}$$

and $T_R$ is the lesser of $T_G$ or $T_L$ (elastic limit of reinforcement and ultimate skin friction force respectively). $S_h$ and $S_v$ are the horizontal and vertical spacing of nails respectively and $L$ the total length of each nail. No specific use of $\chi$ is given in *Recommendations Clouterre*, and this definition of $T_G$ is not the same as one given elsewhere in that document. Introducing the symbol $T_R$ seems unnecessary as nails are designed such that $T_L$ is much less than $T_G$; perhaps $T_R$ originated from earlier work with extensible reinforcement used with reinforced earthfill structures. The second index is therefore more appropriate. It is the one used in the Gigan design charts, it is referred to as the nailing density $d$ given by:

$$d = \frac{T_L}{\gamma S_h S_v L}$$

The charts are produced for different $L/H$ ratios and nail inclinations. They comprise a series of curves for values of $d$ from 0 to 1, plotted on axes of $N = c/\gamma H$ against $\tan \phi$. An example is given in Figure 2.19. Values for $N$ and $\tan \phi$ are calculated from which an unfactored value of $d$ is obtained; a factor of safety (on $c$ and $\tan \phi$) is usually applied and a modified value of $d$ obtained as shown in the example chart. Using the expression for $d$ given by equation 2.39 the value of $T_L/S_h S_v$ is obtained; $S_h$ and $S_v$ are then selected such that $T_L$ is acceptable.

In the analysis used to produce the charts, consideration is only given to circular potential slip surfaces and bending is not taken into account. In most instances it is essential to carry out a comprehensive analysis and design following a preliminary design. The only exceptions quoted in *Recommendations Clouterre* are cases where the maximum height is less than 5 m, the geometry and ground conditions are not complex, and where there is no surcharging or nearby structures that might be affected by adverse deformations.

**Use of the multicriteria method**

Having established the proposed geometry and nail dimensions and density, the internal and external stability of the structure should be checked. This is usually carried out using a computer program TALREN according to *Recommendations Clouterre* (1991). Potential failure surfaces formed by a circular arc are usually considered and stability is checked using a method
of slices with each slice incorporating a nail (*i.e.* a classic limit equilibrium method). Failure surfaces outside the nailed mass as well as those intersecting nails should be considered.

Schlosser (1982) and Blondeau *et al.* (1984) report that the slice method used in the TALREN analysis is based on that of Bishop or Fellenius. These are the conventional methods where both normal and frictional interslice forces are taken into account and force equilibrium for each slice is considered in a direction perpendicular to the base of each slice. Summing all slice forces allows the overall factor of safety to be established. It is usually assumed that the *summation* of interslice forces is zero and so they are not considered individually. This has been shown to be reasonable for cases where the central angle of arc forming the slip surface is small, which is likely for most nailed excavations. No mention is made of how nail forces at slice boundaries are taken into account (some slices are likely to have more than one nail passing through them, in addition to the nail passing through the centre of the base of the slice). The only nail forces taken into account are those on the potential failure surface, it therefore seems likely that the summation of interslice nail forces is also taken to be zero.

In *Recommandations Clouterre* (1991) it states that *the simplified form of Bishop's method is the one in which the additional assumptions give the most realistic results.* In Bishop's simplified method, forces are considered parallel to the slices (vertical in most cases) which eliminates consideration of interslice normal forces. Interslice frictional forces are then assumed to be zero. Again no mention is made regarding interslice nail forces, however their vertical components are likely to be small as nails are generally placed only slightly subhorizontally. In view of this and the uncertainty in determining forces along the nail length, the interslice forces are probably also taken to be zero.

The contribution to stability provided by the nails is calculated using a multicriteria rule combined with the concept of normality and maximum plastic work as described in Section 2.2.5. As it is possible to take into account the resistance of the reinforcements to lateral movement, the method can be applied to both nailed-soil excavations and dowelling of unstable slopes. The forces in the inclusions are incorporated into the calculation by considering the resultant components of shear and tensile axial force in the directions of the normal and tangent to the potential failure surface at the base of each slice involved. Any contribution to stability from compressive nail forces is usually ignored in soil nailing design.

In order to allow for uncertainties in the soil properties, anticipated external and internal loads and approximations in the method of analysis, partial safety factors, load factors and a method factor are applied to the relevant components of the equilibrium equation. Suggested values for these factors are given in Tables III and IV, Chapter 3 of *Recommandations Clouterre* (1991).
2.3.2 The German method

The following description is based on papers by Stocker et al. (1979) and Gassier and Gudehus (1981), which describe an analysis based on a two-part wedge mechanism. A similar analysis is also used by the German company Bauer (Institut für Bautechnik, 1986). The two-part wedge mechanism was adopted following experimental and full-scale tests carried out on nailed-soil walls (reported in Stocker et al., 1979; Gassier and Gudehus, 1981). Different failure mechanisms were studied theoretically by Gassier (1988) with the conclusion that in terms of stability the two-part wedge mechanism is slightly more critical than a single rotational failure and a single wedge mechanism for steep excavations (greater than 80° slopes) in granular soil. However, for cohesive soils with less steep slopes, the slip circle rotational failure mechanism is distinctly less stable. His analyses show that for other combinations of slope, nail length, surcharge and soil plasticity there is not much difference between the two-part wedge and circular rotational mechanisms.

The German method of analysis with a two-part wedge mechanism relies again on limit equilibrium but using a kinematic approach whereby a hodograph and force polygon are constructed to satisfy displacement and load equilibrium, as shown in Figure 2.20. In the Stocker et al. (1979) paper the wedges are separated by a vertical line, the slope of the rear surface of the upper wedge is set to \((\pi/4 + \phi/2)\) and the slope of the lower wedge is varied to give the most critical condition. Gassier (1988) considers arbitrary slopes for the two surfaces where movement occurs adjacent to the soil mass and uses a computer to determine the most critical combination. The surface dividing the two wedges is drawn at an orientation in line with the ends of the nails in the 1988 paper, no explanation being given for cases where the nails are of varying length or where there is relative slip between wedges beyond the end of the nails. In both papers the soil in the calculations is assumed to be homogeneous and no water pressures are considered. Limiting shear stresses mobilised on the surfaces, including that dividing the wedges are calculated using design \(c', \phi'\) values (this is mentioned as in some design methods a reduced frictional value is used for the dividing surface).

The forces in the nails at failure (represented by resultant \(Z\) in Figure 2.20) are determined from pull-out tests; this is the value used for design unless it exceeds the elastic limit of the nail itself. No contribution from bending stiffness is considered.

The global factor of safety is given as the ratio of resisting to activating forces. Later partial safety factors were also applied to the soil parameters \(c'\) and \(\phi'\) and the pull-out resistance. The values of the partial safety factors are determined using a statistical approach considering the distribution of values of the relevant parameters (Gassier and Gudehus, 1983).

The German method is considered in *Recommandations Clouterre* (1991) where attention
is focused on the fact that for rupture taking place exclusively in the wall, the failure surface reduces to a single straight line passing through the base of the wall. It then states that in practice, experiments have shown that the internal failure surface is curved and cannot be approximated by a straight line. If internal stability is found to be most critical, recourse is probably made again to Gassler's work (1988) where the different mechanisms are assessed.

2.3.3 Soil nailing design in the U.S.A.

Soil nailing in the U.S.A. was first reported by Shen et al. (1978), and was referred to as a lateral support system. Bang et al. (1980) describe a method of analysis which is generally considered to be the approach adopted in the U.S.A.; more recent papers indicate that French and German techniques are also used (DFI, 1991).

The method of Bang et al. is a limit equilibrium method where parabolic failure surfaces are used to assess stability. The vertices of the parabolas pass through the toe of the excavation (see Figure 2.21). Bang et al. (1993 and 1996) report that the method has been improved to account for varying soil conditions, geometry of nails and excavation and deep-seated stability; no mention is made, however, in respect of ground water conditions.

In the case where the parabolic surface extends beyond the ends of the nails, the analysis is carried out by considering two blocks of soil, similar to the Stocker (1979) approach. The dividing surface is taken to be a vertical line at the extremity of the nails, which taken to be of equal length. Forces between the blocks (wedges) are determined using a coefficient $K$ defining the ratio between horizontal and vertical effective stresses, taken to be 0.4 and 0.5 for granular and cohesive soils respectively.

A single factor of safety $F$ is applied to $c'$ and $\phi'$ for the soils and also to the limiting tensile force in the nail, $T_G$ or $T_L$. The value of $F$ is equal to the global factor of safety, calculated by comparing the component of the total resisting force in the direction of the driving force with the magnitude of the total driving force.

2.3.4 The method adopted in the U.K.

Soil nailing in the U.K. has been used on relatively few jobs in comparison with the countries considered previously above. Despite this there is now a definitive approach for soil nailing design in the U.K. given in the DoT Advice Note HA68/94, Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing. The method is similar to the German one in that a bilinear two-part wedge mechanism is analysed using limit equilibrium. There are differences which have arisen from the way HA68/94 was developed. Jewell (1990) developed a design approach from his work on reinforced earthfill structures, using a two-part wedge with no friction between the wedges; this was implemented in his program NAIL-Solver. He later produced a set of design charts for reinforced earthfill which
utilise log-spiral failure surfaces; these give more economic designs than the two-part wedge mechanism. Research carried out during the period of the drafting of the current HA68/94 method showed that the benefit from using the log-spiral surface could be incorporated into the design by assigning a friction angle of $\phi/2$ to the vertical inter-wedge surface (Love and Milligan, 1995).

The first stage of design involves establishing the mechanism requiring the maximum stabilising force $T_{\text{max}}$ to be provided by the nails. This is done by assessing overall stability for different wedge geometries and orientations looking for the most unstable condition, and is usually carried out using a computer. The number of nail levels is obtained by dividing $T_{\text{max}}$ by the available nail pull-out resistance which also fixes the length of the uppermost level of nails. The reasoning for this is that the uppermost near-surface nail will be the most critical because it is under the lower overburden pressures. In the second stage of design consideration is given to a limiting mechanism requiring no reinforcement which is referred to as a $T_o$-mechanism, there being an infinite number of these mechanisms bounded by a $T_o$ locus as shown in Figure 2.22. The $T_o$-mechanism where the locus intersects the baseline (corresponding to a line drawn from the toe of the excavation at the same inclination as the nails) is the one considered when selecting the length of the lowest nail, it is known as the $T_{\text{in}}$ mechanism ($T_o$ at the base). The process is in effect equivalent to calculating the width of a gravity retaining structure. The lengths of the intermediate nails are set proportionately between the uppermost and lowest nail lengths. As mentioned in Section 2.1.2, a check should be carried out for the possible case where the lengths of the nails beyond this ‘maximum stabilising force’ failure surface are greater than those in front of it. In this instance pull-out will occur in the front section of the nails. In HA68/94 it is suggested that in such a case, reliance can be placed on the shotcrete facing to prevent front face pull-out but that where there is no facing, the bearing capacity of the front face waling plate should be checked. Upper- and lower-bound solutions for checking this are given in Appendix E of HA68/94. Care should be taken with the approach for the following reasons:

i) the facing of a nailed-soil structure (shotcrete or panel) is not always intended to be structural, its primary purposes are to support local areas of instability between nails, prevent erosion and resist frost action;

ii) the soil stresses near the face will be lower than those further back as a result of the stress relief from the excavation works (it is recognised in HA68/94 that vertical stresses might be reduced because of sloping of the face).

The provision of an extra line of nails near the top of the excavation (the critical area identified by the $T_{\text{max}}$ mechanism) would result in shorter nail lengths being required which
would then remove doubt about the structural integrity of the facing and the bearing capacity of the near-face soil.

The depth to each nail layer is given by a parabolic relationship involving the total depth of the excavation and the number of levels in such a way that vertical nail spacings decrease with depth. Referred to as the optimum vertical layer spacing in HA68/94, this is based on the earth pressure distribution with depth; therefore the density of nails increases towards the base of the excavation. The effectiveness of such a spacing in a nailed-soil structure is questionable as the greatest deformation takes place at the top of the wall; it also means that variable cut depths will be necessary. The approach probably originates from consideration of reinforced earthfill construction or cutting an existing slope at a steeper angle. In the former the greatest deformations are at the base and in the latter a greater proportion of soil is removed from the lower level of the slope.

The method is able to take ground water conditions into account by using the pore pressure parameter \( r_u \). The skin friction is calculated using an approach such as described in Section 2.1.2, although the document recommends that pull-out tests are carried out to verify the values.

HA68/94 suggests that soil parameter values be chosen as those at critical state (i.e. large displacement) for both soil within the mass and at the interface of the nails. No partial safety factors are used; the only exception is for granular materials where using critical state values is deemed to be unconservative. In this case peak values of \( c' \) and \( \phi' \) are factored or, alternatively, an estimate of the plane strain value of \( \phi'_c \) may be based on plane strain values of \( \phi'_{peak} \) and \( \psi \) measured in the shear box using the expression \( \phi'_c = \phi'_{peak} - 0.8\psi \) (Bolton, 1986). No partial load factors are applied either.

The design method given in HA68/94 is equally applicable to reinforced earthfill structures (from where it has its origins). A concise summary of the method with worked examples has been given by Love and Milligan (1995).

BS8006:1995, a Code of Practice for Strengthened/reinforced soils and other fills is now published. This document covers the design of earth retaining structures using techniques such as reinforced earthfill and soil nailing. Limit state design principles have been adopted. The subject of the design of reinforced in situ soil is covered in two chapters: walls with facings within 20° of the vertical (Chapter 6) and slopes at lesser angles (Chapter 7). In the chapter concerning walls there is a short section on soil nailing, but no details are given, only reference to the Clouterre work and a state of the art paper by Gassler (1990). It states that: Soil nailing is widely used in continental Europe and the United States for the construction of either temporary or permanent walls. At present the technique is not widely used in the United
Kingdom. In the chapter on slopes, general equilibrium equations are given expressing the components to be considered in calculating disturbing and restraining forces and moments. Reference is again made to papers describing French, German and American techniques in order to provide details for analysis. In the methods proposed, provision is made for taking the resistance derived from bending and shear of the nails into account. No direct reference is made to Jewell and Pedley’s work in this section, although their 1990a paper is listed in the references. Given the case presented by Jewell and Pedley (1990a) and the discussions made by others which are summarised in Section 2.2 (Section 2.2.6 in particular), it is surprising that this component of resisting force has been included in the new code. However, the document does provide the first steps of a rational design methodology for nailed-soil structures.

2.3.5 Computer programs available for soil nailing design

The nature of the most widely used methods of analysis for nailed-soil structures is similar to that of slope stability where many potential failure surfaces have to be considered before the critical one is found. The use of a computer to carry out such iterations is therefore logical and of great benefit in terms of time.

The purpose of this section is to list, at the time of writing, the main programs that have been developed, describing their method of analysis and capabilities: it is not a comprehensive summary of the subject. No details are given in this section on programs which use numerical analyses (e.g. FDM, FEM, BEM). The programs are listed in a roughly chronological order of when they were developed.

CLOUAGE is referred to by Bangratz and Gigan (1984) as a simplified program based on a modified Bishop analysis taking only tensile forces into account. It was used to generate charts used for preliminary design, as discussed in Section 2.3.1.

TALREN was developed at Terrasol by Schlosser in the late seventies specifically for soil nailing design. The original method is described by Blondeau et al. (1984). Recommandations Clouterre (1991) give more up-to-date details of the analysis used. It is even now probably the most powerful program available as it can also be used to analyse reinforced earthfill and dowelled structures and it allows for variation in soil and ground water conditions.

The method of analysis is that described in Section 2.3.1. The program takes bending stiffness into account which is relevant to soil dowelling of unstable slopes, but careful thought is required when designing nailed-soil structures, as discussed in Section 2.2.6.

NIXESC is mentioned in Recommandations Clouterre (1991) as having been developed by the École Nationale des Travaux Publics de l’État and being essentially very similar to TALREN as it utilises the multicriteria technique.

PROSPER is also similar to TALREN, but was developed by the Laboratoire Central
des Ponts et Chaussées (LCPC) and is described by Delmas et al. (1986). The program allows displacements to be determined, but only in the direction of the potential failure surface. This aspect is therefore more applicable to unstable slopes than nailed-soil structures. Parametric studies using PROSPER have been performed by De Bernadi et al. (1993) and comparisons between the previous three programs are made by Gigan and Delmas (1987).

**AMANDINE** was developed at LCPC by Fau during his doctoral research there. It is a limit equilibrium method which considers a bilinear failure mechanism and only tensile nail forces. Moussy (1990) assessed the program and noted that the factor of safety, nail distribution and inclination are given as input data; the force in the nails and their length and dimensions are then calculated. He points out that the main shortcoming of the program is that the nails are assumed to be of infinite length during the initial calculation and that once the design lengths are determined it is not then possible to check whether there are any more critical failure mechanisms with the proposed layout.

**NAIL-Solver** was developed by Jewell (1990); the method of analysis used in the program is detailed in Section 2.3.4. The program has now been superseded by Reactiv which is described below.

**STAR** is described in *Recommandations Clouterre* (1991): it was developed by Anthoine at the Laboratoire de Mécanique des Solides and is based on yield design theory, considering log-spiral failure surfaces and only tensile nail forces. At that stage of development it could only be used for simple geometries, soil and ground water conditions.

**Snail** is the program used by the Californian Department of Transport for soil nailing. It uses bi-linear wedge analysis for failure planes exiting at the toe of the wall and a tri-linear wedge for those developing below the wall; it uses fully balanced force equilibrium with inter-slice forces included, see Martin (1997).

**Reactiv** has been developed at the Geotechnical Consulting Group by Bond to provide a computer program for design as specified in the DoT Advice Note HA68/94 (Section 2.3.4). It can therefore be used for both reinforced earthfill and soil nailing design. It also includes an extensive database of soil parameters. Further details on the program are given by Love and Milligan (1995).

**CRESOL** has been developed at the Cardiff School of Engineering by Bridle and Davies (1997). The program incorporates the method of analysis originally proposed by Bridle (1989), based on the results from a large shearbox test. The program allows a local analysis of the nails to be made at both intermediate and ultimate states and utilises partial safety factors as recommended in BS8006: 1995.
2.4 ALTERNATIVE APPROACHES TO SOIL NAILING ANALYSIS AND DESIGN

The methods of soil nailing design discussed so far are those currently used in practice, all of which rely on limit equilibrium analysis. In this section alternative methods are considered, some of these are well proven, but more sophisticated approaches such as the finite element method, others are new developments which are still being refined.

One of the main aims of some of the alternative approaches is to provide more information about the deformation of nailed-soil structures under working loads. As such they therefore strive towards design at the serviceability limit state. Many independently proposed design approaches have been published which have similarities to current methods. Detailed discussion of these methods is outside the scope of this study. In the following sections brief description of the salient points of some of the methods are given; references are provided for further information.

2.4.1 Finite element method

The finite element method (FEM) is a well proven technique for analysing geotechnical boundary value problems. In practice it is usually reserved for when there is a complex problem with perhaps variable ground conditions where various constitutive models are appropriate and where information on deformations is required. It could therefore be used for serviceability limit state design. The main drawbacks with the method are the time and cost involved with setting up the mesh, correct boundary conditions and appropriate constitutive models and the numerical solution of the system matrices. Nevertheless the FEM is becoming much more widely used with the increasing developments in computer technology.

One of the uncertainties with soil nailing analysis is how to model conditions at the interface between the nail and the soil. This subject is discussed in Section 4.4; usually interface elements are implemented along the nail boundary, and are governed by a different constitutive model from that of the soil.

One example where the FEM has been used for analysing nailed-soil structures is given by Shen et al. (1981a). A plane-strain analysis was carried out using a composite model obtained by the direct combination of the element stiffness matrices of the constituent materials. The bond stress was modelled by Coulomb’s failure criterion assuming that adhesion was maintained even if slip occurred and that normal stress is equivalent to the overburden pressure. The study highlighted which factors were important to consider in design, and the results from the analysis were what provided the basis for adopting the parabolic failure surface in the U.S.A. design approach. Charts were produced showing relationships between depth and depth-to-length ratio and between angular distortion and lateral displacements at ground level at the top of the excavation. Results from the analysis were compared with movements measured in two nailed-
soil structures and found to be in agreement (Shen et al. 1981b).

A case where the finite difference method of analysis was used is given by Gnilsen et al. (1991) and is described as part of an advanced soil nailing design. The overall stability was checked using conventional methods and the working deformations were obtained from the finite difference analysis, but no details are given of the analysis. The resulting contour plots of stress and displacement appear reasonable, although there are no field monitoring results of the designed structure to compare with the predictions.

2.4.2 Methods incorporating nail bending and shear stiffness

Two methods are described which have a similar approach to that of the multicriteria analysis proposed by Schlosser (1982, 1983), but with further refinements.

**Kinematic limit analysis**

This design approach, was initially introduced by Beech and Juran (1984), and has subsequently been developed by Juran in a series of papers (Juran, 1987; Juran et al., 1988; Juran et al., 1990a and b; Juran and Elias, 1990; and Elias and Juran, 1991). The method is based on a limit analysis method where a kinematically admissible displacement/failure mode is considered in combination with a statically admissible limit equilibrium solution. The method examines the local stability of each level of nails by considering slices which are parallel to and encompass each nail as shown in Figure 2.23.

The main design assumptions stated are that (i) failure occurs by rotation of the active zone along a log spiral failure surface; (ii) at failure, the locus of maximum tension and shear forces in the nails is coincident with the failure surface; (iii) the quasi-rigid active and resistant zones are separated by a thin layer of soil at a limit state of plastic flow and that the shearing resistance of the soil, defined by Coulomb's failure criterion is entirely mobilised along this surface (as for conventional method of slices); (iv) the horizontal components of the interslice forces acting on both sides of a slice incorporating a nail are equal; and (v) the effect of a slope (or horizontal surcharge) at the upper surface of the nailed-soil mass on the forces in the inclusions decreases linearly along the failure surface (refer to Figure 2.23).

Nails are classified in three groups: flexible nails that can only sustain tensile forces; rigid nails which can sustain tensile and shear forces without deforming; and nails with a finite bending stiffness which governs their deformation and hence capacity to sustain shear force.

**Approach proposed by Bridle**

An approach similar to that of Schlosser has been proposed by Bridle (1989). The method involves a limit equilibrium approach using a log spiral failure plane. It has been developed at Cardiff University as part of a research programme investigating the bending and shear resistance of soil nails using a large shearbox. This has been carried out in parallel with
the development of a machine for installing nails ballistically. As such the design method is reported to be intended for fired nails that are installed ungrouted, at a steep angle and which only penetrates beyond the slip surface to a limited extent (Bridle and Davies, 1996).

There are several subsequent references giving details of its development (Bridle and Barr, 1990a and b; Myles and Bridle, 1992). In the latest publication (Bridle and Davies, 1997), the analysis allows a local analysis of the nails to be made at both intermediate and ultimate states. This has been checked against a case study in the literature and shows good agreement.

2.4.3 Rigid-block global analysis

The two methods of analysis described in this section are grouped together as they model the reinforced mass as a single unit.

Method of homogenisation

The design of reinforced earthwork structures by the method of homogenisation was discussed by Buhan and Salençon (1987). The method involves reducing the problem of analysing a composite structure comprised of different materials to that of an equivalent structure of one homogeneous material, but with anisotropic properties.

The approach was originally proposed for analysing reinforced earthfill structures where the reinforcing layers are flat, continuous and usually regularly spaced, thus being well-suited to such an idealisation. In the case of structures with intermittent reinforcing elements, such as soil nails, the ground mass is split up into a series of ortho-rhombic cells each of which is referred to as a representative base cell in the Buhan and Salençon paper, see Figure 2.24. The process of defining a macroscopic behavioural criterion is the fundamental aspect of the approach. The criterion is determined from the characteristics of the individual components of the system (soil and reinforcing elements). Only the reinforced part of the ground is reduced to a homogeneous medium; the ground beyond the effective zone remains unchanged.

Once the reinforced mass has been reduced to an equivalent anisotropic homogeneous mass an overall stability analysis is performed. There are therefore two distinct stages involved with the application of the homogenisation method.

i) The determination of the macroscopic characteristics of the reinforced soil using the known characteristics of its constituent materials.

ii) The solution of the equivalent homogeneous stability problem using the equivalent characteristics.

In the Buhan and Salençon paper, a generalised solution is given, where the characteristics of the soil and reinforcements are represented by functions, $G_r$ and $G_n$, describing their bounding surfaces, which are not explicitly expressed, but $G_r$ is said to define the domain of admissible stress, $\sigma$. The domain representing the macroscopic bounding surface, denoted
$G_{hom}$, is obtained by considering all admissible macroscopic states of stress, $\Sigma$, for the intermittent stress fields ensuring that equilibrium and possible states defined by the bounding surfaces $G_s$ and $G_r$ are respected.

In considering stability, a simplified approach is used, which is described again using a generalised mathematical formulation which is beyond the scope of the present summary. Two methods of analysis are covered which are expressed in terms of a single load-related parameter $Q$ and its extreme value $Q^{hom}$. One method involves an internal static approach which yields a lower estimate of $Q^{hom}$, and the other an external kinematic approach which leads to an upper estimate of $Q^{hom}$.

The generalised mathematical formulations have not been studied detail. In principle, the concept of representing a composite structure by an equivalent homogeneous one is attractive regarding its stability and deformation analysis.

The crux of the method, as stated in the paper, is the representative simulation of the macroscopic behaviour of the structure. In the section discussing its determination, the soil-inclusion contact is assumed to be fully frictional. In the paper, attention is drawn to three other conditions concerning the method, namely that: (i) it is not able to take local stability into account, only global stability; (ii) the reinforcing inclusions are assumed to be arranged in a regular manner, and (iii) it is essential that the spacing of the reinforcement can be considered as small compared to the overall dimensions of the works.

In conclusion, the method could be useful for carrying out preliminary designs, but is probably too simplified by necessity for detailed design. Given the present advances in computer technology, more accurate and detailed analyses could be carried out more effectively and economically by utilising the FEM with advanced constitutive models.

**Method of yield design**

A method of yield design for reinforced soil structures has been proposed by Anthoine (1989 and 1990). It is still in a process of development but is reported as being mechanically rigorous, and flexible in its application, more so than the method of homogenisation described above.

The method incorporates both 'mixed modelling' techniques, using upper and lower bound methods within the framework of yield design theory. Anthoine (1989) concludes that the mixed modelling provides an adequate means of describing soils reinforced by inclusions (expressed one- or two-dimensionally) but that the lower and upper bound methods are strongly influenced by the way in which the reinforcement is considered.
2.5 CONCLUSIONS REGARDING ANALYSIS AND DESIGN

In this chapter most of the current methods of analysis and design are described. The first section of the chapter deals with the calculation of the bond strength generated from tensile forces in the inclusions. This is followed by a method for assessing the contribution to stability of the mass from the development of bending and shear resistance in the inclusions in the vicinity of the potential failure surface.

The degree of complexity in the analysis of these two components is marked. Certain assumptions are necessary for determining the bond strength, but these can be substantiated by experience from the analyses of other geotechnical applications such as piles or reinforced earthfill. Further confidence can be gained from pull-out tests carried out prior to and during the construction works.

Incorporating bending and shear resistance into the design is far more complex and requires several assumptions and non-trivial analysis. A multicriteria type approach is required that can accommodate combinations of tensile and shear force and moment whilst satisfying the bearing capacity limitations of the soil.

Intensive research has been undertaken to quantify the contribution of these two components. The conclusion is that ignoring shear and bending resistance results in only slight over-design, Schlosser (1991) states that the principal resistance remains the tensile force exerted in the nails and that the bending and shear resistance have always a limited effect on the global safety factor (less than 15 %), generally it is considered to be much less. Moreover, these benefits are only gained after large displacements when the nailed-soil structure is approaching collapse.

The decision as to whether to incorporate bending and shear resistance should perhaps be based on the acceptability of large deformations. As suggested in Chapter 1, it seems appropriate to consider two separate cases. The first where the nails are supporting excavations or steep newly cut slopes where stress relief at the face is taking place and the nails are put into tension with negligible development of shear forces or bending. Secondly, where the soil mass contains a pre-existing or potential slip surface, no stress relief is taking place and stability is to be improved and movements arrested by the development of shear forces in the inclusions. In the second case, the reinforcing elements are considered to be acting as dowels.

In the methods discussed there is no provision for design at serviceability state. Movements of the reinforced mass are usually restricted by limiting the forces in the nails to a set value established from pull-out tests, or by adopting certain construction techniques.
CHAPTER 3

Previous and current research philosophies

The purpose of this chapter is to review previous philosophies of research into soil nailing and introduce the approach used for this study. The cases presented in this chapter act as examples of the approaches used by others. They do not represent an exhaustive collection of all previous research into soil nailing. Similarly, a detailed appraisal of each individual study into different aspects of soil nailing is not intended.

Most previous research has been carried out in three main areas: experimental model testing; full-scale field trials; and numerical analysis. Field observations of the construction of commercial nailed-soil excavations and their numerical analysis are also included. These are then further sub-divided into more specific studies. Under some of the headings, research into closely affiliated subjects is also included, for instance reinforced earthfill and piling.

Having discussed previous philosophies, the research described in this thesis is introduced. Details are given about conditions believed to be acting in the field and the boundary conditions used to simulate them experimentally. The boundary conditions used to model the experimental apparatus numerically are also covered.

3.1 EXPERIMENTAL STUDIES

The scope of research under laboratory conditions is divided into five areas. Small-scale models of walls reinforced with soil nails, loaded under static conditions are intended to provide information on overall modes of deformation and failure. So also are most of the models that have been tested in centrifuge apparatus.

Element type tests usually provide information on the behaviour of a single nail loaded in such a way as to replicate a particular type of field loading. These are discussed separately under headings of nails loaded axially, such as pull-out tests and transversely, such as shearbox testing.

Specific aspects of behaviour, such as interface friction, are studied using equipment such as the shearbox apparatus or dilatometers.

The above approaches are carried out under conditions where loads and displacements are controlled and measured under known boundary conditions. The nature, mode of deposition and loading history of the materials being tested can usually be specified.

3.1.1 Small-scale model testing

Small-scale model tests are usually performed to provide information on overall modes
of deformation and failure within a soil nailing operation. Various influencing factors can be assessed such as the density of reinforcing elements, their rigidity, the stiffness of the facing, and the connection between facing and nails. Often, the results from the model tests are used to create a theoretical framework (or used to verify one) or are compared with results from numerical analyses.

The research referred to in this section can be divided into several groups depending on which common theme is considered. In simple terms, Gassier and Gudehus (1983), Beech et al. (1984), Mapplebeck (1987), Schwing and Gudehus (1988), and Nishida and Nishigata (1996) investigated various conditions with soil nailing. Saran et al. (1992) and Kodaka et al. (1995) considered reinforced soil without specific mention of soil nailing.

The models usually comprise a reinforced soil mass enclosed within the confines of rigid or flexible boundaries depending on whether they are displacement or stress controlled. The dimensions of the tests considered here are typically 0.5 to 1.5 m high, 0.2 to 1.5 m wide (perpendicular to the reinforcing elements), and 0.8 to 3 m overall length (parallel to the elements). Side-wall friction is usually minimised using lubrication of some sort (e.g. teflon surfaces or greased membranes). Because deformations and failure modes within the mass are often being investigated, a glass side is sometimes provided to allow displacements of marker bands or grids to be observed.

In most small-scale models the soil tested was sand for convenience and uniformity of placing and also speed of testing without having to wait for dissipation of pore pressures. In the study by Nishida and Nishigata iron ore was used as a backfill to provide an increased unit weight and hence active pressure.

Deformations within the models were generally induced either by direct loading at the upper surface (e.g. Mapplebeck, Schwing and Gudehus, Saran et al., Kodaka et al.) or by displacing the front facing linearly or by rotation (e.g. Gassler and Gudehus, Nishida and Nishigata). In the case of Beech et al., failure of the wall was induced by excavating until collapse occurred.

The inclusions were modelled using a variety of materials, installed by different techniques. In some cases the reinforcing elements were placed as the sand was deposited, and in others installed as the front of the wall was excavated. Often the nails were instrumented with strain gauges to allow the development of force in the nails to be determined. In Mapplebeck’s study the nails were installed as the wall was constructed and expanded in place to investigate the possibility of using expanded ‘wedge-pile’ type inclusions (see also French, 1990).

There are several problem areas in small-scale modelling. Often it is not possible to apply uniform pressures over the whole soil mass. Friction along side walls can be significant
if precautions are not taken. Probably the most important consideration is that scaling laws and
kinematical constraints are observed, and if they cannot be then the assumptions made and their
influence should be clearly stated. Particular problems arise with the size of the model nails in
relation to that of the soil being tested, especially if they are to be instrumented. Mobilisation
of friction on nails with surface perimeters of similar dimension to grain size can be highly non-
uniform. This effect along with very small cross-sectional curvature of the inclusions can lead
to non-representative measurements of forces or deformations within the inclusions being made.

Regardless of these drawbacks, the small-scale tests mentioned here provide examples
of where valuable information has been gained on aspects of behaviour such as:
- deformation and failure mechanisms;
- the distribution and magnitude of earth pressures acting against the facing;
- the distribution and magnitude of maximum force in the nails;
- the effectiveness of reducing deformations by increasing the density of nails; and
- the influence of the inclination of the nails and the wall stiffness.

Besides these specific aspects, the studies have shown that models can be used to verify theoretical and numerical analyses.

3.1.2 Centrifuge testing

Centrifuge testing constitutes a specific type of small-scale modelling, where dimension
and time quantities are stepped-up by applying loads at accelerations greater than that of gravity.
Because of this advantage soil nailing in clays can be modelled in reasonable time periods.
Examples of research into the behaviour of slope stabilisation by soil nailing in sand using centrifuge apparatus are given by Davies et al. (1997). Several studies were also carried out as part of the Clouterre project (Recommandations Clouterre, 1991).

Another application using the centrifuge is described by Vucetic et al. (1996) where the behaviour of nailed-soil excavations under dynamic conditions were modelled using a shaker device mounted on the model box. The soil tested in this instance was moist sand. The study provided information on failure mechanisms and qualitative behaviour of different types of nailed-soil excavations under strong shaking.

3.1.3 Element testing by direct loading

The objective of element testing is to investigate a particular aspect of behaviour isolating as many influencing factors as possible and then controlling and varying others to observe their effect. The element testing is divided into two sections covering direct and indirect loading. Initially a brief review of element testing of model piles is given as there are similarities between the development of skin friction on piles and those on nails in the resistant zone. This latter behaviour is often modelled (under laboratory conditions and in the field) using
pull-out tests, which are the next topic.

**Element testing of model piles**

Investigations into pile behaviour by testing of model piles has been the subject of many research programmes. A selection of studies where piles are loaded axially are described here, considering the boundary conditions, method of loading and instrumentation used on the model piles.

Studies of pile behaviour in sands have been made by Robinsky *et al.* (1964), Hanna and Tan (1973), Levacher and Sieffert (1984), and Amira *et al.* (1995), thus covering four decades. There was no control of boundary stresses in any of the tests, one of the primary objectives being to assess forces generated during driving, the walls of the chambers containing the sand providing lateral restraint. In most cases the sand was medium grained, essentially dry and tested in loose to medium dense states.

Robinsky *et al.* were interested in assessing the effects of the shape and volume of driven piles and used internal strain gauge instrumentation to achieve this. Their study has some relevance to the research described in this thesis in terms of their observations on the effect of pile shape and embedment volume on capacity and also the influence of arching. These subjects are discussed in the context of the results in Chapters 8 and 10.

Hanna and Tan investigated the loading in tension as well as compression of long piles in sand which had been pluviated around them. They too recognised the significance of arching and volume change and their effect on the normal stresses acting on the pile.

The tests of Levacher *et al.* were performed in tension after the piles had been installed using one of three techniques: placing sand around the piles, driving them by drop-hammer, or vibro-driven. The model pile used was not instrumented, although a load cell was located at its head. The main conclusion concerns the significance of the placement method (a coefficient is introduced to account for it in determining the lateral stress acting on the pile) and the density of the sand on the ultimate pull-out resistance.

Amira *et al.* 'buried' piles in sand and then also tested them in tension as well as compression. Three methods of pull-out were implemented involving loading from (i) the top, (ii) the tip (via an internal rod) and (iii) from the tip after prestressing the pile. The pile was acrylic and strain gauges were bonded along its length allowing load-transfer mechanisms to be studied. The shaft friction in tension using the three loading methods was compared to that in compression and found to be reduced by 50 to 80 % for loading modes (i) to (iii) respectively. The remaining friction (at least 20 %) they consider to be lost through other phenomena such as arching. They also conclude that the ultimate shaft resistance is mobilised after displacements of 5 to 15 % of the pile diameter and that the ratio of shaft friction to overburden stress is
constant after displacements of 5% of the pile diameter and is uniquely related to the radial stress. The cases of studies of the behaviour of model piles in clay are given by Steenfelt et al. (1981), Chandler and Martins (1982), and Mochtar and Edil (1988). All were performed using kaolin consolidated from a slurry with the tests performed under known boundary stresses.

The research by Steenfelt et al. investigated the effects of pile installation by jacking using a model pile which was instrumented to measure axial and radial stresses and pore water pressures. One of the principal aims of the work was to evaluate the use of an undrained cylindrical cavity expansion model, further work on the subject was subsequently carried out by Francescon (1983).

Mochtar and Edil and Chandler and Martins both report studies into the shaft resistance of model piles in clay, as such the piles were installed with minimum disturbance. The piles were of diameters ranging from 10, 15, 17 and 27 mm. Conclusions from these studies indicate that the interface angle of friction is independent of initial stress ratio of the sample, and overconsolidation ratio. Also the shaft resistance is governed by the interface angle of friction and the normal (radial) stress acting on the pile, hence supporting the use of effective stress approaches to pile design.

The conditions and results from these model tests in sands and clays are comparable with those of pull-out tests for investigating the development of shaft friction on inclusions used in reinforced soil structures. These subjects are referred to in the discussion of the results obtained from this study in Chapter 10.

**Pull-out tests**

Pull-out tests are the traditional means of establishing the bond resistance of reinforcing elements in the field. These tests are performed for most types of reinforcement including anchors, reinforced earthfill materials, and soil nails as a means of establishing design parameters and also to check those adopted once the construction works begin. In order to understand better the processes that take place in situ, several laboratory studies have been carried out to model the boundary conditions imposed in pull-out testing. As in the previous sections, the intention here is to provide a few examples of the methods of approach used rather than an exhaustive review.

Comprehensive studies of the frictional forces generated along reinforcing textiles used in reinforced earthfill have been made to investigate factors such as the influence of surface roughness, stress level, soil density and dilation (Schlosser and Elias, 1978; Schlosser and Guilloux, 1981; Palmeira and Milligan, 1990; and Milligan et al., 1990). Most of these studies relate to tests in sand.
Milligan et al. observe that with the grid type of reinforcement pull-out resistance is almost entirely in bearing against the transverse members. Other researchers have noted that ribbed strips generally provide greater resistances than smooth, for the same reason. Mechanisms taking place against the transverse members are suggested by Jewell et al. (1984), this mode of mobilisation of force is not a consideration for most soil nailing applications except the case of lateral bending as discussed in Section 2.2.

The effects of dilation have been observed both in the field and in the laboratory, elevated values of angles of interface friction have been recorded where dilation is restrained from taking place (Schlosser and Guilloux, 1981). The phenomenon is controlled principally by the soil density, stress level and surface roughness.

Another factor that strongly influences the behaviour of textile reinforcement is its extensibility. Grids that are extensible tend to a progressive type of failure under pull-out conditions, with softer responses being observed at higher stress levels. Rheological and temperature considerations also should be taken into account where possible.

Milligan et al. (1990), suggest that the pull-out resistance of sheet materials can be obtained directly from interface tests using modified shearbox apparatus but that pull-out tests are required for replicating the bearing resistance mode of behaviour. The interpretation of these latter tests is not straightforward and care should be taken to take into account the boundary conditions of the test. Palmeira and Milligan (1989) investigated these effects and concluded that interface friction angles between soil and reinforcement can be overestimated because of friction on the internal front wall of the box in small-scale tests. They suggest that the degree of influence can be minimised by lubricating the front face and increasing the scale of the test.

Pull-out testing in the case of soil nailing is generally considered to model conditions in the resistant zone. Chang and Milligan (1996) modelled a nailed-wall using small scale pull-out tests. They identify a transition zone between the active and ‘self-stable’ zones bounded by lines passing through the toe of the wall at the angle of Rankine active failure and at the angle of repose, given by the internal angle of friction of the soil. They conclude that dilation should not be considered within or in front of this zone, but that beyond it, further into the soil mass, it seems too conservative to ignore its benefit. They also point out that in situ pull-out tests are more likely to relate to conditions in the resistant zone than active or transition zones as the tests are performed without overall soil failure taking place.

Barley et al. (1997b) recognise this condition and suggest alternative methods of field testing, where a standard pull-out test is conducted in the soil in the resistant zone, while ‘push-in’ tests are performed to obtain parameters for the active zone.

A considerable data-bank of pull-out test results have been compiled as part of the
Clouterre project (*Recommandations Clouterre*, 1991), most of which were conducted in the field under full-scale conditions. The effects of restrained dilation have been witnessed in many of these tests and also those performed under laboratory conditions (Schlosser and Guilloux, 1981; Guilloux, 1984; Plumelle, 1987; Chang and Milligan, 1996; Milligan *et al.*, 1997). An apparent coefficient of friction, $\mu^*$ is introduced to express the effect, where

$$\mu^* = \frac{\tau}{\sigma_0}$$

with $\tau$ being the interface shear stress and $\sigma_0$ the normal stress. The real coefficient is given by:

$$\mu = \frac{\tau}{\sigma_0 + \Delta \sigma}$$

where $\Delta \sigma$ is the component of normal stress generated by restrained dilation. Values of $\mu^*$ can be four to six times greater than $\mu$ depending on the initial stress level and density of the sand. Similar effects are reported by Wernick (1977) with reference to the testing of anchors. Further information on the effects of restrained dilation are being investigated by Milligan *et al.* (1997) using instrumented model nails of 100 mm diameter grouted into sands and clays within the confines of a pull-out test apparatus.

### 3.1.4 Element testing by indirect loading

Three modes of indirect loading are considered in this section, the first relates specifically to soil nailing and the latter two to reinforced earthfill applications.

#### Shearbox testing

A number of studies have been performed to investigate the behaviour of soil nails when under a lateral loading. These usually take the form of shearbox tests where the nail (or nails) is installed perpendicularly across the shearing plane and the two halves of the box are moved relative to each other as in conventional shearbox testing. Such studies have been described by Juran *et al.*, 1981; Juran *et al.*, 1983; Marchal, 1984 and 1986; Pedley *et al.* 1990a and b; Barr *et al.* (1991); and Davies and Le Masurier (1997).

The dimensions of the shearbox varies from about 0.5 m square to that employed at the College of Cardiff of 1.5 m by 1.5 m by 3 m. Generally boundary stresses are applied on one plane of the apparatus, sometimes not at all and provision is usually made to prevent rotation between the two halves of the box.

The primary purpose of these studies is to examine the shear and bending resistance of the inclusions and their contribution to the overall strength of the soil. Influencing factors that have been assessed include the rigidity and orientation of the inclusions, their number and interaction, their surface roughness and method of installation, and the stress level and density
These studies were used in the development of the multicriteria method discussed in Chapter 2. Jewell and Pedley (1990a and b) used the results described by Pedley et al. in their papers on the contribution of bending stiffness in the context of soil nailing design, discussed in Section 2.2. More recently, Bridle and Davies (1997) have suggested another method of soil nailing analysis taking shear and bending resistance into account using the results from tests in the large shearbox.

The results from these studies have direct application to the mode of deformation that operates under the conditions of soil dowelling, as defined in Chapter 1. In terms of soil nailing this mode of loading only becomes significant after large deformations when the structure is on the point of failure. The contribution of bending stiffness to overall stability is discussed by Jewell and Pedley (1990a).

**Triaxial testing of reinforced soil**

Conditions in reinforced earthfill structures have been modelled under two modes of indirect loading using triaxial apparatus.

Moroto (1992) and Nakai (1992) placed reinforcing elements in the form of discs of the same diameter of the sample at set intervals along and perpendicular to its axis. In these cases the discs were of paper and brass respectively. The samples were loaded in compression, thus putting the elements into extension radially. The results from these studies indicate that the strength of the sample increases with the number of reinforcing discs, their roughness and tensile resistance. The axial strain at failure is reduced compared to that of the unreinforced sample, thus the discs increased its stiffness.

Tests in extension are described by Shen et al. (1988) where a hollow cylinder sample is set up with a concentric layer of reinforcing material midway within it. The reinforcing material is fixed to the base platen and shearing in extension is performed after isotropic consolidation. Detailed results are not presented, the main purpose of the paper being to emphasise the mode of testing as a means of simulating the loading conditions within a reinforced earthfill structure.

**Plane-strain testing of reinforced soil**

McGowan et al. (1978) present a series of results from ‘unit cell’ tests. In these tests a rectangular sample with a reinforcing membrane placed horizontally at its mid height is loaded in plane-strain, by constraining lateral movement of its long sides by fixed rigid lubricated platens. The sample is loaded at its upper surface under an all-round pressure. In this way the reinforcing material is indirectly put into tension.

Several conclusions were drawn from the study by McGowan et al. relating to the
limiting tensile rupture strains of the reinforcing materials tested, and their orientation. Inclusions termed ‘relatively inextensible’ have tensile rupture strains less than those that would occur in the sample without reinforcing. Their onset of rupture limits the possible improvement to the strength of the system and increases its brittleness. Conversely, ‘relatively extensible’ inclusions, whose limiting rupture strains are greater than those in the unreinforced soil, cannot rupture, always strengthen the soil and reduce brittleness.

Tests were performed with the inclusions placed at different angles to the horizontal, the results from which confirmed that soil improvement is optimised by placing the inclusions along the directions of principal tensile strain and in the zones of maximum tensile strain of the soil alone, under the same operational conditions.

A development of the unit cell test, known as the ‘automated plane strain reinforcement’ (APSR) cell is described by Whittle et al. (1992). In this cell a rectangular sample is loaded in the same way as the unit cell but the reinforcement projects from one side of the apparatus and is connected to a load cell and displacement transducer. Its position can thus be maintained and the force in it measured as the test proceeds. Fixing one end essentially reduces the boundary conditions to those of half an element loaded as in the unit cell set-up.

3.1.5 Conclusions regarding experimental modelling

Several approaches to the modelling of different aspects of soil nailing conditions have been described, giving details of the boundary conditions imposed and some of the conclusions drawn from the studies.

The aim of the research discussed in this thesis is to study the interface resistance of soil nails under the conditions of a nailed-soil excavation. To date no studies have been carried out under conditions of an inclusion being loaded indirectly by the changing soil state acting around its boundary as in the case of a nailed-soil excavation or a pile loaded by heaving ground. Also there is very little information on the changing conditions acting at the element interface during loading, particularly concerning radial stresses.

The studies described above provide a framework from which the most suitable approach can be formulated. Detailed information on conditions at the interface under different controlling parameters is probably best achieved using element tests. The boundary conditions to impose can be decided from those thought to be acting in practice, these have been described in Chapter 1. This subject is covered after further consideration of soil nailing conditions obtained from observations of large or full-scale works and numerical studies.

3.2 LARGE SCALE TESTING AND FIELD MONITORING

The case studies given in this section are divided into three groups: large scale model
tests constructed under field conditions but not to full scale; full-scale tests; and field monitoring of construction works.

3.2.1 Large scale model tests

Studies under these conditions are described by Sawicki et al. (1988), and Kim et al. (1996). In the former study an experimental wall 6 m deep and 7 m wide was excavated at an angle of 78° and reinforced with 22 drilled and grouted nails of 3 m length. Failure in the wall was induced by loading the crest with 'concrete plates'. Several of the reinforcing elements were instrumented with strain gauges. The results from these nails are presented and compared with those from two types of analysis. The first involved modelling conditions in a similar way to the homogenisation method described in Section 2.4.3, with the soil assumed to be homogeneous but transversely anisotropic, the second was a limit plasticity approach. Both methods showed good agreement with the observed behaviour. The nails were anchored to the facing and as a consequence the maximum force in the nails develops towards the head of the nails. No information is given about the deformation of the wall or crest.

The model described by Kim et al. was 2 m deep and wide and was excavated in steps and a total of 16 drilled and grouted nails installed at 0.5 m spacings. The wall was loaded to failure by applying load to its crest. Several of the nails were instrumented with strain gauges and pressure cells were installed at positions behind the wall at three levels and orientated to measure vertical, diagonal and horizontal stresses.

The results from the pressure cells indicate a decrease in horizontal stress during excavation, which is then regained during load application. The vertical and sub-vertical stresses remain constant during excavation and then steadily increase with applied loading. The nails show a classical force distribution (as indicated by the measured strains) with maximum values at a distance of about a third of the wall height in from the face at the top of the excavation, the maxima locations then move in towards the face with depth. Cracking was observed in the nails which increased towards the top of the excavation where forces were greatest.

3.2.2 Full-scale tests

There are two main series of full-scale tests that were carried out under the German and French research projects Bodenvernagelung and Clouterre respectively. The former is described by Stocker et al. (1979), Gudehus (1982), Gigan (1986) and Gässler (1992). Seven walls were constructed 6 m deep with five or six levels of drilled and grouted nails, three in medium dense fine sand, one in a stiff heavily over-consolidated clay and the remaining three in layered clay and sand or a silty clay. The results from the sand and the clay tests are given in the above references.

In two of the sand tests 'plane-strain' conditions were imposed by forming bentonite
filled slots 40 mm wide either side of the tested length, as shown schematically in Figure 3.1. One of these was loaded at the edge of the crest and the other with the load applied further back. The resulting failure mechanisms are shown in Figure 3.2a and b. Various results from the crest loaded test are shown in Figure 3.3, including wall and crest deformations, forces in the nails and earth pressures behind the wall at different stages of loading. The distribution of axial forces generated along the reinforcing elements imply that the nails were rigidly connected to the facing as maximum forces are generated there. During the initial stages of loading the nails at the top of the wall carry more load than those towards the base, the trend is reversed as loads approach failure. The results from these tests are strongly influenced by the connection of the nails to the facing and the method of loading, little emphasis is given to the behaviour of the wall and the development of nail forces during construction.

The three full-scale tests performed as part of the Clouterre project are described in several papers (e.g. Plumelle, 1987 and 1991, Schlosser et al., 1992) and Recommandations Clouterre, 1991. The tests were carried out in fine Fontainebleau sand which was initially excavated, replaced and recompacted using set compaction control. The walls were 6 to 7 m high with nails at roughly one metre levels. Three methods of inducing failure were adopted. (i) The grouted nails were designed such that the wall had a low factor of safety of about 1.1. After completion of construction the sand was flooded, reducing effective stresses and increasing the unit weight. Overall failure through the nailed mass resulted. (ii) The wall was initially excavated to 3 m depth with a level of nails at each step of 1 m. Excavation then continued with shotcrete only and no nails. Failure occurred after a further three steps, initially at the base of the wall, followed by collapse of the soil above. (iii) Telescopic nails were used such that after construction the nail lengths could be reduced. This induced an overall failure with the slip surface behind the retracted nail lengths.

The test set-up and dimensions for the first wall (load condition (i)) are shown in Figure 3.4. The horizontal displacements of the wall after construction and creep, and the development of maximum axial force in the nails during the phases of construction are shown in Figure 3.5a and b. The results are more typical of observations made on other full-scale structures since then (see Recommandations Clouterre and the case studies given in the next section). Wall displacements are maximum at the top and nail forces increase with the maximum values away from the facing as excavation proceeds. The line of maximum force in the nails is almost vertical over most of the height of the wall by the end of construction.

Besides these two main series of tests, there is another study excavation reported by Shen et al. (1981b) which was carried out at a landfill site on the campus of the University of California, within alternating layers of silty fine sand, sandy silt and silty clay. The study was
to provide data with which to validate a method of numerical analysis (see Shen et al., 1981a). The excavation was about 9 m deep with a minimum width of about 12 m and was reinforced with five levels of grouted nails each at 1.85 m apart.

The monitoring comprised the measurement of surface and sub-surface ground movements and of axial force in the central vertical line of nails using strain gauges bonded to their reinforcing steel. The progressive development of horizontal surface and sub-surface displacements are shown in Figures 3.6a to c (note difference in scales between graphs a and b). The lateral movements occur over the full height of the excavation decreasing with depth and distance from the face, in a similar manner to those seen in the Clouterre experiment. The development of force in three of the nails is shown in Figure 3.6d. The distributions are generally uniform with a maximum at the face decreasing towards the end of the nails, indicating, as with those of the Bodenvernagelung experiment, that the nails were well connected to the facing.

3.2.3 Field observations

The above experimental studies provide useful insight into the behaviour of nailed-soil excavations. These are now supplemented further by three examples of field observations on commercial nailed-soil excavations. In some respects field monitoring provides information that is more representative than that from testing. This is because the cases presented are generally operating at a serviceability state, excavated to greater depths allowing full development of forces in the upper nails and have not been taken to failure by artificial means.

Case study 1

Stocker and Riedlinger (1990) present the results from monitoring over a period of ten years after construction, of a nearly 15-m deep excavation in Stuttgart, Germany as shown in Figure 3.7a. A section showing the arrangement of nails and the soil profile of silt underlain by Keuper Marl is given in Figure 3.7b. The nails were 6 or 8 m long and drilled and grouted. Four of them in a vertical section (marked M3 in Figure 3.7a) were instrumented with resistance strain gauges welded to the reinforcement. Horizontal displacements with depth at three distances into the soil mass at section M3 are shown in Figure 3.7c. The profiles are similar to those observed in the Davis study.

Measurements along the uppermost of the instrumented nails are given in Figure 3.8a, which show the progressive development of force in the nail with excavation down to a certain depth below it, beyond which there is little increase. Similar observations were noted in the Clouterre experimental result shown in Figure 1.3.

The distributions of force along all four nails after the final excavation stage are shown in Figure 3.8b. The maximum values in the upper two instrumented nails are at about the same
distance along their length. At lower levels the maximum force is nearer the face. The
distributions seen here are different from some of those observed in the nails of the experimental
walls in that the forces generally decrease towards the facing, indicating that there is no rigid
connection there. The nails are therefore providing restraint and increasing stability as a result
of the interaction taking place between them and the soil with its stress relief, rather than
operating as part of a reinforced block.

The monitoring undertaken provides valuable information on the long-term behaviour
of nailed-soil excavations. The variation of nail force with time and the effects of frost on them
are shown in Figure 3.8c and d. The forces remain essentially constant with time. Greater
tensile forces were induced by the frost but these reverted to previous values after it had thawed.

**Case study 2**

Thompson and Miller (1990) describe the construction and monitoring of a nailed-soil
excavation 16.8 m deep in fill underlain by sand and gravel and very dense lacustrine sands and
sилts in Seattle, Washington. The nails were about 10 m long and were drilled and grouted. A
section through the wall, with the nail arrangement is shown in Figure 3.9a. An inclinometer
tube was installed behind the facing position prior to the excavation. The horizontal
displacements during excavation steps are shown in Figure 3.9b, these are similar in shape and
magnitude, given the difference in soil types to those observed in case 1. Five of the nails in
one of the sections were instrumented with vibrating wire strain gauges along their lengths, again
these were welded to the reinforcing steel. Their positions and that of the inclinometer tube are
shown in Figure 3.9c.

The results from the sixth nail down during excavation and in the long-term are shown
in Figure 3.10a and b. The results are given in microstrain, the development of strain and hence
force with excavation can be seen as was noted in case 1. Miller and Thompson emphasise the
difficulties in converting the output from strain gauges into force. They discuss the differences
in stiffness and relative area of the steel and grout materials and also the possibility of the grout
cracking. With these considerations they propose two methods of determining the force. In one
a composite stiffness is adopted and account is taken of the creep potential of the concrete with
time, this is referred to as the ‘indirect method’. The second approach is to assume that jumps
in the strain readings relate to cracks forming in the grout and that thereafter forces are fully
transmitted to the reinforcing steel. It is assumed that the load either side of the jump remains
constant so that the stiffness of the uncracked composite section can be back-calculated, this is
referred to as the ‘direct method’.

The maximum nail loads for each level at the end of excavation using these two
approaches are shown in Figure 3.10c. There is a considerable difference between them. The
authors conclude that forces determined from the direct method are more realistic as those from the indirect method are considerably in excess of those required for active conditions which are thought to exist. The distribution of force along each of the bars is shown in Figure 3.10d. Superimposed on it are those derived from reinforced earthfill and the Davis method (see Section 2.3.3). The maximum force points, the position of which are unaffected by the conversion method used, fall between the two theoretical distributions. In some instances there is another peak in the distribution along the bars, close to the facing. These are ascribed to bending moments from the facing hanging on the nails.

Case study 3

The third case concerns a nailed-soil excavation in Japan within predominantly decomposed granite underlain by slightly weathered granite, described by Mino et al. (1988). The excavation was 11.6 m deep and the nails were grouted steel bars of 3 to 7 m length, two of which were instrumented with strain gauges.

This case study is included because the observed behaviour is different to that of the previous two cases. The displacements observed imply lateral rigid body movements of the upper strata of clayey soil and the decomposed granite. Mention is made of cracking but no details are given of where it occurred, presumably cracks would have formed parallel to the crest in conjunction with the displacements.

The results from the strain gauges indicate that the nails were in compression within 2 to 3 m from the face, with the remaining 4 to 5 m in tension. During the month after excavation there are several jumps in the readings, mostly in the compressive sense. These are possibly related to cracking of the grout as discussed by Miller and Thompson (1990).

There is insufficient information to define clearly the mechanism that is operating. The case has been included as it indicates that the mechanisms expected with soil nailing do not necessarily apply. The authors conclude that the observed behaviour is more akin to that of a concrete retaining wall. This is perhaps due to the soil type which probably still has a significant degree of bonding.

3.2.4 Conclusions from the large scale case studies

The studies discussed above provide valuable information about the behaviour of nailed-soil excavations. In these cases the effects of scale have been eliminated. The most useful information comes from the monitoring carried out of the case studies 1 and 2. The works involved in these cases were full-scale, with many excavation levels and observations were made at serviceability state over a significant long-term period.

The experimental studies, although full-scale, were of a reduced size compared to the case studies. Also emphasis was placed on failing the walls as well as observing their behaviour.
during construction. The purpose of the study described in this thesis is to investigate the development of interface shear stresses during excavation, *i.e.* at serviceability state, rather than at conditions of failure. For this reason attention is focused on the case studies when deciding on the experimental boundary conditions to impose.

### 3.3 NUMERICAL MODELLING

Numerical analysis (*i.e.* using FEM) is covered under three headings in this section: the modelling of conditions at the nail-soil interface specifically, parametric studies, and the modelling of experimental models and full-scale nailed-soil excavations in general. Assessing the ability of any particular finite element program to predict or simulate the conditions under examination is outside the scope of this research. The purpose of this brief review is to outline the approaches that have been adopted.

#### 3.3.1 Modelling conditions at the interface

In numerical modelling the behaviour of a nailed-soil excavation, one of the fundamental requirements is the representative modelling of the conditions at the grout-soil interface. Several methods of achieving this have been proposed, some of which are borrowed from research into interface conditions with piles and reinforced earthfill.

The most basic interaction is achieved by assuming a fully frictional boundary, tying the nodes of the soil elements with those of the reinforcing. Alternatively interface elements (sometimes referred to as joint elements) are used. These take different forms ranging from thin elements that control conditions by means of displacement criteria imposed between relative pairs of nodes either side of the long edges (*i.e.* specifying degrees of freedom) to assigning the element a behaviour governed by a constitutive model.

Frank *et al.* (1982) propose a type of interface element with two nodes allowing slip with friction, 'unsticking' and 'resticking' (a form of stick-slip behaviour). The different criteria are expressed as functions of contact [normal] forces and relative displacements between the materials.

Shafiee (1986) and Juran *et al.* (1985) adopt either the fully frictional approach or an element assigned with an 'orientated [directional] plasticity' criterion is used. The criterion is implemented in very thin elements whose behaviour is fully frictional up to the point when the ultimate shear stress is reached, thereafter tangential displacements are produced under a constant ultimate shear stress.

Chaoui (1992) summarises several approaches to numerically modelling the interface behaviour of soil and reinforcing members. In her study which involved 2- and 3-dimensional modelling of nailed-soil slopes and excavations, the interface elements were assigned a 'friction-
unstick’ (stick-slip) behaviour. The elements were given ‘fictional’ characteristics, with tensile and frictional criteria, the latter being considered as a rigid perfectly plastic non-associative material.

Kodaka et al. (1995) simulate the results from small-scale model tests using the FEM with interface elements that obey a ‘no-length change’ criterion. A linear constraint condition, where the length between two soil nodes along the reinforcement remains constant, is imposed on the velocity field in the soil mass at limit state. The method of analysis allows the axial force in the reinforcement and the velocity field of the soil mass to be determined at a limiting equilibrium state.

Unterreiner et al. (1997) model conditions at the interface in an analysis of one of the full-scale Clouterre experiments using a linear-elastic perfectly-plastic behaviour. The elastic region corresponded to that of the initial mobilisation of lateral friction and the perfectly plastic to that of unit skin friction, both of which were based on the results of in situ pull-out tests.

The subject of modelling conditions at the interface is discussed in relation to the numerical modelling of experimental conditions for the research discussed in this thesis in Section 4.4.

3.3.2 Parametric studies

Parametric studies are generally performed to establish which variables have the greatest influence on the boundary value problem under consideration. In some respects they are the numerical equivalent of element tests performed in the laboratory.

Shafiee (1986) performed numerical parametric studies to investigate the influence of the phases of construction, the rigidity of the inclusions and their inclination on the mobilised stresses on them as well as on the displacement of the wall of the excavation. His findings are summarised by Juran et al. (1985). Analyses were made adopting a fully frictional interface behaviour as well as that described in the previous section.

The investigations into the construction processes led to similar conclusions that have been discussed previously, e.g. that two zones are formed and that the maximum force in the nails is not at the facing connection. The two methods of modelling conditions at the interface produced an appreciable effect on the results, showing the importance of correctly modelling this area. The inclination of the nails was found to affect the shape of the plane of maximum tensile force in them, a smaller active zone resulted when they were horizontal, producing a near vertical line behind the upper part of the wall. The displacements of the facing also increase with sub-horizontal inclination. The results from the analyses indicated that the bending stiffness had hardly any effect on the behaviour of the structure under working loads. The results from this parametric study were compared to those from a reduced scale model test and found to be
in good agreement.

Chaoui (1992) investigated the effects of analysing conditions using 2 and 3-dimensional analyses. The main part of her work was concentrated on analysing 'piled slopes' (i.e. dowelling) but analyses were also performed to model pull-out tests. Both inextensible and extensible reinforcing inclusions were considered, the latter being more relevant to reinforced earthfill. The interface was modelled using a 'stick-slip' behaviour as discussed above.

The comparison for the inextensible inclusions, relevant to soil nails, indicated that the shape of the reinforcement has a significant effect on the results. In the 2-dimensional case, where the nail is modelled as an equivalent plate, displacements of soil and nail, and the force in the nail are under-estimated. Also quite different relative displacements between the soil and the reinforcement take place. In the 2-dimensional case they are non-negligible while the 3-dimensional analysis indicates that the soil and the nail act as one in the zone of interaction except at the rear of the nail, the relative nail-soil displacements are therefore zero. This effect is evident from the zone of maximum soil displacements which forms as a cylindrical bulb around the nail which moves towards the rear of the 'plate' as the load in it increases. In the 3-dimensional case this zone is in the form of a thin layer along the length of the nail. It would have been interesting to have compared the results from these two approaches with those from an axi-symmetric analysis.

A parametric study has been performed by Ehrlich et al. (1996) to investigate the most important factors influencing a nailed-soil excavation. A 10-m deep by 15-m wide excavation was modelled using a plane-strain analysis with equivalent plates representing the nails. No information is given about how conditions at the interface are modelled, it might therefore be assumed that the connection was taken to be fully frictional. The authors have paid particular attention to the effects of bending stiffness and conclude that it does help control soil yielding. Nails with high stiffness values and orientated close to the horizontal provide the greatest degree of control. However, the inclination and bending stiffness of the nails influence the internal forces in the nails, often to the detriment of the axial tensile forces that can be developed.

The conclusions drawn from parametric studies should be viewed in close conjunction with the methods of analysis and the assumptions made with regard to the boundary conditions. Areas of particular concern are the modelling of the soil and reinforcing elements, their interface, and the connection to the facing.

3.3.3 Comparisons between numerical analyses and observed behaviour

Conditions observed either in experimental models or full-scale works are often modelled numerically to provide further information on overall conditions and at the same time to validate the method of analysis and assumptions being used. Often only brief details are given of the
way in which the analysis was performed, and so it is difficult to make meaningful assessments of the results. For this reason this section only includes some references to provide examples of studies where numerical analyses have been used to model experimental tests or full-scale works.

Juran et al. (1983) analysed the conditions within a direct shearbox used to investigate the influence of the bending stiffness of the nails. Boundary conditions were chosen to simulate certain features observed in the experiments, e.g. the interface was modelled as being frictionless and the contacts with the sides of the box as being fully frictional. Plane-strain conditions were adopted with the nails modelled as equivalent plates. The results from the analyses were in good agreement with those from the experiments and provided useful insight into the mechanisms taking place within the apparatus.

The analyses carried out by Kodaka et al. (1995) were mentioned in Section 3.3.1 with regard to modelling the interface. Linear elastic and rigid-plastic numerical simulations were performed. Good agreement between the experimental and numerical results was achieved. The soil mass was shown to have an improved response in resisting applied external loads with the presence of the nails, and that the efficiency of the nails increased when facing panels were provided.

Two of the full-scale tests described in Section 3.2.2 were subsequently modelled using numerical analyses. Shen et al. (1981b) modelled conditions relating to the 10 m excavation at the landfill site. Benhamaida et al. (1997) and Unterreiner et al. (1997) did likewise for one of the experiments performed as part of the Clouterre project. In both cases good agreement between predicted and measured behaviour was achieved.

3.4 RESEARCH PHILOSOPHY

The three groups of studies described in the previous sections have considered conditions overall, at both small-scale and full-scale using both experimental, observational or numerical techniques. Studies involving element tests have concentrated on pull-out type behaviour or the influence of the bending resistance of soil nails.

The behaviour of the ground and reinforcing elements of a nailed-soil excavation have been discussed in Chapter 1 and with reference to previous studies in this chapter. The aspect of behaviour that has not been considered in detail so far, is that within the active zone where the nails are restraining outward soil movements resulting from stress relief at the face. The significance of this zone has become clear from the point of view of understanding the mechanism of soil restraint and quantifying it. These aspects are necessary for improved, more rational design methods and also for formulating more representative constitutive interface
models for numerical analyses.

The approach adopted for the research described in this thesis has therefore been to concentrate on conditions in the active zone using experimental techniques. It was decided to adopt the philosophy of element testing so that the soil and nail behaviour could be observed under known stresses and displacements. Conditions around a single nail were to be modelled.

In order to assess conditions at the interface it is necessary to measure forces and stresses acting on the nail. Primarily axial force at more than one point should be measured to allow interface shear stresses to be calculated. Radial stress measurements allow a fuller interpretation of normal stresses developed on the nail. This combined with the shear stress values enables the angle of interface friction to be calculated.

In the initial stages of the research project the intention was to perform element tests in clay and the following discussions are based on this precept. However most of the arguments are equally applicable to the sand tests that were also carried out.

### 3.4.1 Field boundary conditions to be modelled

The conceptual soil displacements and shear stresses developed during a nailed-soil excavation are shown schematically in Figure 1.5, and have been reproduced in Figure 3.11. The nails are drawn horizontal to simplify interpretation.

The relative displacements between the soil and the nail are shown in Figure 3.11b, the distribution is discussed in Section 1.5. The shear stress developed as a consequence of the soil displacements along the nail boundary are shown in Figure 3.11c and the force induced in the nail as a result in d.

The position of the sample has been superimposed around one of the nails such that the sample is in the active zone and the nail passes through its axis. Following from the conceptual distributions shown in Figure 3.11, the boundary conditions to be modelled experimentally can be summarised as follows.

(i) The sample is to lie within the active zone with the nail passing through its axis.
(ii) Axial stress relief is to be imposed.
(iii) Excavation takes place under drained conditions.
(iv) The normal stresses acting on the nail have components of horizontal and vertical stress.
(v) There is a plane/point along the nail where there are no relative soil to nail displacements and where axial force in the nail is a maximum.
(vi) There is a smooth transition from negative to positive shear stress.

### 3.4.2 Experimental boundary conditions

The experimental boundary conditions imposed to model field conditions in the active
zone are shown schematically in Figure 3.12 and summarised as follows.

(i) The sample is cylindrical and tested under triaxial conditions.

(ii) Axial stress relief is achieved by unloading the ram pressure under constant cell pressure.

(iii) Sample unloading takes place under drained conditions.

(iv) It is assumed that a uniform normal stress acts around the sample, and hence the nail, because of the nature of cylindrical triaxial conditions.

(v) The nail is fixed rigidly to the base platen to model no relative soil to nail displacement at this point.

(vi) Lubricated end platens were used to minimise friction and the development of shear stresses at the sample ends.

(vii) The nail at the top of the sample passes freely through the upper platen, thus axial force and shear stress at this point are zero.

Radial drainage via side drains was adopted because of the use of lubricated ends. This can affect the uniformity of consolidation in the sample if shearing rates are too fast as discussed by Atkinson et al. (1985). In order to avoid this effect and the development of excess pore water pressures very slow shearing rates were adopted. A mid-height pore-pressure probe was also incorporated in the sample to confirm the latter effect.

In order to eliminate the effects of installation, it was decided to install the nail in a 'wished-in-place' condition. The procedure for achieving this is described in Section 5.3.1, along with the other aspects of the experimental procedures.

The stress path to be followed was one of drained axial extension under constant radial effective stress. In terms of $p' - q$ space, this relates to a path with a slope of $\delta q / \delta p' = 3$ with both $p'$ and $q$ decreasing. The stress paths for an element of soil adjacent to an excavation, and the equivalent path followed under triaxial conditions, are shown in Figure 3.13. The conditions of rising or falling ground-water can be simulated by decreasing or increasing the cell pressure under constant axial stress. The associated stress paths would have a slope of 1.5 with increasing $q$ and decreasing $p'$ for the rising ground-water case and *vice versa* for a drop in ground-water level.

### 3.4.3 Aims of the research project

The primary aim of the research project is to provide information on how shear stresses are developed along the nail interface and the displacements required to mobilise them. By incorporating instrumentation along the length of the nail, the distribution of shear stresses can be assessed and compared with those produced in pull-out tests, which are generally used for determining the frictional properties to be used in design.
As part of the research project, numerical analyses were performed to assess sample conditions within the experimental apparatus. The main aim with this aspect of the work was to optimise the dimensions of the nail and sample to aid interpretation of the results. The approach taken is discussed in Chapter 4. Comparisons can then be made between the predicted interface behaviour and that indicated from the experimental results.

Although the boundary conditions discussed and the numerical analyses performed relate directly to clays, sand tests were also performed to widen the scope of behaviour with respect to grain size. The experimental conditions for these tests are discussed in Chapter 6.

The final aim of the project is to extrapolate the results so that they can be interpreted in relation to full-scale field conditions. This subject is dealt with in Chapter 10.
CHAPTER 4

Numerical analyses of experimental conditions

In the previous chapter the boundary conditions and ground behaviour of a full-scale nailed-soil excavation were discussed and a conceptual way of simulating them at small scale in a controlled laboratory environment discussed. The mode of testing under triaxial conditions as well as the way of implementing the boundary conditions were also described.

Prior to detailed design of laboratory equipment it was necessary to establish the nail and sample dimensions to satisfy two criteria. It was required that:

a.) stress conditions of reasonable magnitude and uniformity would be achieved within the sample, especially at the nail-soil interface and;

b.) sample failure would be induced along the nail-soil boundary rather than across the sample.

The boundary conditions of the problem under investigation are such that there will always be a variation in shear stress along the nail. This is the case, even after large displacements, because of the connection of the nail to the base platen, thus restraining development of shear stress there. The smaller the nail diameter, for a fixed sample size, the greater the likelihood of achieving uniform stress conditions over most of its length at a given displacement. However, the smaller the nail the less its contribution to restraining sample deformations, and the less shear strength mobilised on its surface.

On the other hand if the nail diameter is increased too much, failure will occur through the sample, probably at one of its ends, with minimum mobilisation of the full strength along the nail interface. The aim of the study is to investigate conditions along the boundary of the nail as stress relief of the soil surrounding it takes place. If the nail diameter were too great, this would be precluded, only limited information could be gained about the progressive development of stress and the restraint of sample deformations.

A compromise between these two cases is therefore required, to optimise sample and nail dimensions so as to maximise the development of interface shear stress along the length of the nail while not over-reinforcing the sample.

Most of this chapter describes the initial numerical analyses and the results obtained from them aimed at optimising the nail dimensions. At a later stage in the research, during the development of instrumentation on the model soil nail, further detailed analyses were carried out to investigate conditions in the vicinity of the radial stress measuring devices. The results from these ‘detnail’ analyses are also described. Once testing was underway, it was decided to repeat an analysis similar to that of the parametric study, using the nail and sample dimensions adopted,
but with much smaller displacement increments. The purpose of this analysis was to provide information on conditions throughout the sample to supplement measurements made during the tests which were restricted to the sample boundaries and the nail interface.

4.1 FINITE ELEMENT ANALYSIS

The two criteria given above were checked by performing numerical analyses using the finite element method. This powerful method of analysis is used to model many geotechnical situations in practice. Its use is becoming increasingly widespread and a description of the underlying principles of the technique are not included here.

The problem to be considered is usually first simplified so that only a representative plane within the soil need be analysed (e.g. a plane of symmetry in plane strain conditions, an axi-symmetric slice in an axi-symmetric case). This plane is then divided into a series of discrete finite elements. Element sizes are chosen depending on the complexity of behaviour expected at different points within the representative plane and the final grid is referred to as the mesh. Initial stresses are assigned to the elements in the plane and on the boundaries to simulate those existing in the case being modelled. The appropriate choice of constitutive model is a fundamental requirement for achieving representative results.

Conditions at the boundaries of the problem being considered are usually controlled by applying increments of stress or displacement to the appropriate parts of the finite element mesh, depending on the nature of the problem.

4.1.1 Imperial College Finite Element Program (ICFEP)

The analyses for this study were made using the versatile and highly sophisticated Imperial College Finite Element Program (ICFEP), developed and continually updated by Prof. David Potts and his co-researchers. ICFEP for the majority of the work was run using a Prime750C computer; the later work was run using a Sun work station. A full description of the many features incorporated into ICFEP is outside the scope of this work, (see Day, 1990; and Ganendra, 1995). However, some aspects relevant to this study are now given.

The program has the facility to perform analyses using various constitutive laws ranging from the most basic linear elastic to the highly complex Lade's double hardening model. It is also possible to run consolidation and large strain analyses, although these features were not available at the time when the analyses for this study were made. There are various element types available such as solid, beam, joint and membrane elements as explained in later sections. The solid elements used were always eight-noded, iso-parametric elements of the Serendipity family. Meshes for the analyses were automatically generated by the program, with a renumbering routine implemented to optimise available computer space.

During the runs the stiffness matrices were updated during each increment and a stress
point algorithm was also implemented. The position of the current stress was checked for all points so that the stress state did not lie outside the current yield surface. If this was found to occur, it was corrected using a projecting back routine. Concise details of these features (for the program version used in these runs) incorporated in ICFEP are given by Day (1990).

Conditions in the experimental apparatus were modelled using displacement control with step size increasing with displacement. Generally solutions converged within twenty iterations for each increment and for the series of steps used, stresses were fully mobilised along the nail-soil boundary within twenty to thirty increments. Data sets were usually saved at every second increment initially and then at every fourth. Thus, the program could be restarted at any one of these save points and these were also the stages at which the data could be processed after running the analysis.

4.1.2 Parametric studies

The purpose of the parametric studies was to establish sample and nail dimensions such that the two criteria described in Section 4.1 would be satisfied. The same triaxial apparatus used by Martins (1983) was to be utilised (see next chapter, Section 5.2). As he had performed model pile tests, the possibility of using the same sample and pile dimensions adopted by him was considered at the time so that some common equipment components could be reused. The initial analyses were therefore performed using these dimensions. After it was established that the necessary criteria were not satisfied with these dimensions, a series of parametric analyses were carried out. First, the sample dimensions were kept constant and the nail diameter varied and then the sample size was varied, using the smallest practicable nail.

4.1.3 Difficulties in achieving a realistic simulation

The main uncertainty with the analyses was modelling conditions at the nail-soil interface in a representative way. Various approaches were used to investigate conditions at the upper and lower limits of possible behaviour, i.e. by using fully frictional and frictionless boundaries, one of the main objectives of the research being to establish these stress conditions at the nail-soil boundary.

4.2 NUMERICAL MODELS USED

Three models were used to model conditions within the clay samples. The behaviour of the solid elements representing the clay sample were always modelled as an elasto-plastic continuum using the Modified Cam Clay constitutive law. During the detailed runs discussed later the steel and grout elements were assigned a simple linear elastic behaviour, for which the main input parameters were Young's modulus and Poisson's ratio of each respective material.

Some of the runs were performed using joint elements to model interface conditions at the nail-soil boundary. These joint elements which have no thickness were governed by the
Mohr-Coulomb failure criterion.

4.2.1 Modified Cam Clay

The Cam Clay constitutive model was originally conceived during the early years of the development of critical state soil mechanics at Cambridge University. The model was later refined by changing the shape of the yield surface to give a rounded shape rather than a corner when under isotropic stress. It then became known as Modified Cam Clay (MCC) (Roscoe and Burland, 1968).

The model has four principal characteristics: elastic properties; a yield surface; a plastic potential and a hardening rule. In theory, the model as such is complete, as it is able to simulate all aspects of soil behaviour. There are several factors to be considered regarding the application of the model to this study.

The original model was formulated from the results of a series of triaxial tests performed in compression. Most of the modelling done in this study relates to testing in extension. In the literature it is usually implied that the model can be used in this region of stress space, simply by adopting negative values of deviator stress and considering a mirror image about the isotropic axis.

The tests on which the model is based were performed on samples of reconstituted, isotropically compressed Spestone kaolin. Although such samples might not necessarily follow natural soil stress histories, they closely simulate the stress history followed by the samples in this study, which were of Speswhite kaolin (see Section 9.2).

Although the shape of the yield loci assumed in this model may not closely resemble those observed for natural clays, the advantage is that their form can be described using just one shape with two parameters to define it ($M$ and $p_0$). This reduces the number of input parameters (which are based on experimental test results) and its simplicity makes for faster and more economic computer running. Moreover, it is often noted that for many practical applications the differences between the simplistic Modified Cam Clay model and other more realistic models are inconsequential anyway, (see Muir Wood 1991, Chapters 10 and 11).

As the samples in this study were all normally-consolidated, the results from the numerical analyses should match those observed experimentally reasonably well. This is because results from samples on the ‘wet side’ of critical state tend to be much better conditioned than those on the ‘dry side’.

Coincidence of the yield locus and plastic potential (i.e. normality or associated conditions) was always assumed in the analyses. Also the yield function was assumed to be a surface of revolution about the space diagonal (i.e. as in the extended Von Mises criterion) assuming a constant value of $M$ (the stress ratio, $q/p'$, at critical state).
4.2.2 Mohr-Coulomb

The Mohr-Coulomb strength criterion was used to model soil behaviour at the interface between the soil and the nail boundary using interface elements which are discussed in Section 4.4.2.

This criterion gives information on strains and deformation in the case of the joint elements and provides a limiting strength cut-off. The input parameters required are the angle of internal shearing resistance, $\phi'$, and the cohesion intercept, $c'$ and normal and shear stiffnesses.

4.2.3 Soil properties used

Seven basic parameters are required for the MCC constitutive law. The values adopted were based on those given by Martins for his Model A study, which assumes a circular yield surface and plastic potential in the deviatoric plane. The values used are summarised below and were used for the parametric studies and for the soil properties in the detailed runs performed later.

The primary soil parameters were:

- $M = 0.9$ = slope of the critical state line (in $p' - q$ space);
- $\lambda = 0.25$ = slope of the virgin consolidation line;
- $\kappa = 0.05$ = slope of the swelling line;
- $\nu_1 = 3.65$ = specific volume on the virgin consolidation line for $p' = 1$;
- $G = 18000\text{kPa}$ = elastic shear modulus (determined from the load-displacement curves from one of Martins pile tests).

Parameters defining the shape of the yield function in the deviatoric plane were:

- $Y = 0.0$ = first parameter;
- $Z = 1.0$ = second parameter.

4.2.4 Soil properties and model used in a later numerical study

It was decided at a later stage of the research to perform further numerical analyses to examine sample behaviour at much smaller displacements than those used in the parametric studies. This followed a review of the clay test results described in Chapter 9. Certain behaviours were observed in the nail and overall in the sample and further numerical analysis were performed to provide insight into the mechanisms occurring. The opportunity was taken to use more refined parameters and also the runs were made using both approaches for MCC where (i) a constant $M$ value is used (as in the parametric studies), and (ii) a constant angle of internal shearing resistance, $\phi'$, is used. The intention was to assess the influence of these two approaches. The values used were the same as those above except that in the first approach $M = 0.65$ was adopted. This is more appropriate to kaolin behaviour in extension. In the second approach $\phi' = 26^\circ$ was adopted (again a typical value for kaolin in extension).
The results from these analyses are discussed later in Section 4.7.

4.3 MODELLING EXPERIMENTAL CONDITIONS

The physical boundary conditions to be imposed in the experimental work and their numerical implementation are discussed. The mesh used for modelling the sample behaviour is then described. Within the mesh there were certain areas that needed special consideration; these and the means of modelling the nail-soil interface behaviour are then discussed.

4.3.1 Experimental boundary conditions

Five main criteria were identified from the boundary conditions discussed in the previous chapter.

(i) Stress release should be controlled from one end of the sample which should be free to move. In the triaxial system this was achieved by unloading the ram pressure. The sample end attached to the ram system would be the movable one, the other end would be fixed rigidly.

(ii) Drainage during shearing should be through one end of the sample if stress relief of the face of the excavation is being modelled by dissipation of pore water pressure. Drainage in the test set-up was principally via radial drains because of boundary condition (v) below. Drainage paths did not play a significant role in the numerical analyses as will be seen in the next section.

(iii) The nail should be fixed rigidly to one of the sample platens. It was decided for practical reasons to fix it to the base platen.

(iv) Initial normal stresses around the nail should be equal, which assumes isotropic conditions in the field with negligible difference in vertical stress above and below the nail. This was modelled using a constant all-round cell pressure.

(v) Sample ends should be smooth (frictionless) and rigid. Lubricated ends were implemented to achieve this although these do prevent drainage from sample ends. The platens in the apparatus used were rigid.

The axi-symmetry of the conditions within the triaxial sample make it possible to greatly simplify the numerical analyses.

4.3.2 Numerical imposition of boundary conditions and experimental operation

In modelling the experimental set-up, the axi-symmetrical condition was utilised as shown schematically in Figure 4.1. It was therefore only necessary to consider a thin radial slice extending from the sample (and nail) axis to the sample periphery. Membrane effects were ignored for these parametric studies as the regions within the sample of greatest interest were those in the vicinity of the nail.

Referring again to Figure 4.1, the following numerical boundaries represent components or boundaries within the experimental set-up:

AG is the axis of the sample and nail;
-BC and DF are the lower and upper sample platens respectively;
-CD is the outer periphery of the sample which would be adjacent to the membrane, and;
-BF is the interface between the nail and the soil.

The boundary conditions to be imposed in the experimental set-up were implemented as follows. 

*Condition (i)* was achieved by fixing boundary DE (i.e. restraining vertical movement along its length) while boundary CBF was moved down. Thus the axial unloading of the sample was performed using displacement control. During the initial unloading, displacement increments were kept small (usually to 0.25 or 0.50 mm) for the transitions from elastic to elasto-plastic behaviour, after which larger displacements could be tolerated (usually these are limited to 1 mm or 2 mm). These were usually continued up to 20 mm displacement.

Moving the whole boundary CBF as one, implies that the nail whose outer boundary was represented by BF was rigidly fixed to the base platen BC. Thus boundary condition (iii) was already imposed by this action. No relative vertical movement between any of the nodes along boundary CBF implies that it is completely rigid in this sense. Note that in order to allow movement between the nail at F and the upper platen (strictly speaking represented by DF) a small gap, EF, was left to allow nail movement: this is discussed further in Section 4.3.4.

*Condition (ii).* During the runs it was not possible to specify any drainage conditions: the analysis was either performed either fully drained or undrained. In the undrained case the build-up of pore pressures corresponded to those generated during the respective increment. 

*Condition (iii)* was achieved as a consequence of the displacement control used where no relative movement was allowed between any nodes along boundary CBF (see paragraph about (i)).

*Condition (iv).* A constant all-round stress of 250 kPa was applied on boundary BCDF with zero pore water pressure. The initial stresses imposed on the sample at the start of the analysis were maintained at its boundary throughout the analysis. The back pressure used in the experiment was not modelled numerically.

*Condition (v).* Modelling of the lubricated ends was achieved by not implementing any horizontal displacement control along the platen boundaries BC and DE.

Other test conditions implemented were that the nail was modelled as being completely rigid by allowing no horizontal movement of nodes along boundary BF. This assumption is judged to be acceptable after quantifying movements of the radial stress measuring devices in Chapter 7. Moreover, in the event clay tests were not performed with these devices. Also the results from the detailed analyses indicate that the stresses within the sample were not adversely affected by it. The radial surface of the sample was allowed complete freedom of movement, again by default by not imposing any displacement control along its boundary (CD). In practice this is realistic except for constraints from the sample membrane and radial filter strip drains,
which are considered negligible for the purpose of this study.

4.3.3 Finite element mesh used

In choosing the layout of a mesh two main opposing factors need to be considered and balanced. Firstly there need to be enough elements such that the behaviour of the soil is being modelled in sufficient detail. This is particularly so in regions of the soil which might be subjected to large distortions. Such areas are generally of greatest interest because failure initiates from them and propagates through the soil. If the elements are too large then considerable stress variations are created across them, with the possible result of the numerical model not being able to accurately predict the resulting stress/strain state at the end of the increment. Ideally, the more elements used the greater the likelihood of obtaining more accurate results, especially in areas of large distortion. However, as the number of elements increases so also do the computer resources required to obtain a solution.

The mesh used for the parametric studies is shown in Figure 4.2a. The compromise concerning the number of elements can be seen with the smallest elements adjacent to the nail and platen boundaries. Element sizes then increase in size towards the periphery of the sample. The mesh is a straightforward rectangular grid; care was taken to avoid assigning any element dimensions with an aspect ratio greater than 1:10.

In performing the parametric studies when the nail diameter was changed, the mesh was always regenerated to the correct size by adjusting the width of the outer larger elements so that the fine elements and details adjacent to the nail remained the same.

4.3.4 Problem areas within the mesh

Two areas within the mesh needed special attention. The elements adjacent to the nail were the most problematic because they were subjected to the greatest distortion under the operating conditions. At the same time this area of the sample was of greatest interest and so good numerical convergence was sought. These elements are discussed in the next section.

The other area that proved problematic was that in the vicinity of the 'gap' mentioned in Section 4.3.2. It was necessary to leave a small gap between the nail and the upper platen to allow movement between these components, (in Figure 4.1, EF describes the gap). The gap specified was 0.5 mm, i.e. the width of the smallest element adjacent to the nail.

Problems arose because the soil adjacent to the gap was unsupported by either platen or nail. This area also suffered the greatest distortion because the nail is being pushed past the fixed platen boundary; while at the other end of the sample the nail and platen are fixed together. The result was that the soil in this corner failed almost immediately. Sample distortions from the nail displacement initiated in this area and ultimately sample failure occurred across the top of the sample with very little strength being mobilised along the nail boundary.

In order to restrain these effects nodes across the gap were artificially displaced in
accordance with the nail displacement. The node adjacent to the nail was displaced an equal amount to that of the nail, whilst that at the other end of the gap was not displaced at all. The nodes between these two were displaced proportionally between these extremes.

These two actions restrained the adverse effects from the gap satisfactorily, allowing shear stresses to develop along the nail boundary. Failure of the soil did still occur first in this region but it did not propagate across the sample in the same way as before.

An alternative approach might have been to replace some of the elements in that corner with a small block of elastic material. This method was not implemented as it was decided that it would over-complicate the situation. It was also suspected that the soil elements adjacent to the elastic block would still be subjected to large distortions, but that they would be induced further down the nail thus further reducing the chance of inducing and observing a steady build-up of stress along the length of the nail.

4.3.5 Additional runs performed for other stress paths relevant to soil nailing

As discussed in Chapter 3, the main stress path to be followed in triaxial stress space to simulate stress changes in the ground from a nailed-soil excavation, was drained axial extension as described in Section 3.4.2. This stress path is shown in Figure 4.3(i).

Although this was to be the principal path studied it was decided at this stage to perform numerical analyses for three other stress paths to check stress conditions within the sample (i.e. to make sure that nail and sample dimensions were appropriate for all likely cases). The three additional stress paths considered are shown in Figures 4.3(ii) to (iv) and described as follows. Path (ii). Drained axial compression under constant radial total stress, i.e. a path of the same slope as (i) but moving in the opposite sense. This path was considered purely from a hypothetical point of view.

Path (iii). Drained radial extension under constant axial total stress. This path simulates the case of a rise in ground water or the removal of overburden with the nail under working conditions. Path (iv). Drained radial compression under constant axial total stress. This path is the same as for (iii) but in the opposite sense and thus simulates a drop in groundwater or increase in overburden while the nail is under working conditions.

4.4 NUMERICAL MODELLING OF INTERFACE BEHAVIOUR

Conditions at the nail-soil interface were modelled using two approaches which are now discussed. The first method provides an opportunity for examining soil behaviour at the interface under a limiting condition. The other method allows very specific characteristics to be assigned to the interface.

4.4.1 Modelling using a fully frictional interface

This idealised approach was used for all the initial parametric runs. A fully frictional
boundary is achieved by the displacement of all nodes along the edge of the thin elements adjacent to the nail boundary; i.e. the displacement of CBF (but specifically BF) as discussed previously, (refer to Figure 4.1). By moving the boundary as one, the nodes are moved with the distance between any pair of them always fixed.

Performing the runs under this interface condition gives the worst case for possible stress non-uniformities along the boundary. In practice some reduced friction would be expected from phenomena such as interface slip, strain softening or the formation of fabric features such as Riedel shears. It is unlikely that a friction angle greater than that of the soil itself can be sustained. Therefore, if uniform stresses are indicated from an analysis with a fully frictional boundary, they should be expected for other interface conditions.

Performing analyses with a frictionless boundary should produce uniform stresses within the sample, particularly as the end platens are modelled as being frictionless as well.

4.4.2 Modelling using interface (joint) elements

In recent years interface elements have been developed to model conditions at a junction between two materials (e.g. soil-pile; soil-concrete). These elements have no physical thickness but have two nodes across their width, they lie alongside the solid elements bordering on the interface boundary; the boundary may define a change of material (e.g. soil-steel) or it may simply be an imposed rigid boundary. The idea behind these elements is that they are assigned a constitutive behaviour, usually based on strength, different to that of the soil to account for the characteristics of the interface (e.g. by using a residual angle of friction along the side of the pile during driving). The joint elements are always of the same length dimension as the solid elements they border so that each pair of nodes across the joint element lines up with the adjacent node in the solid element.

It was decided to perform some analyses using joint elements to compare the results with those from using the fully frictional boundary. The interface elements were governed by the Mohr-Coulomb failure criterion (see Section 4.2) using initially a friction angle of 17.5°. This was chosen as being midway between a critical state angle ($\phi_{cs} = 23^\circ$) and a residual angle ($\phi_{r} = 12^\circ$).

In performing these analyses the edge of the interface elements is displaced along with the lower platen boundary. The stresses in the solid elements are then analysed.

4.5 RESULTS FROM THE PARAMETRIC STUDIES USING SIMPLIFIED CONDITIONS

The parametric studies were performed in two stages. Initially the nail diameter was varied keeping the sample dimensions constant, adopting the sample dimensions used by Martins (1983), and then both nail and sample dimensions were varied. The initial stresses in all the
following runs were isotropic and of magnitude 250 kPa. Fully drained analyses were performed throughout.

4.5.1 Varying nail diameter with constant sample dimensions

The summarised plots from these runs are presented in Figure 4.4. The sample dimensions were held constant with a diameter of 100 mm and height of 150 mm. The nail size was varied in steps as shown on plots from the 15 mm diameter used by Martins to an infinitesimal size (i.e. zero diameter where the nail is represented by a line, the axis of the sample, which was displaced with the platen boundary). The plots are from the analyses with the fully frictional boundary (Section 4.4.1) and represent conditions after 15 mm of axial displacement.

The stress quantities shown on the four plots are those acting in the solid soil elements given by the Gauss points closest to the nail-soil interface. The stresses, as indicated in the figure, are a) effective radial stress, b) effective axial stress, c) interface shear stress and d) stress level, $S$. This last parameter is a measure of the degree of deviatoric stress mobilised in the soil.

$$S = \frac{J}{J_{\text{max}}}$$

(4.1)

where $J$ and $J_{\text{max}}$ are the current and maximum values of the second invariant of deviatoric stress tensor respectively, defined by:

$$J^2 = \frac{1}{6} \left[ (\sigma'_1 - \sigma'_2)^2 + (\sigma'_2 - \sigma'_3)^2 + (\sigma'_3 - \sigma'_1)^2 \right]$$

(4.2)

When the deviatoric stress is fully mobilised, i.e. the soil is at its critical state, then $S = 1$, while under isotropic conditions with no shear stress, $S = 0$.

After 15 mm axial displacement, the radial effective stress along the nail boundary has increased above the initial value of 250 kPa for all nail diameters (see Figure 4.4a). Over the lower half of the nail, towards the fixed end, the values reach a roughly constant value of 300 kPa. Beyond this range, towards the free end with the gap, the $\sigma'_r$ values increase progressively with increasing nail diameters from 2 mm, only the infinitesimal nail shows any decrease in stress. The maximum radial stresses observed range from about 300 kPa to 430 kPa for nail diameters of 2 to 15 mm respectively.

At the free end of the nail the $\sigma'_r$ values drop dramatically. This behaviour can be seen in all the stress plots in Figure 4.4 and is thought to result from the considerable distortion that the soil is undergoing in the vicinity of the ‘gap’ between the nail and the upper platen. The cause of the increase in radial stress observed over the upper half of the nail is not obvious, although it is almost certainly related to the development of shear stress in that region. The more constant values observed lower down the nail might result from rigid-inclusion effects as
The relative nail to soil stiffnesses are the same regardless of the variation in diameter.

The axial effective stresses in the vicinity of the nail are shown in Figure 4.4b. At the lower end, the axial stress generally increases with decreasing nail diameter, from its initial value of 250 kPa. In the upper half of the sample this trend reverses so that the smaller the nail diameter the lower the axial stress, although the axial stress is still greater than that at the start of the shearing.

Figure 4.4c shows the shear stress distributions in the soil adjacent to the nail for the different nail diameters. There is a greater uniformity of stress along the nail boundary as the nail size decreases and is only uniform for the 2 mm diameter nail. At the 15 mm diameter, even after 15 mm displacement there is a significant variation of shear stress along the whole length. The connection of the nail to the base platen boundary condition imposes this behaviour, the fact that with a 2 mm diameter nail shear stresses are uniform, indicates that this size of nail has negligible contribution to the overall strength.

The shear stress distributions are reflected by the stress level plots given in Figure 4.4d. Shear stress has only been fully mobilised in the smallest nail sizes which realistically were completely impracticable for the experimental work. The larger nail sizes induce mechanisms within the sample which tend to cause failure through the sample rather than along the nail boundary.

4.5.2 Varying sample dimensions

After assessing the results from the series of parametric studies for a series of nail diameters within a set sample size, it was decided to vary the sample dimensions. The intention being to try to optimise the nail and sample dimensions such that the mobilisation of stress along the nail interface is maximised, without causing failure through the sample. Initially the nail diameter was kept constant at 15 mm as used by Martins. Results from three of the sample sizes used are shown in Figure 4.5. A maximum diameter of 160 mm was dictated by the space available in the apparatus to be used. The plots showing shear stress and stress level (i.e. c and d) along the nail boundary indicate that increasing the sample diameter caused a greater proportion of the shear stress to be mobilised. Decreasing the sample height to 100 mm made little difference in terms of the degree of shear stress mobilised because a greater proportion of the length was influenced by the fixed-end condition at the base.

Following from the analyses using these sample dimensions, a further analysis was carried out with the increased diameter of 160 mm, height of 150 mm but a reduced nail diameter of 8 mm. More uniform stress conditions were achieved with shear stress fully mobilised (i.e. stress level of unity) over two-thirds of the nail length, as shown in the same Figure 4.5.

No further parametric studies were performed after this stage. There was little scope for
further variation as there was insufficient space in the triaxial cell to test a larger diameter sample and the nail diameter was of a size just sufficient to feasibly instrument.

4.5.3 Verification of stress conditions within sample using joint elements

Having established a set of workable sample and nail dimensions which resulted in a reasonable distribution of stress with a fully frictional nail-soil interface, it was decided to model the behaviour more closely using joint elements along the nail-soil interface.

The joint elements were assigned a Mohr-Coulomb failure criterion with a zero intercept value and an angle of internal friction, $\phi' = \frac{1}{2}(\phi_s' + \phi_f') = 17.5^\circ$. The dimensions used were for sample height of 150 mm, sample and nail diameters of 160 mm and 8 mm respectively. The results are presented in Figure 4.6, which is of the same format as the previous figures relating to the parametric studies but showing the development of stresses with increasing axial displacement. Matching plots for the fully frictional boundary are given in Figure 4.7.

In both of these figures it can be seen that a large percentage of shear stress was mobilised at an early stage (see graphs (c)), i.e. less than 2 mm axial displacement and both indicate negligible increase in shear stress after approximately 10 mm displacement (corresponding to about 6.7 % axial strain). The shear stress distributions are more uniform over most of the sample height for the joint element run compared with the more gradual build-up with the fully frictional case. Shear stresses mobilised for the joint element analysis are considerably lower than those from the fully frictional case. This is a consequence of using a reduced $\phi'$ value compared with the unfactored constant value of $M$ used in the fully frictional run, which implies a varying $\phi'$ value.

The radial stresses shown in graphs (b) increase in both cases by similar magnitudes, although those for the fully frictional case are slightly greater. Axial stresses at the interface decrease initially in both cases (i.e. after 2 mm displacement). The trend continues with the joint element analysis (Figure 4.6a), while with the fully frictional case they then start to increase (Figure 4.7a). The results from the parametric analyses, where the sample dimensions were held constant, showed different distributions of axial stress along the boundary with varying nail diameter (Figure 4.4b). This results from the different degrees of shear stress being mobilised along the length of the nail. As the nail diameter increases, there is a greater likelihood of sample failure at one of the ends rather than along the nail-soil boundary. In other words the failure mechanism is controlled by the diameter of the nail. Although the results from the analyses with the two interface boundary conditions do not indicate different failure mechanisms, the axial stress generated appears to be related to the degree of shear stress mobilised, in the same way as observed from the results from the earlier parametric study.

4.5.4 Consideration of alternative stress paths

Analyses were performed for the three additional stress paths described in Section 4.3.5
and shown in Figure 4.3. The results from these analyses indicated that uniform stress conditions developed along the length of the nail for each of these paths, even using the sample and nail dimensions adopted for the first of the analyses (i.e. those used by Martins, with sample height of 150 mm, diameter of 100 mm and inclusion diameter of 15 mm). This was the case even at low stress levels.

This well conditioned behaviour is thought to be a consequence of the paths’ contractant behaviour in the case of (ii) and (iv), and the increasing deviatoric stress in case (iii). Changing the sample height and nail dimensions to obtain uniform conditions was not necessary, as in case (i). Varying sample dimensions should not adversely affect the uniformity of stress or the mechanism of failure for these additional three paths because of their sense of shearing.

There was insufficient time to investigate these three paths experimentally, they are not therefore referred to or considered further.

4.5.5 Conclusions from the parametric studies

The initial analyses were performed using the sample dimensions adopted by Martins with a fully frictional interface between nail and soil. The results indicate that uniform stress conditions would not be obtained for a sample being sheared in drained axial extension unless the inclusion within it were of 2 mm diameter or less. It is now recognised that the $M$ value used in the MCC constitutive law for modelling the soil was probably not appropriate for this stress path, under these interface conditions. However, using a lower value, more relevant to drained extension, is unlikely to significantly change the results.

It was decided that the minimum nail diameter, that could be practicably instrumented for the experimental work, should be limited to 8 mm. Using this nail size and varying the sample dimensions within the physical constraints of the apparatus, the stress conditions achieved with a sample height of 150 mm and diameter of 160 mm were the best that could be expected under these given boundary conditions. Interface stresses developed progressively along the length of the nail with these dimensions, becoming uniform over about two-thirds of the length after large displacements.

Once these sample and nail dimensions were decided, another further analysis was carried out using a more realistic interface model. Joint (interface) elements were incorporated along the nail boundary, with their behaviour governed by a Mohr-Coulomb failure criterion. This analysis indicated that uniform stress conditions should exist along almost the whole nail length, even at small displacements.

Numerical analyses were also performed to check conditions within a sample subjected to other drained shearing stress paths. These were chosen to represent, for example, conditions where there is a rise or fall in the ground-water level. A uniform distribution of interface stress resulted in all cases, even for the original nail and sample dimensions adopted.
Following the numerical analyses and practical considerations, the nail diameter adopted finally was 9 mm (allowing for an 8 mm central instrumented core, with a 0.5 mm thick annulus of grout). The sample height was kept at 150 mm while the diameter was slightly reduced to 152 mm (6 inches) to be compatible with other components of the testing apparatus that might be required. The design of the apparatus and model nail to these dimensions is discussed in Chapters 5 and 7.

4.6 DETAILED NUMERICAL ANALYSES

After designing and testing the second prototype instrumented model soil nail (described in Appendix 7.2), detailed numerical analyses were performed to check for stress concentrations in the vicinity of the radial stress sensing devices.

The prototype 2 model nail is shown in Figure A7.2.1. The radial stress measuring devices are thin-walled shells of stainless steel with internal strain gauges to measure strains induced from applied radial stress. The concern was that radial stresses would not be fully transmitted to the membrane due to cell-action effects. *i.e.* as the device deformed from the applied stress, there would be a local inward movement of the soil towards the nail, causing local arching and a reduction of the true radial stress that would act if the nail were completely rigid.

The intention was to cast grout around these devices and cover the thin-walled shells with a lubricant so that the grout did not adhere to them. Changes in strain gauge output would then result mainly from radial stresses, and cross-effects from axial stresses induced in the shells would be minimised. For this same reason the shells rested on very narrow supports to minimise friction.

Two analyses were carried out as previously, with the sample sheared in axial extension. In one case the outer surface of the shell was frictionless (lubricated) as intended, while in the other, conditions were modelled where the grout was fully bonded to the shell.

4.6.1 Boundary conditions and mesh employed

The overall boundary conditions used for the detailed runs were essentially the same as those for the parametric studies as shown in Figure 4.1. The main differences were along the nail boundary and within the nail itself. Each component of the instrumented nail was modelled with an appropriate element type and a 0.5 mm annulus of grout was simulated around the nail. The boundary control was the same as for previous analyses except that the nail and fixed end platen were displaced from the opposite end than in the previous analyses. This does not affect the results, but means that the distance axis is reversed in each plot.

The mesh used is shown in Figure 4.2b, with the different materials indicated. It was necessary to provide very small elements for the components of the nail. The adjoining grout
and soil elements at the soil boundary were assigned the same thickness. They were therefore of a much smaller size than in previous analyses. The soil elements were increased in size, away from the nail vicinity, using non-rectangular elements as shown. Element sizes in the material considered to be outside the region of nail influence were considerably enlarged to minimise the number of elements.

All elements were solid except those modelling the thin-walled radial sleeves for which shell elements were used, which have length and an axial and bending rigidity but are of zero thickness. A detailed enlargement of the mesh in the vicinity of one of the radial stress measuring devices being modelled is shown in Figure 4.8. The three types of solid element can be seen along with the shell elements that modelled the thin-walled sleeves. The supports to the thin-walled radial sleeves were simulated by tying the radial displacement degrees of freedom at nodes A and B, i.e. maintaining a constant radial distance between A and B in Figure 4.8. These points correspond to the positions where the sleeves were supported in the real nail (i.e. on the thin shoulder at the end of Component 3, labelled 7, in Figure A7.2.1). Further details concerning the interface conditions imposed are given in Section 4.6.3.

4.6.2 Material properties and models used

The kaolin clay was modelled using the same MCC constitutive law and parameters as for the parametric studies (given in Section 4.2). The steel and grout were assigned elastic parameters in terms of Young's modulus and Poisson's ratio values of which were provided by the respective suppliers. These materials were modelled using a basic elastic model.

The parameters used were:

- for the steel \( E_{\text{steel}} = 199150 \) MPa and \( \nu_{\text{steel}} = 0.3 \);
- and for the grout \( E_{\text{grout}} = 3300 \) MPa and \( \nu_{\text{grout}} = 0.3 \).

4.6.3 Interface conditions between nail and grout

The objective of these analyses was to assess conditions within the soil to establish whether cell-action effects caused reductions in radial stress (i.e. the normal stress acting on the nail) and hence shear stress in the critical material alongside the nail boundary. It was also to establish to what degree the results were influenced by whether the thin radial sleeves were 'bonded' to the grout or able to slide freely past it. In practice the radial sleeves were lubricated with a light coat of silicon grease to minimise the effect of grout bonding to them. This was to avoid the development of axial force on the sleeves which would diminish the effectiveness of the passive gauges used in the strain gauge arrangement within them.

The case where the interface between the grout and the sleeve was fully frictional is denoted DETNAIL_A. This was achieved by having common nodes between the grout elements and the shell elements as shown in Figure 4.8a.

The second case where the outer radial sleeve surface was to be assumed frictionless is
denoted DETNAIL_B. In this instance the frictionless boundary was simulated using tied degrees of freedom between nodes in the beam and corresponding adjacent nodes in the grout elements as shown in Figure 4.8b. The shell elements were 'floating', being held in position by the fixed radial displacement condition between nodes A and B. A constant distance was maintained between adjacent pairs of nodes in the shell elements and the grout elements by tying their radial displacements. These pairs of nodes therefore could move radially by the same amount, the magnitude being governed by the analysis. No restraint was placed on vertical movements and the pairs of nodes could move vertically with respect to each other.

4.6.4 Results from the runs DETNAIL_A and B

The results from the DETNAIL_A and B runs are shown in Figures 4.9 and 4.10 respectively. The 'distance' on the abscissa is that from the free end of the nail. Five plots are shown, for each case, of radial and axial stress, stress level and shear stress in the soil adjacent to the grout and also shear stress in the grout. Lines corresponding to increasing degrees of axial displacement are shown.

The results indicate that there is little difference in the distribution of stress in the soil regardless of whether or not the grout bonds to the radial sleeve. The main difference is that shear stresses develop after smaller displacements for the fully frictional case in the early stages (DETNAIL_A); after 8 mm displacement the profiles are essentially the same. There are also similar differences in radial stress but these are not significant.

The effect of the bonding is quite evident when considering the shear stress in the grout itself as shown in Figures 4.9e and 4.10e. In the fully frictional case (DETNAIL_A) grout stresses are about 20% greater than those developed in the adjacent soil (note that the scales in graphs (d) and (e) are different). In the case where the grout slides freely over the radial sleeves (DETNAIL_B) no shear stress develops in the grout over these lengths, otherwise the distributions are about the same.

In both cases, large concentrations of shear stress are developed in the grout at the free ends of each of the radial sleeves. These result from the short lengths of unsupported grout at the ends of the radial sleeves, as can be seen in Figure 4.8 (the distance between node B and the steel element). It is surprising that these stresses are not reflected in the soil shear stress profile. There is a marked change in the sense of the shear stress in the grout modelled for the fully frictional case, it changes from negative to positive over the very short lengths of these regions.

There is little difference in the results from the detailed analysis with the fully frictional case (DETNAIL_A) compared with those obtained from the parametric studies with similar sample and nail dimensions (c.f. Figures 4.7 and 4.9).

4.6.5 Conclusions from the detailed analyses

The following conclusions can be drawn from the detailed numerical analyses of the
prototype 2 instrumented model soil nail (Figure A7.2.1).

(i) The profiles of radial stress indicate that cell-action effects are negligible for this design of radial stress measuring device (with a thin sleeve) when installed in clay. There is little disruption to the shear stress profile either.

(ii) The results indicate that there are no adverse effects on soil stresses regardless of whether or not the grout bonds to the thin radial sleeves.

In practice other problems were encountered with the devices from influences outside the conditions modelled numerically, e.g. temperature. The passive gauges were affected by axial stresses causing them to become essentially 'active'. Another mechanism, that could occur in the case of sands, is arching around the nail, where radial stresses are translated into circumferential stresses by virtue of the particulate nature of the material and the structural arrangement of the grains. This effect is discussed in Chapter 8 in the context of the sand test results.

4.7 ANALYSES WITH SMALLER DISPLACEMENT INCREMENTS AND REFINED PARAMETERS

After performing the main programme of clay tests, it was decided to carry out further numerical analyses to obtain a better understanding of conditions during the early stages of shearing. An instrumented nail was incorporated in the samples during these tests, from which it was observed that after an initial progressive build-up of interface shear stress at the top of the nail there was a sudden decrease while axial displacements were still less than 1 mm, subsequently the rate of development decreased.

Relatively large displacement steps were used in the parametric studies starting with an initial value of 0.5 mm. The observed change in response occurred at displacements of about 0.3 mm. It was therefore decided to rerun the analyses used for the parametric studies but with reduced displacement steps. The first step was set at 0.05 mm for two increments followed by 0.1 mm for five increments and increment sizes then progressively increased, until a final total displacement of 16.0 mm was reached.

The opportunity was also taken to implement updated input parameter values that are more relevant to triaxial testing of kaolin in axial extension. Two series of analyses were carried out using different approaches with the MCC constitutive model. The first was performed as for the parametric studies with $M$ (the slope of the critical state line) held constant. In the second, the effect of holding $\phi_c$ constant was investigated.

4.7.1 Boundary conditions and input parameters

The two analyses were identical except that in the first $M = 0.76$ was used and in the other a value of $\phi_c = 26^\circ$ was adopted, each being held constant in the respective runs.
The boundary conditions were the same as described in Section 4.3.2 and Figure 4.1 with initial stresses of 250 kPa, conditions of drained shearing and the same gap boundary control to minimise adverse effects in the region where the nail passes through the top platen.

The sample and nail dimensions used in practice were chosen so that experimental conditions would be modelled as closely as possible. Sample dimensions implemented were height and diameter of 150 mm and 152 mm respectively with a grouted nail diameter of 9 mm.

4.7.2 Results from the analyses

The results from both runs are not presented here as there was little noticeable difference between them except for the following comments concerning conditions on the nail boundary.

(i) There was no discernable difference in values of radial stress.

(ii) The constant $M$ analysis gave slightly higher values of axial stress towards the fixed end of the nail, which are negligible at small strains.

(iii) Interface shear stresses were essentially the same with differences less than 1 to 2 kPa.

(iv) Shear stresses were mobilised slightly more quickly with the constant $M$ analyses.

The results from the constant $\phi' = 26^\circ$ analysis are given in Figure 4.11 which show the development of axial and radial stresses and shear stress and stress level with increasing displacements. The results are very similar to those from the initial parametric studies with the corresponding dimensions. Decreasing the value of $M$ has caused a reduction in axial, radial and shear stresses (comparing values at 2 mm and 16 mm displacement) but otherwise differences are small.

The results from the early stages of the analysis with the small initial displacements indicate that radial as well as axial stresses along the nail boundary decreased initially. This trend was only observed for the axial component of stress in the original analyses. The radial stress started to increase again after a displacement of about 0.4 mm, corresponding to when the drop in interface shear stress at the top of the nail was observed in the experimental tests. After displacements of about 2 mm axial as well as the radial stress had started to increase and by 5 mm their magnitudes were greater than those applied at the boundaries.

The results from these final analyses are discussed further in the context of the measurements from the experimental tests in Chapter 10. Contours of stress across an axisymmetric slice are presented to give further insight into conditions within the sample. Analyses were also performed where the nail is pulled out of the sample to simulate the conditions of a pull-out test. The results from these analyses are also presented and discussed in Chapter 10.
CHAPTER 5

Experimental apparatus and procedures for clay testing

5.1 INTRODUCTION

The objective of this study, as discussed in Chapter 3, was to investigate the build-up of stresses that occur along the soil-nail boundaries during the construction of a soil-nailed excavation. Due to the impracticability and size constraints of modelling a full length nail, attention was focused on studying the behaviour in the active zone between the stress relieved face and the plane of $T_{\text{max}}$ (i.e. the zone where shear stresses exerted on the reinforcements act outwards towards the face of the excavation, see Figure 1.5). The boundary conditions for the clay tests were chosen to model this region. In some respects, but to a much lesser degree, because of symmetry in the full-scale ground conditions and the use of side drains in the experiment, the mobilisation of interface stresses in the passive zone from the stress relief at the face is also modelled using these conditions. The difference is that in the passive zone the generation of interface stress is mostly induced from the action of the nail being pulled from within the active zone (as discussed in Chapter 1).

The approach taken in the experimental investigation was to perform single nail element tests within a large effective stress triaxial apparatus with the nail positioned along the axis of the sample. Accurate control and monitoring of boundary stresses acting on the sample during testing is thus possible whilst also allowing interface stresses on the nail to be measured.

The boundary conditions chosen to model the soil-nail situation (as described in Section 1.5) were satisfied using the following conditions.

(i) Stress-relief from the excavated face was achieved by unloading the sample axially under a constant cell pressure allowing drainage from the sample; the extension rate was kept sufficiently slow to ensure that there was no development of excess pore pressures.

(ii) The location along the nail where there is no relative movement between nail and soil (plane of $T_{\text{max}}$) was modelled by rigidly fixing the nail to the lower sample platen.

(iii) As it was the build-up of stress along the nail in the active zone of the soil that was being studied, the nail was allowed to move freely relative to the top platen. The free end of the nail models conditions at the face of the excavation where axial forces in the nail are zero (assuming no rigid facing connection).

(iv) The smooth transition from positive to negative shear stresses across the plane of $T_{\text{max}}$, where shear stresses along the nail boundary are zero, was achieved by using frictionless (lubricated) ends. Other reasons for applying this condition are discussed later.
(v) The overburden stress was represented by the cell pressure. As this acts radially all around the sample there is an implicit assumption that the in situ soil is isotropic and that the difference in magnitude between the overburden stress above and below the nail is not significant.

In some respects the experimental boundary conditions over-simplify the in situ conditions. However, a more meaningful interpretation of the fundamental mechanisms being investigated can be achieved when the problem is simplified with a minimum of variables.

In order to isolate the effects of installing the nail, a 'wished-in-place' installation procedure was adopted similar to that used by Martins (1983). He demonstrated that the procedure caused negligible fabric disturbance. Microfabric analyses were used to verify that the same applied for this study (presented and discussed in Chapter 9).

The intention within the test programme was to perform a series of tests on samples of the same composition and stress history up to the beginning of the shearing stage. From then on different aspects of behaviour were investigated using in some cases instrumented nails and in others preparing the samples for thin-section analyses (thus covering the processes of interface stress transfer and commensurate fabric changes). Forming samples of similar initial fabric was achieved by preparing them from a high water content slurry, details of which are given in Appendix 9.1.

5.2 MODIFICATIONS TO THE TRIAXIAL APPARATUS

The apparatus used was a large effective stress triaxial cell in which axial and radial stresses could be independently controlled (see Bishop and Wesley, 1975). The particular cell used was originally designed as a plane-strain apparatus (Atkinson, 1973), then subsequently modified to perform conventional triaxial tests on 100 mm diameter samples. Martins (1983) later modified the equipment to perform a series of model pile tests to investigate the shaft resistance of axially loaded piles.

The choice of this cell was made principally because of the requirement to install the inclusion axially into a triaxial sample. Various fittings, connectors and ancillary equipment were available from previous work (Martins, 1983; Mahmoud, 1989) and it was originally intended to utilise these.

After the numerical analysis described in Chapter 4, it became evident that it was necessary to make new components for the experimental apparatus. The sample diameter had to be increased to 150 mm (six inch platens were adopted) with a height of the same dimension. With these dimensions the nail diameter had to be restricted to about 8 mm in order to achieve a reasonably uniform mobilisation of friction along its length.

A new internal system was therefore designed to incorporate the larger sample size and
testing procedures necessary for installing the nail and shearing the sample whilst adhering to the boundary conditions described in Section 5.1. Other alterations to the overall system were also necessary.

Prior to making the new components a series of trials and four pilot tests were performed using a modified version of Martins equipment to test individual components and to practise the experimental procedure: especially that of installing the nail.

5.2.1 Renovation and improvement of ram system

As the apparatus was originally designed to perform plane-strain tests on London Clay, the maximum allowance for travel of the ram system used to control vertical stress, was only 17.5 mm. Such limited travel would limit the consolidation ranges possible and the subsequent extent to which the sample could be sheared in axial extension. Modifications were therefore made to extend its range. Besides wishing to increase the travel, the ram system required stripping down and renovating, as in certain positions the force required to overcome friction was of the order of several hundred Newtons.

The modifications made are detailed in Figure 5.1, with the new components shown hatched. Two spacer rings were fitted to the lower chamber to extend the travel and allow a more common type of Bellofram to be used with an extended range. The original Belloframs were specially made with a D-section flange which meant they were expensive and required several months manufacture time. The central ram shaft was lengthened when a replacement was made and a new Rotalin bearing and barrel fitted. The upper Bellofram against which the cell pressure acts was not changed, but because of the extended travel, it would have to operate over its full stroke rather than a half stroke. This causes increased wear but is not significant in the case of triaxial apparatus use, where the maximum number of cycles would not exceed several hundred over the life of the Bellofram.

After greasing, reassembly, and deairing the ram travel was extended to 40 mm and the maximum force required to move the system freely was reduced to 6 N. This was checked at various times during the testing programme.

5.2.2 Measurement of overall axial force in the sample

In order to achieve accurate control and monitoring of the sample stress state it is necessary to measure axial load directly. Martins (1983) relied in his work, on the fact that there was negligible friction in the ram system, which he checked. He calculated the overall axial stress using the equation given by Bishop and Wesley (1975), which expresses axial stress as a function of the ram and cell pressures and the relative areas of the Bellofram and sample. Such an approach was quite satisfactory for his tests where the magnitude of axial stress was mainly relevant to consolidation, thereafter it was only the pile that was loaded rather than the
sample. There was also concern in this case that the corrosion of the ram shaft might recur with consequent stick-slip behaviour and unknown ram friction over different ranges of travel. This would preclude accurate use of the Bishop and Wesley equation.

Martins' approach was adopted during the pilot tests on 100 mm diameter samples. However, for the main testing programme, a modified Imperial College design load cell was fitted beneath the lower platen as shown in Figure 5.2 (component 19), which shows a cross-section through the internal system of the apparatus. This position was chosen because of the requirement to access the top of the sample for nail installation. The load cell was of 12 kN (2700 lbf) capacity, its base was screwed into the upper platen of the ram system and the top screwed and 'loctited' into a 100 mm diameter component labelled 'load cell connector' in Figure 5.3. Wires from the load cell were soldered onto targets located in a recess in the connector and then fed out into the cell via a waterproofed tube. Prior to sealing the load cell connector onto the component above, the recess was filled with silicon oil, and sealed using an o-ring. This had two purposes: to prevent ingress of water from the cell and to prevent corrosion of targets and wires from moisture.

5.2.3 Measurement of axial displacement

Movement of the lower sample platen was monitored using a submersible linear variable differential transformer (LVDT). The body of the device was clamped in position at one of the internal tie rods while the plunger rested on a fixed attachment to the lower platen (see Figures 5.7 and 5.8). With the upper sample platen fixed rigidly to the top of the triaxial cell, measurements can thus be made of the displacement of the lower platen and axial strains calculated.

Although axial strain is generally calculated in conventional triaxial element tests, in the case of the nail tests it is equally important to consider relative displacements. As discussed in Section 5.1, there is no relative movement between nail and soil at the base, the measurement from the LVDT represents the relative nail to soil movement at the top of the sample where the nail passes freely through the top platen.

During the stages of the test when a nail was installed, the strain conditions within the sample were unlikely to be uniform, the strain values calculated are averages. This subject is covered in further detail during the discussion of the results (see also Section 8.3.1).

Local strain measuring devices were not used. Inaccuracies generally associated with overall measurements of axial displacement (see Jardine et al. 1984) were reduced because of the apparatus arrangement. The main cause of errors were from sample bedding at the ends and the compression of the gauze used to facilitate drainage (described in Section 5.3.3).
5.2.4 Design of platens and connecting components

As shown in Figure 5.2, only the sample end platens themselves were made to the full 150 mm diameter. There were several advantages to this approach. All other components were 100 mm diameter which optimised use of materials and space in the cell whilst also providing the option of testing 100 mm diameter samples using the same components. The resulting overhang of the platens allowed direct access to the sample for drainage, back pressure and flushing lines. Standard 1/8 inch Saren tubing was used for these, connected to the platens using Enots fittings. The thickness of the platens was mainly governed by the requirement for there to be enough space to comfortably attach three o-rings either end of the sample to seal the membrane.

The lengths of the components immediately above and below the platens were calculated to optimise the available space to allow clearance for lines and cables and also provide sufficient room to accommodate features necessary for the installation of model soil nails (discussed in Section 5.3.2). With the access openings used for these operations blocked (as shown in Figure 5.2) the cell could be used in a conventional manner to perform effective stress tests on samples with height and diameter of 150 mm.

5.3 ADDITIONS TO THE TRIAXIAL APPARATUS FOR SINGLE ELEMENT MODEL SOIL-NAIL TESTING

5.3.1 Concept of the ‘wished-in-place’ nail

Martins (1983) describes different possible options for installing an inclusion into a sample; these are reproduced in Table 5.1. He concludes that the method least likely to cause disturbance to sample fabric or effective stresses is by introducing the inclusion into the sample under conditions of zero radial stress.

The procedure for achieving this ‘wished-in-place’ condition is as follows. The sample having been consolidated to some predetermined stress, is isolated from its drainage lines and has its ends flushed with air to remove any surplus water. Otherwise the free water tends to be sucked into the sample during the next stage when it is unloaded. Axial and radial stresses are reduced simultaneously whilst maintaining constant axial sample height; as the drainage is closed the sample volume remains constant. In theory, under these conditions, there should be no change in mean effective stress as there is no volumetric strain. In practice, because the axial strain is held constant, there might be a redistribution of radial and axial effective stress.

At this point, with most of the total stress off the sample, a hole is drilled and a rigid model inclusion positioned centrally within it. The nail, which is of smaller diameter than the hole, is then grouted in place. Once the grout has set, stresses can be reapplied incrementally
as for unloading, *i.e.* whilst maintaining constant axial height and sample volume.

The radial stress acting on the inclusion might be expected to equal the cell pressure (*i.e.* applied radial total stress) during and after reapplication of stresses. In practice, as was observed in the sand test results, rigid-inclusion effects and arching might alter the transfer of applied boundary stress through the sample.

<table>
<thead>
<tr>
<th>Method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i) Consolidating from slurry around a pile</td>
<td>-</td>
<td>Severe fabric disturbance if soil can drag against pile during consolidation. Rigid inclusion effects</td>
</tr>
<tr>
<td>(ii) Jacking or driving a pile</td>
<td>-</td>
<td>Disturbance to stress field and soil fabric</td>
</tr>
<tr>
<td>(iii) Jacking a thin-walled sample tube with inward directed cutting shoe</td>
<td>Minimal disturbance to stress field</td>
<td>Serious local disturbance to soil fabric; axial and buckling rigidity of tube in question</td>
</tr>
<tr>
<td>(iv) Boring a cavity with the sampler under stress and then grouting the cavity</td>
<td>Only suitable for heavily over-consolidated materials</td>
<td>Disturbance to stress field and soil fabric</td>
</tr>
<tr>
<td>(v) Boring a cavity with lateral stresses reduced to zero then introducing a tightly fitting rigid pile; lateral stresses then reapplied</td>
<td>Minimal fabric disturbance if pile undersized</td>
<td>Disturbance to soil fabric and stress field if the pile is oversize; large disturbance to the stress field if the pile is undersized</td>
</tr>
<tr>
<td>(vi) Boring a cavity whilst the radial total stresses are reduced to zero under undrained conditions; cavity then grouted to form a rigid inclusion of same diameter as cavity; lateral stresses then reapplied</td>
<td>Minimal to small disturbance to the stress field; minimal fabric disturbance</td>
<td>Any radial stress measuring device must be expanded while it is in place to fit exactly against the side of the hole</td>
</tr>
</tbody>
</table>

Table 5.1. Summary of possible nail installation techniques (after Martins, 1983)

5.3.2 Auxiliary systems for triaxial testing with the model nail

A hole of 8 mm diameter was required in the base to locate and anchor the nail. During the initial consolidation stage, prior to installing the nail, it was necessary to block this hole (see Figure 5.3). This was achieved using a small slightly undersize piston (component 5 in Figure 5.3), fitted with two o-rings, positioned level with the face of the sample platen. Its position was maintained by a sealed reservoir of deaired water behind the piston. Whenever this piston had to be removed for nail installation, the line to the reservoir was vented and the piston pushed to the base of the hole (its position shown in the figure). The lower surface of the piston was
chamfered so that it could be pushed back up again after a test using hydraulic pressure.

Another hole running normal to the piston chamber was incorporated to enable the nail to be locked in position after it had been installed. This feature was included to satisfy boundary condition (ii) in Section 5.1, i.e. the condition that there was no movement between the nail and the lower platen. Thus the plane of no-relative-movement between nail and soil was at the lower sample boundary. The locking-in-place was achieved by providing a hole at the lower end of the nail through which a 1/8 inch stainless steel pin was pushed, the hydraulic mechanism for this operation is shown in Figure 5.3 (component 3). Pressures within the device and within the piston chamber were isolated by o-ring connections.

The height of this lower component (labelled 16 in Figure 5.2) was kept to a minimum to allow maximum clearance above the top platen and to maximize ram travel.

The component above the top platen (labelled 4 in Figure 5.2) acted as a spacer to allow clearance between the top platen and the top of the cell and also hold it in position. During apparatus assembly it was bolted to the platen and the combined unit then held tightly against the underside of the top plate of the triaxial cell by bolts passing the centralising unit above (component 2 in Figure 5.2). The three components thus bolted together were sealed with o-rings and have a 14 mm hole passing axially through them. This accommodated the following inserts which were required for the model nail tests:
- first a blank for the initial sample consolidation (as shown in Figure 5.2, component 1);
- two guide inserts with 8 mm and 9 mm holes passing through them for drilling and reaming operations respectively;
- finally, a guide through which the free end of the nail passed, holding it in place centrally whilst at the same time allowing movement along its axis (it also sealed off the back pressure).

The latter guide was fitted to the nail before making the final wiring connections in the case of the instrumented nails. It had a groove running down its outer side to vent air between soil and nail during grouting (see component 3 in Figure 5.6). The groove did not quite run to the top of the guide as otherwise leaks would have occurred from the back pressure system. At the top of the guide an o-ring was provided, during grouting the guide was positioned so that the groove was just visible above the centralising unit. When grouting was complete the guide was screwed down, (using two screws passing into the centralising unit), so that the o-ring then sealed off the groove and the outer surface of the guide. The different components used in the grouting operations are shown schematically in Figure 5.6.

**5.3.3 System for flushing sample ends prior to unloading**

As explained in Section 5.3.1, flushing the sample ends with air prior to unloading was necessary to avoid free water being sucked into the sample. During the initial trials, using 100
mm diameter samples, bauxite porous stones were used and it was found to be difficult to expel the free water (evidence of the sample taking in water comes from the moisture content profile through the sample, see for example Figure 9.9).

End drainage conditions were therefore changed to a combination of a thin (1 mm thickness), rigid stainless steel plate which was placed in contact with the sample, behind which was a disc of stainless steel gauze. This had two advantages: first the gauze retained very little water when gently purged with air and any water remaining was isolated from the sample by the plate. Secondly, the face of the plate in contact with the sample was very highly polished to make it as frictionless as possible (i.e. to provide an equivalent lubricated or frictionless end).

The sample drainage conditions and the system for flushing the ends are shown in Figure 5.4. Prior to unloading the sample, the drainage to each end was closed, an air line connected to the flushing line and then using the regulated air pressure, controlling the back pressure through an interface (the volume gauge in fact), the water at each end of the sample was flushed out. The flushing was carried out by using an air pressure slightly elevated over the back pressure (equivalent at this stage to the pore pressure in the sample). The operation was performed for each end of the sample individually.

When the sample stresses were reapplied, after the nail had been installed, the above sequence was repeated again but in reverse with water being flushed in.

5.4 BOUNDARY CONDITIONS IN AND AROUND THE SAMPLE

5.4.1 Provision of lubricated ends

As mentioned in Section 5.3.3, frictionless ends were achieved by using highly polished lightly greased stainless steel end plates. These upper and lower plates had central holes in them of 14 mm and 8 mm to accommodate the inserts and nail at top and bottom of sample respectively.

During the initial trials the ends were lubricated using a system of layers of overlapping, radially cut latex discs with high vacuum silicon grease sandwiched between them (see Head, 1990). There was no positive way of assessing the effectiveness of this method and none of the trial samples was sheared to a sufficient extent to see whether any degree of necking occurred. However, on stripping down the samples, these latex discs were found to be badly deteriorated with no vestige of the grease remaining.

The sample shape during and at the end of the tests with the polished discs indicated that considerable relative sliding between the sample and plate had taken place (i.e. radially inwards). The shape of the sample was still essentially right-cylindrical with maximum deviation in radius of 2.75 mm. A photograph of the sample from the end of clay test NAIL0 is given in Figure
5.5a. A close-up in the vicinity of the platen is shown in Figure 5.5b, the inward radial movement is clearly visible. The edge of the polished plate and the gauze, discussed in the previous section can also be seen in the photograph (note that the bauxite porous stone beneath the gauze was only used to support the sample after removing it from the apparatus).

The use of lubricated ends, apart from satisfying the specific boundary conditions for this case, can be important when considering the mode of failure of an unreinforced sample. Lade and Tsai (1985) investigated the causes and effects of line and zone failure on the strength of soil samples. They found that for cylindrical triaxial extension tests there was a significant decrease in strength due to the development of line failure. However, this occurred whether the ends were lubricated or not, even with samples of equal height to diameter ratios.

These conditions which apply to the experimental set-up used cannot be easily avoided. Lade and Tsai found that testing under plane-strain conditions is more likely to provide zone failure. This would necessitate a completely different approach to the study. Besides this, plane-strain apparatus have other detrimental boundary effects that are avoided under triaxial conditions. For the purpose of this study, the lubricated ends were considered to be the best means of ensuring uniform stress conditions. Also, comparisons and observations between tests can be better justified because the same means of end lubrication were used throughout the testing programme.

Lade and Tsai also observe that the likelihood of line failure occurring increases with OCR and the soils tendency to dilate. The samples tested in this study were normally-consolidated at the onset of shearing and so should be less prone to this effect. As a final point they question which mode might be more relevant to conditions in situ where often line failure might occur due to variations in density and the actual surrounding ground conditions.

5.4.2 Sample drainage

The necessity for lubricated ends meant that direct drainage from the sample end faces was not possible. Therefore drainage was achieved using four filter paper strips approximately 10 mm wide which were wound equidistantly around the sample at a slope of about 1:2 and lapped over the stainless steel discs. Such radial drains have a negligible effect on sample stiffness and strength (Gens, 1982). Drainage then proceeds directly to the back pressure system through the gauze (see Section 5.3.3 and Figures 5.4 and 5.5b).

During consolidation, the drainage lines at both ends of the sample leading to the back pressure system were both open. The original intention during shearing was to drain the sample from only one end thus simulating flow towards the face of the excavation. Fabric changes induced from the confined drainage path would then be encompassed within the sample. Pore pressures at the sample ends and mid-height were monitored independently and checked to
ensure that the shearing rate was not too fast to generate excess pore water pressures.

Despite having the drainage closed at one end, because of the lubricated ends, flow was predominantly radially outwards towards the sample boundaries. As a result, there would be little effect on the fabric whether flow takes place via one or both ends. In hindsight, the effects of drainage are difficult to simulate for several reasons. Suctions would be generated in the immediate vicinity of the face during the initial excavation. Drainage might occur with time, in fact drains are often installed within and at the front of the excavation which would also alter the flow regime. The conclusion is that as the same drainage conditions were used throughout the testing programme, any fabric changes resulting from the radial drainage were common to all samples.

5.4.3 The model soil-nail

The main requirements to be satisfied in simulating a full scale soil-nail are that the nail is rigid and of much greater stiffness than the soil being tested and that it is adequately bonded to the soil.

The diameter of the solid part of the nail was chosen to be 8 mm, this being a compromise between the findings of the numerical analyses and the minimum dimension deemed practicable to instrument. This was then surrounded by a 0.5 mm annulus of grout, bonded to the soil. Details concerning the nail instrumentation are described in Chapter 7. For the work described here, the central 8 mm diameter of the model nail was always of rigid construction, i.e. made from brass or stainless steel. These materials typically have stiffnesses of about one hundred times that of the soil tested at low strain levels. The outer surface of the solid part of the nail was provided with shear keys so that shear stress transfer could be made through the grout to the nail. In all nails a central duct was provided for a length of 1/16 inch diameter hypodermic tubing (and in the case of the instrumented nails, wires from the transducers) through which the grout was pumped.

5.5 THE GROUT AND GROUTING SYSTEM

5.5.1 Required grout properties

The grout finally chosen was that which best satisfied the following criteria.

(i) The grout when set should have a much greater relative stiffness than the soil being tested. However, it also had to be flexible enough to allow deformations resulting from stress changes to be detected by instrumentation installed in the inner core of the nail.

(ii) It should set to full strength and bond to the soil which is in a damp condition.

(iii) It should not react with or absorb water.

(iv) It should be sufficiently liquid when mixed so that flow through the 0.5 mm wide annulus
was possible under nominally low pressures.

(v) Excessive heat from exothermic reactions occurring during setting should be minimal (to avoid affecting nail instrumentation or the saturation of the sample).

(vi) Shrinkage during setting should be minimal.

5.5.2 Grouting system

The annular gap was not considered to be sufficiently wide for the grout to be poured down from the top and fully cover the nail without trapping air bubbles or creating voids. For this reason the grout was pumped in using a syringe through hypodermic tubing which led to a port at the side of the nail towards its base, just above the lower platen (see Figure 5.6). There was thus an upward flow acting only against atmospheric pressure. Any excess flow at the base did not matter as this end of the nail was fixed anyway. However, it was important that the grout did not bond to the nail at the top where it had to move freely. In the early tests this was achieved using several approaches. The quantity of grout being pumped in was carefully monitored; a short wire fed down the groove in the side of the nail guide was used to check for signs of the grout over the top 10 mm of the sample; and finally by liberally applying silicon grease in the gap between the nail and the guide. This system worked in all cases.

A more refined system was used in the final test where a second tube was installed down the inside of the nail that exited just below the upper platen. During grouting air was drawn through this tube using a small venturi device and the air pressure monitored on a gauge. When the grout reached the level of the second tube it was sucked in, creating a marked increase in vacuum as registered by the gauge and grouting was then stopped.

5.5.3 Grout trials and choice of grout

Grout trials were performed prior to the pilot tests using lengths of glass tubing of 9 mm internal diameter and 150 mm length, set in the triaxial apparatus to model the hole in the sample exactly. A nail was installed in the tube and a grout injected down from the hypodermic tubing exiting from the top of the cell.

In this way the flow of grout could be monitored and typical required quantities measured. Once the grout had set (checked using samples retained and cast against small quantities of remoulded clay) the tubing was removed and the glass broken off the grout. The suitability of the grout was then assessed qualitatively, considering its rigidity, flow into the shear keys and presence of occluded air bubbles.

Three grouts were tested:

(i) an epoxy resin from Scott Bader - Kollemox 607 (resin) with Kollercurc ND-1 (hardener);
(ii) a silicon elastomer from Dow Corning - Sylgard 170 A&B;
(iii) an epoxy resin from Maresco (a sub agent of Ciba Geigy) - Marprime W.
The latter of these was chosen. Marprime W is a specially formulated epoxy resin for use in damp conditions which was found to best satisfy the criteria listed in Section 5.5.1.

5.6 THE OVERALL TESTING SYSTEM

A photograph of the apparatus, without a sample but with an instrumented nail in the position where it would be installed during testing, is shown in Figure 5.7. The complete testing system with its ancillary units, pressure sources, transducers and data logging systems is shown schematically in Figure 5.8.

5.6.1 Pressure sources

The primary source of pressure was an air compressor with a maximum supply pressure of about 850 kPa. This pressure was transmitted to the cell, back and ram pressure systems via air-water interfaces. Pressures to the individual systems were controlled using manually operated manostat valves with a working range of 10 to 1000 kPa.

5.6.2 Transducers

Various transducer types were used; the quantities measured and make of transducer are listed in Table 5.2. Transducers installed in the instrumented model soil-nail are specifically described in detail in Chapter 7.

5.6.3 Calibration of transducers

Prior to performing any calibrations the stabilities of the transducers were checked over several days with them all plugged in and hence ‘warmed up’ under the 5 Volt energisation voltage. It was important that the transducer arrangement was not altered after initial set-up unless a new set of calibrations were to be performed, as adding or removing a transducer often resulted in a small change in the resistance of the circuit being energized or a shift in transducer readings. Care was also taken to ensure that all lines and transducers were deaired.

All transducers were calibrated over a range extending beyond that anticipated during any stage of the testing. Generally measurement increments were applied up to a certain value and then an unloading cycle performed to check for drift and hysteresis. Smaller load or pressure increments were applied over working ranges.

New transducers being used for the first time (such as the load cell, pore pressure probe and the Keller transducer) were cycled several tens of times to ‘bed-in’ the gauges.

Having performed the calibrations, the points were regressed using in all cases a least squares linear regression. During the course of the testing programme, checks were made on the transducer readings to check for drift and stability.

A brief summary of the calibrations performed for each transducer is now given.
Load cell

The load cell for calculating axial stress in the sample was calibrated separately over tensile, low compression and full compression ranges. The first two were performed with dead weights applied directly and the latter using a Budenberg calibration rig where the force/pressure applied from a dead weight is stepped up using an hydraulic ram system with differential areas. The load cell could resolve to within 0.01% of the applied load. Scatter of points was generally not significant, however some hysteresis was exhibited during the tensile calibration, (this was not unusual for this type of device), but this disappeared as zero load was reapproached. As the tests were to be principally carried out in extension with load steadily decreasing, only the loading cycle was used for the tensile range. The three ranges of load were regressed individually. As the load cell was to work mainly in tension this was the dominant range and the compression ranges were adjusted to provide a continuous fit to it.

<table>
<thead>
<tr>
<th>Quantity being measured</th>
<th>Type and make of transducer</th>
<th>Working range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample load</td>
<td>IC design load cell</td>
<td>-2700 to 2700 lbf</td>
</tr>
<tr>
<td>Cell pressure</td>
<td>Bell and Howell pressure transducer</td>
<td>0 to 150 psi</td>
</tr>
<tr>
<td>Ram pressure</td>
<td>Soil Instruments pressure transducer</td>
<td>0 to 150 psi</td>
</tr>
<tr>
<td>Pore pressure at base of sample</td>
<td>Bell and Howell pressure transducer</td>
<td>0 to 150 psi</td>
</tr>
<tr>
<td>Pore pressure at mid-height of sample</td>
<td>Druck pore pressure probe</td>
<td>-50 to 150 psi (using a 3 bar filter stone at the face)</td>
</tr>
<tr>
<td>Pore pressure at the top of the sample</td>
<td>Keller pressure transducer</td>
<td>0 to 150 psi</td>
</tr>
<tr>
<td>Axial deformation of sample</td>
<td>Sangamo Weston Controls submersible LVDT</td>
<td>0 to 17 mm</td>
</tr>
<tr>
<td>Sample volume change</td>
<td>MPE Transducers Ltd displacement transducer mounted on an IC volume gauge</td>
<td>0 to 100 cc</td>
</tr>
<tr>
<td>Cell water temperature</td>
<td>Device made in-house using a temperature sensing integrated circuit (LM35 from National Semiconductors)</td>
<td>0 to 100 Deg C (10mV/Deg C)</td>
</tr>
</tbody>
</table>

Table 5.2. Summary of triaxial apparatus transducers
Standard pressure transducers

The transducers for measuring cell, ram and pore pressure at the base, mid-height and top of the sample were all calibrated together in their test set-up positions by linking them through the triaxial cell. The calibrations were performed using the Budenberg calibration rig. The rig has a limited flow capacity for applying pressure before it needs recharging, whilst during pressure application within the large triaxial cell perspex significant expansion takes place. For this reason the compliance was initially taken out of the triaxial system for each increment by manually taking the pressure to nearly that required using one of the manostat valves, and then completing the final part of the increment with the Budenberg. These transducers are capable of resolving to almost 0.1 kPa for most of the working range and have very little scatter or hysteresis.

Pore pressure transducer

The Druck pore pressure probe was only calibrated under positive pressures. However the device operates reasonably well down to pressures of $-300$ kPa. Examples of this effect and discussion are given in Chapter 9; it has also been the subject of extensive research at Imperial College, see Ridley (1993).

Displacement transducer

The submersible LVDT was calibrated on a geared vernier calibration rig and was found to be capable of resolving to 0.001 mm. Again this transducer exhibited negligible scatter or hysteresis.

Volume gauge

The displacement transducer to register changes in sample volume was set to operate over the main central range of the 100 cc volume gauge (the transducer’s range just covers that of the volume gauge). The volume gauge was then linked to a calibrated burette partially filled with paraffin and water and both systems pressurised at the anticipated back pressure of 200 kPa. As the burette only had a 25 cc capacity, two calibrations were performed over the central half of the full range. The calibrations indicated some hysteresis but the errors involved when the final regressions were made were found to be negligible. The volume gauge system can resolve to better than 0.02 cc which represents less than 0.0008 % of a typical sample volume.

Temperature transducer

This device was made for the system following trials with earlier models of the instrumented nail when it was found that often the strain-gauge devices were very sensitive to temperature. The transducer comprised a temperature sensing integrated circuit sealed into a water-tight plug that fitted into the top of the triaxial cell. Temperature variations within the cell water could be monitored for diurnal fluctuations with this device. The instrument was
calibrated by placing it in ice and water at different temperatures recorded using a thermometer. The response was linear and the sensing circuit rated as capable of resolving to 0.9°. This was quite adequate for the purposes required, where the trends in temperature change were more important than absolute measurements.

5.6.4 Data logging systems

Two data logging systems were used during the period of testing. During the initial tests, (trial tests and tests NAIL0 to NAIL4, see Chapter 9), all the cell transducers as described in Section 5.6.2 were locally energized but remotely logged (i.e. signals converted from analogue to digital) using an Orion data logger. In transferring the signals to the Orion, signal resolution was lost due to noise affecting the transfer line, there was therefore a scatter in output of about 3 µV. The Orion data logger system was run in line with a Prime 750C computer. Thus the Orion logged the data which were then transferred and stored on the Prime system using in-house software. The stored data could then be processed using again in-house software.

It was decided after testing the first two prototype model nails that the transducers would be much less affected by noise and have much smaller scatter if they were energized and logged using a more up-to-date Measurements System Ltd (MSL) a/d converter, in line with a microcomputer for storing the data. The MSL unit was capable of transmitting data with the scatter reduced to 0.6 µV (i.e. by a factor of 5) and because of the close proximity of the computer, noise was significantly reduced. The relevance of reducing the scatter is discussed further in Chapter 7.

Tests NAIL2 to NAIL4 were run in this way with the nail transducers logged separately to the triaxial system transducers. In processing the results it was necessary to carefully splice the two sets of data together using time as a common base line.

5.7 SAMPLE PREPARATION FOR CLAY TESTING

The clay chosen for performing the fine grained material tests was kaolin. The reasons for choosing kaolin, its properties and the philosophy behind obtaining repeatable uniform fabrics are given in Chapter 9.

The purpose of the following sections is to describe the experimental procedures used during the initial stages leading up to installation of the sample in the triaxial apparatus.

5.7.1 Slurry preparation

The slurry for each sample was prepared by mixing 10 kg of dry Speswhite kaolin powder with 13 kg of distilled water. The mixing was initially done by hand, followed by several hours in a mechanical paddle mixer, the slurry was then of a smooth creamy consistency.

The slurry was placed in an air-tight container and left for a minimum of one month to
allow full hydration to occur. At several times during this hydration period the slurry was stirred again by hand to remix water expelled by sedimentation.

This procedure was followed to be consistent with previous research carried out at Imperial College (e.g. Martins (1983); Mahmoud (1989)). No references could be found to support the idea behind it. Perhaps the procedure originates from the time when triaxial tests were performed without a back pressure, the extended hydration period possibly increasing the chances of the final sample being fully saturated.

5.7.2 One-dimensional consolidation

The first stage of sample consolidation was within a large oedometer of 9 inch diameter and eleven inches height with an extension piece bolted on. Drainage could take place from the top and bottom of the sample through coarse bauxite stones. A filter paper disc was placed between these and the slurry to prevent clogging. The stones were boiled in distilled water prior to placing the slurry to fully deair them.

Having remixed the slurry by hand to a uniform consistency, it was placed in a vacuum chamber with small paddle mixer incorporated and gently stirred under vacuum for several hours. Because of the quantity of slurry being used it was necessary to do this in two stages.

The oedometer with its lower stone deaired was set up as shown in Figure 5.9 with the sides bolted on. The inner surfaces of the walls were lightly smeared with silicon grease to reduce friction at the sample sides. Prior to laying the filter paper on the base, the stone was flushed with distilled water using a pressure of about 10 kPa. The filter paper disc was then placed with the drainage off and about 50 mm of water in the base, care being taken not to trap any air.

The slurry was placed in the oedometer using a small scoop, which was immersed each time beneath the water in the base (which rose as the slurry height did) to avoid trapping air. Care was taken to place the slurry as evenly as possible working from the sides inwards. Although no signs of bedding could be seen by eye, with either wet or dry samples, however, these features were evident in the microfabric analyses made later in the study. As the layers lie principally in a plane perpendicular to the axis of the nail and to the fabric features resulting from shearing the sample, their presence is thought unlikely to affect sample behaviour adversely. The oedometer was filled to the top so that all the excess distilled water was displaced and the top surface planed off with a straight-edge.

Drainage was opened to the base (without applying any suction) and the sample left overnight to sediment and consolidate. The remaining slurry was used to refill the oedometer (typically about 25 mm settlement would have occurred). According to the references discussed in Appendix 9.1, the fabric should not have been affected by this ‘topping-up’ as sample stresses
were well below the 15 kPa quoted.

Once the full quantity of slurry was in the oedometer, consolidation was started. Initially a suction of about 10 kPa was applied to the sample through the base drainage. After one or two days the top cap with saturated stone was carefully placed in position. Consolidation was allowed for another period of about 48 hours when the hanger system was set up so that everything was balanced (see Figure 5.9). Loads were applied incrementally to the hanger such that the first load caused an axial stress equivalent to twice that from the applied suction and top cap, \((i.e. 25 \text{ kPa})\). Each load was applied for about two days, until the sample had sufficient strength to withstand extrusion from sides of top cap during the next load application. The final vertical consolidation stress was 200 kPa and this was left on for several days until final consolidation and creep were complete. The final consolidation stress chosen was dictated by the need for the relatively large sample to be strong enough to handle and trim.

The sample was then ready to be removed. Surplus water used to prevent drying of the top cap and sample, was syphoned off, the drainage closed and the sample unloaded as quickly as possible. With the hanger system removed, the sample within the confining ring was lifted off the base of the oedometer so that its base was exposed. The ring was independently supported several millimetres above a strong, clingfilm covered board and the sample extruded by applying a light pressure to the top cap. Once the sample had started moving and made contact with the board, the top cap was removed and the confining ring carefully slid off over the top of the sample.

The fully exposed sample cake was of a firm consistency. Its dimensions were measured, and about 5 mm trimmed off the outer surfaces to remove the filter papers, and any surplus grease and disturbed zones. Several moisture content samples were then taken to check sample uniformity and assess possible drying of the sample during storage. The cake was wrapped in clingfilm, avoiding air pockets and the whole sample coated with several layers of paraffin wax. All samples were stored for several months prior to testing as it was necessary to start the sample production at the beginning of the study (as each sample took more than one month to prepare). They therefore all had similar ageing periods.

5.7.3 Sample preparation for triaxial testing

It was only possible to obtain one triaxial sample from each cake because of the triaxial sample size required. The cake was roughly trimmed around the sides before placing it in the soil lathe where it was trimmed first with a wire saw and then with a straight-edge to its final diameter of six inches. Using the central part of the sample and trimming 1½ inches from the outer radius should have removed most of the soil with fabric disturbance from side-wall friction in the oedometer. The sample was quite vulnerable at this stage being too heavy to move by
hand without damaging the prepared sides. The mould for trimming the sample ends was therefore assembled around the sample whilst it was still on the lathe platen and then carefully rotated onto its side for end trimming. Again a wire saw was used followed by a straight edge, (the wire saw is not a suitable tool for the fine trimming of soft soil as it only cuts and does not remove the waste soil, which tends to stick to the sample). After trimming each end square, a highly polished, lightly greased end-plate was placed in position (see Section 5.4.1 for reasons for these plates). The sample could then be handled with greater ease. At this stage the sample was weighed and the four side drains affixed (see Section 5.4.2).

The membrane for enclosing the sample was prepared the day before. It was checked for leaks and a grommet fitted, at its mid-height and sealed with two coats of latex, for holding the Druck pore pressure probe in position against the sample.

A large membrane stretcher operated with a vacuum pump was used to position the membrane. Care was always taken to ensure that the grommet position was midway between two side drains and at the mid-height of the sample (the position of the mid-height probe and the radial drains can be seen in Figure 5.5a). Each end of the membrane was then rolled back to expose about 1 cm length of sample. A short loop of membrane approximately 2 cm wide, previously trimmed from the original length of membrane, was positioned so that its mid-height was in the same plane as the gauze to be placed behind the polished plate. These membrane loops were to prevent puncturing of the main membrane by it being forced onto the gauze (some edges of the gauze discs remained ragged despite being dulled with solder). The gauze discs were put in place and the membrane ends slightly rolled out to hold them in place. The sample was then ready for installing in the apparatus.

Prior to installing the sample, various checks were made to see that all lines were deaired, the piston and nail locking systems were working, the probe was saturated and so on. Three o-rings for sealing the sample membrane were placed ready behind the upper and lower platens. Output from the cell transducers would also have been monitored for several days to check for stability and drift.

The sample was slid onto the lower platen and the initial load cell reading taken. The lower part of the membrane was rolled down, all air pockets removed and the o-rings fitted, (this was done by hand as there was no room for a stretcher). The mid-height probe was then installed, being sealed by two o-rings and two coats of latex around the grommet. Contact with the upper platen was made by slowly raising the ram using the hand pump (see Figure 5.8) until a small load was registered on the load cell (this occurred after the gauze was fully flattened). The membrane was rolled up, making sure the protecting membrane loop was in position and o-rings fitted as for the base of the sample. The membrane ends were then carefully and tidily
rolled back. This completed installation of the sample in the apparatus.

The triaxial cell perspex was lowered over the cell and bolted in place and the cell water pumped in. Under these conditions with no pressure applied (and the ram system closed off to prevent axial movement) the sample was in an almost initial isotropic stress state following the undrained unloading from the $K_o$ consolidation in the oedometer.

### 5.8 EXPERIMENTAL PROCEDURES FOR CLAY TESTING

The specific procedures adopted during the different stages of the triaxial testing are covered in the following sections. Sample and nail behaviour during the relevant stages are presented and discussed in Chapter 9.

#### 5.8.1 Isotropic consolidation

Throughout the testing programme, isotropic consolidation stress paths were followed prior to nail installation and shearing. This path was chosen for ease of control and achieving repeatable tests given that the applied cell and axial pressures were operated manually. Martins (1983) found that the initial consolidation stress ratio did not affect peak pile strength in his work (although his testing was not the same as that performed in this study: he loaded the inclusion directly, while here it was loaded indirectly by changing the sample stresses).

The consolidation stresses were applied in one step by increasing the cell and ram pressures simultaneously up to the final values with the drainage closed. As all tests were performed under a back pressure of 200 kPa, the final cell pressure required was 450 kPa, giving a sample effective stress of 250 kPa once the drainage was opened and consolidation was complete. The ram pressure required to maintain isotropic conditions was calculated taking into account the relative difference in sample and Bellofram areas and the weight of the sample and fittings below it which also bear on the ram system, (e.g. platen, load cell and connecters) using the equation given by Bishop and Wesley (1975).

The final consolidation stress of 250 kPa was chosen as it represents a feasible stress level that might be encountered in situ with a deep nailed-soil excavation. The value is slightly higher than might be expected for a near surface normally-consolidated soil deposit but it was desirable that the final stress exceeded that reached under one-dimensional consolidation. Evidence that features from the one-dimensional consolidation were overcome by those from the isotropic consolidation can be illustrated by means of strain increment vectors. This approach was used by Martins and is based on the precept that when clay undergoes virgin consolidation at a constant stress ratio, $\eta$, the plastic strain increment vectors lie at a fixed orientation with respect to the consolidation stress vector. Therefore as the sample consolidation becomes more dominated by the applied isotropic stress, the ratio should asymptotically approach a constant
value. Note that as there should be no shear stresses during isotropic consolidation the value of the ratio for the consolidation in this study should be about zero.

Consolidating the sample to a higher stress level was also avoided in anticipation of the undrained unloading stage when suctions generated in the sample would reflect the consolidation stress. The pilot tests revealed that the mid-height probe could sustain suctions of about 250 kPa for a period of hours (necessary for nail installation) without cavitating. This was another reason for limiting the upper bound of the consolidation stress.

Consolidation was commenced by opening the drainage top and bottom under a back pressure of 200 kPa. The back pressure was necessary to help saturate the sample. Once final effective consolidation stresses were reached the sample was left to age until the axial and volumetric creep rates were less than one hundredth of the proposed shearing rate (usually taken to be 0.5 % axial strain per day). The intention was to remove creep effects so that the sample height could be accurately maintained during the next stages of nail installation, to minimise fabric disturbance.

5.8.2 Flushing the ends of the sample and unloading

The procedures for nail installation commenced once the consolidation stages were complete and creep criteria satisfied. The nail was in all cases installed in the ‘wished-in-place’ condition as discussed in Section 5.3.1.

Prior to unloading the sample, the free water in the gauze discs behind the polished plates was gently flushed out with air. This was to prevent water being sucked into the sample while the boundary stresses acting on it were unloaded, i.e. to maintain undrained conditions. Full details concerning this procedure are given in Section 5.3.3.

During undrained unloading, axial and radial total stresses were simultaneously decreased incrementally in steps of approximately 10–15 kPa whilst maintaining constant axial height. Prior to unloading a series of readings from the internal LVDT were taken and an average calculated to the nearest 5 μm, the height of the sample was then maintained to within about ±10μm of this value.

5.8.3 Installing the model soil-nail

Having unloaded the sample, the blank insert, used to block the hole in the upper platen during consolidation, was removed and the first of the guides, with an 8 mm diameter hole, fitted in place. This guide was for drilling the pilot hole for the nail, the guide hole was made to a very close tolerance with the drill size so that possibility of lateral play was minimised. The drill was a standard 8 mm wood bit attached to a length of brass of the same diameter.

Drilling was carried out by hand twisting. The hole was advanced in lengths of 15 mm (i.e. ten borings over the sample length) after which it was retracted and the cuttings placed in
a moisture content tin which was immediately sealed. The tins were weighed at the end of the drilling using a high accuracy balance (to ± 0.0001 grammes), necessary because of the very small quantities of sample. It was thus possible to obtain a moisture content profile over the sample height.

Once contact was made with the base platen and the hole cleaned (by rotating the drill once or twice at base) the drill and guide were retracted. The second guide with 9 mm diameter hole was then fitted and the hole in the sample opened out to 9 mm using a sharp reaming tool. This was also retracted every 15 mm length to avoid clogging up the blades. The hole was then at its final diameter.

The 8 mm guide was refitted and the drill used first to clean any loose cuttings at the base of the hole and second to push the piston in the lower platen to the base of its hole; this was done after the reservoir behind the piston had been vented, (see Section 5.3.2 and Figure 5.3). The drill and guide were then once again removed. The hole was then ready for the nail to be inserted.

The nail was lowered to the base of the hole, taking care not to touch the sides of the cavity, and the guide already fitted to it (as in component 3, Figure 5.6) located in the upper centralising unit to prevent lateral movement. It was then fed to the base of the hole in the lower spacer block so that its end was in contact with the piston previously pushed down (see Figure 5.3, components 1, 4, and 5).

The nail was rotated as required so that the hole in its end lined up with the nail locking device. The pressure difference across the piston in this device (component 3 in Figure 5.3) was increased using the hand pump, until the pin was driven through the hole in the base of the nail. Movement of the pin was observed from the outside, through the cell perspex; its movement could also be felt from the drop in pressure at the hand pump. The nail at this stage was securely locked in place at the base and held central at the top by the guide which was protruding about 10 mm to allow air to escape during grouting, (see Figure 5.6 for details).

The Marprime W epoxy resin used as the grout material was prepared, mixing 5 cc in the ratio 4:1, resin to hardener. This was sucked up using an hypodermic syringe, and as much air removed as possible. The grout was gently injected via the hypodermic tubing leading from the top of the nail to the exit point just above the lower platen. Approximately 3 cc was required to fill the annular void but generally more was used because of loss into the base. In the final stages the grout level was checked frequently using a length of fuse wire as dip-stick down the vent groove in the guide (see component 3, Figure 5.6). Grouting the nail into the guide would invalidate the test. In addition to checking the grout level to avoid over-pumping, liberal quantities of silicon grease were applied to the underside of the guide and the gap
between the nail and the guide. The more refined method of detecting when the grout was at the required level, described in Section 5.5.2, was used for the final test.

Once the grout had reached the top of the sample, the syringe was removed and the guide pushed down and sealed in place. The grout was fully workable for about 30 minutes, after which it started to stiffen, giving sufficient time for these operations. Surplus grout was applied to remoulded clay from the reaming operation to give an indication of when it had set against the damp soil. The grout was usually left for 6–7 hours to ensure that it had set fully.

5.8.4 Reloading and refleshing the ends of the sample

Reloading the sample was performed in the same way as for unloading except in reverse: i.e. the cell and ram pressures were increased simultaneously in small increments of 10–15 kPa whilst maintaining the sample height constant. Towards the end of the stage it was usually found that the full stresses could not be reapplied without allowing changes to the sample height. A compromise therefore had to be made between maintaining sample height and achieving the previous stress state. Generally the stress state was reapplied. The reasons for this and the consequent effects are given in Section 9.3, where the test results are discussed.

Prior to reopening the drainage, the sample ends were flushed with water using a water pressure slightly higher than that registered by the probe. Each end was flushed individually. Flushing was continued for a few minutes until no air could be seen coming through; bleeding connections were made back to the pressure transducers and the flushing pressure source isolated (details of the flushing system are given in Figure 5.4).

This refleshing and sealing of the drainage was carried out as swiftly as possible after the reloading stage to avoid sample consolidation by expulsion of water into the voids at the sample ends. At the end of this operation the sample was still in an undrained state.

5.8.5 Sample reconsolidation

Under ideal conditions, reconsolidation would not have been necessary at this stage as the sample effective stresses would not have changed and the pore pressures would return to those acting before unloading. In practice some water from the sample ends and radial drains was always been sucked in and hence reconsolidation was necessary.

Reconsolidation commenced as the drainage was opened and back pressure of 200 kPa reapplied. The effects of reconsolidation on sample strains are discussed in Section 9.3.5.

At the end of this stage the sample was aged to minimise creep effects, i.e. up to the point when volumetric and axial strain rates were less than one hundredth of the shearing rate, taken to be 0.5 %/day (as for the isotropic consolidation stage).

5.8.6 Shearing in drained extension

Shearing was performed using a constant rate of strain pump (CRSP) system, comprising
a hand pump, in this instance turned by a driving motor, linked through to the ram pressure system (see Figure 5.8).

The pre-shearing procedure was temporarily to isolate the sample from the ram pressure interface, and connect the latter to the CRSP so that the ram pressure was also being supplied to the hand pump. The driving motor was then started, moving in the required sense (in this case to draw water from the ram system), at a higher speed than required to take the slack out from the gear system and connecting rods. At this stage the pump was only moving water from the interface and so there was no change in sample stress. This was usually left for about half an hour after which the speed was set to that required.

The speed at which water was moved into/out from the pump was controlled by an eight speed gearbox for the coarse range and this bracketed speed was fine tuned using a variator, both linked to the driving motor. The driving system had been calibrated previously to give ram speeds for each gear over the variator range, so that the samples could be sheared at a fixed rate of axial extension. The rate of shearing was generally less than 0.5 %/day to avoid excess pore pressures being generated.

With all the slack removed, the interface ram pressure was isolated so that the ram pressure was being decreased by the CRSP system thus commencing shearing.

The results from monitoring the sample and the instrumented nail during these stages up to and including shearing are presented and discussed in Chapter 9.

5.9 UNLOADING SAMPLE AND REMOVING IT FROM THE APPARATUS

At the end of the shearing stage the sample stresses were held constant by setting the ram pressure through the interface to match that currently in the CRSP system. This was done so that the sample could be unloaded in a controlled manner prior to removing from the apparatus. The procedure was necessary for the samples being prepared for microfabric analysis so that spurious fabric features were not induced by changes in sample height.

Once the ram pressure was connected, the CRSP system was shut off, and the sample stresses held constant. The drainage was closed and the sample ends flushed with air using the same procedure as prior to unloading the sample for nail installation (Section 5.8.2). The sample was then unloaded by decreasing the cell and ram pressures simultaneously in small increments whilst maintaining a constant sample height. Once the cell pressure had been reduced to zero, the cell was drained and the perspex removed. All surplus water was mopped up and the o-rings top and bottom moved aside.

5.9.1 Dissection of samples with instrumented model soil-nail

In the case of tests where the microfabric was not being studied, the membrane was cut
away and the radial paper drains removed with the sample in place in the apparatus. Sample dimensions were carefully measured to assess uniformity of shape after shearing and the effectiveness of the polished ‘lubricated’ plates.

Dissection commenced by cutting back the clay to see if any visible fabric features could be observed and also to check the condition of the nail and uniformity of the grout. Moisture content samples were taken at various positions within the sample during this operation. The grouted nail was wiped clean and its dimensions carefully measured for subsequent calculation of interface shear stresses. In the case of test NAIL2 the instrumented nail was left set up in the cell in the same position as for testing, and calibrated in place with the grout thickness and condition unaltered (see Chapter 7 for details).

5.9.2 Dissection of samples to be prepared for microfabric analysis

A different procedure was used for samples being prepared for microfabric analysis. The sample had to be removed from the cell with the nail intact within it. It was therefore necessary to unbolt and remove the top of the cell and attached guide and fittings and where necessary the transducer box at the top of the nail. Care was taken to avoid moving the nail or disturbing the sample during these operations. The upper polished plate was left in position and the nail unlocked from the base using the nail locking device in reverse. The sample could then be carefully lifted from the lower platen using the polished plate to support it. It was weighed and set on a stand with a hole to accommodate the nail. The membrane was then removed and again the sample was dissected to obtain moisture content profiles except that a central slice was left as described in the next section.

5.10 SAMPLE PREPARATION FOR MICROFABRIC ANALYSIS

The microfabric analysis was carried out by impregnating the saturated soil sample with molten Carbowax (polyethylene glycol) which is fully miscible with water and thus penetrates the sample and replaces pore water in the voids by wax. Once this wax has cooled it solidifies and becomes very hard and the sample can be treated as a rock. Thin sections from the sample can therefore be cut and mounted on microscope slides for examination. These techniques are described by Tchalenko (1967).

5.10.1 Sample preparation

A diametric slice was cut from across the centre of the sample about 25 to 30 mm thick with the nail in the middle. In this way the volume of the sample to be impregnated was reduced considerably whilst a sufficient thickness of sample was left to allow for cutting disturbance and to prevent excessive cracking during wax penetration and cooling. The upper and lower ends of the slice were trimmed back to reveal the extent of the grout and to check on
its uniformity. The sample was carefully wrapped in strong tissue and placed on a fine gauze sheet supported within a strong coarse mesh tray. The tray was carefully lowered into a bath of pre-prepared molten Carbowax and placed in a constant temperature oven at 60°C. It was left in the bath for about two weeks to allow the wax to impregnate, the wax around the sample being stirred frequently. The sample was then immersed in a fresh batch of wax and left for a further week to ten days. The necessity to change the wax depends on the size of the sample being impregnated and the nature of the soil. The kaolin samples were quite large and of quite a high water content, the precaution of changing the wax was therefore taken to ensure that all water in the sample was fully replaced.

At the end of the impregnation period the tray was lifted from the bath and the surplus wax allowed to drain and the sample left to harden at room temperature. After about a day the sample was marked up, enclosed in polythene bags and stored in a fridge until ready for thin sectioning.

5.10.2 Preparing the thin-sections

Sections from the hardened slice were usually cut in two orthogonal planes: one diametric by splitting the sample along its axis, running just off centre of the nail to provide a vertical section and the other perpendicular to the nail axis for a horizontal section.

Initial cuts were made using a diamond saw with oil lubricant and coolant. The nails were always removed prior to sample grinding. Previous experience (Martins, 1983; Mahmoud, 1989) has shown that the difference in hardness between the nail and the sample causes problems in grinding sections of uniform thickness and also 'smearing' can occur if the inclusion is made of brass. A typical sequence of cutting and grinding of the specimen is shown in Figure 5.10.

The principles and methods involved with the observation and analysis of thin-sections are discussed in Section 9.7 along with the results from the slides prepared from three of the tests.
CHAPTER 6

Experimental apparatus and procedures for sand testing

6.1 INTRODUCTION

In practice, soil nailing as a means of earth reinforcement is mostly carried out in granular, often cemented soils and stiff fine-grained materials. At the outset of this research the intention was to concentrate on the study of nail-soil interface conditions in clays. After the development of the instrumented nail which was capable of measuring radial stresses and axial forces, it was decided that the study could be usefully extended to cover sands as well. The scope of testing eventually completed involved tests on sands of three grain sizes representing fine, medium and coarse sands in states varying from a loose to a dense condition.

This chapter describes the apparatus that was developed and the procedures used for setting up the samples and carrying out the tests. Details of the sands tested and the results from the tests are given in Chapter 8.

6.2 DESIGN AND DEVELOPMENT OF APPARATUS

6.2.1 The basic concept

The boundary conditions adopted for the clay testing are equally applicable to other soils and so almost the same set-up was required. It was not possible to utilise the triaxial cell used for the clay testing because of insufficient room and mobility within it required for setting up sand samples. In addition to this constraint, it was decided that improved, more uniform conditions would be achieved using a larger sample for the increased grain size of a granular material.

As setting up a large sand sample in a conventional triaxial system was not viable with the equipment available (i.e. by using split moulds etc.), a different approach was adopted. A cylindrical chamber was used, in which a pressure source within a membrane was incorporated within its outer curved boundary for applying the radial stress. The sample was formed by filling this chamber and sealed using another pressure membrane at the top for applying axial stress. Details of the apparatus and testing procedures are described in the next sections.

6.2.2 Modifications to a triaxial cell

In selecting a suitable chamber for performing the sand tests, several alternatives were considered, mostly utilising large diameter tubes of plastic or metallic materials, with specially designed end components for sealing the tube and applying an axial pressure source. A chamber was created from a disused triaxial cell suitably modified for the purpose as shown in Figure 6.1,
(the cell is inverted).

The triaxial cell, was a version manufactured by Shape instruments. It had the useful feature of having one end of it split into two parts (10 and 12 in Figure 6.1) so that quick-release nuts could be activated to release the upper part of the cell (i.e. the perspex, top of cell and lower ring, which were clamped together with three tie rods: denoted 16, 21, 12 and 14 respectively in diagram). It was therefore possible to design independent systems for applying radial and axial pressures to the sample.

Radial pressure system

The radial pressure system comprised a membrane (18), usually used for testing 6 inch diameter triaxial specimens (a thicker membrane than usually supplied, of 0.020 inch thickness was obtained from the manufacturer), which was stretched over one end of the cell perspex (16) by about 20 mm and held in position by an o-ring. The perspex was then inverted, whilst holding the free end of the membrane up, and fitted into the base of the cell (21) and the tie rods (1 and 14) screwed into place. A double thickness of plastic mesh (17) was then placed around the inner face of the perspex (16) and the intermediate adaptor ring (13) located on the upper edge of the perspex. The free end of the membrane (18) was then carefully stretched uniformly over the adaptor ring (13) and the clamping ring (12) fitted in place and firmly bolted down against the three tie rods (14). The intermediate adaptor ring (13) was specially made to allow access of lines A and D into the void between the membrane (18) and the boundary wall of the chamber (i.e. the perspex (16)). The 3/16 inch nylon lines A and D were Loctited into position in holes passing through the ring; small discs of fine gauze were fitted over the exit holes adjacent to the membrane to prevent it being sucked into the holes during vacuum application. These details are shown in Figure 6.2. The procedure for setting up the radial pressure source is described in Section 6.3.2, it was often necessary to set up this system prior to each test.

Axial pressure system

The axial pressure system was fitted to the top cap (10) in the originally intended position of the sample pedestal component. This system comprised a specially made membrane (4 in Figure 6.1), which is discussed in detail in Section 4.2.4, held in position (as shown in Figure 6.2) by a stainless steel plate (6) which was clamped to the underside of the top cap by three bolts. Two holes in the plate (6) were provided for lines B and C which were sealed using standard fittings and Dowty washers as shown in Figure 6.2. The top cap with attached axial pressure system could then be lowered into position on the clamping ring (11) and held in place by the three tie rods (1): spacer rings (5) and nuts on the threaded ends of the tie rods replacing the original quick-release fittings.
The system as described above could be used to test sand samples, without any nail, under triaxial stress conditions. Further details of the control of the boundary stresses are given in the next section.

In order to test with the nail placed axially within the sample and under the boundary conditions described in Chapter 3, certain provisions had to be made. In this experimental set-up the nail passed through the base of the cell (21) via a central hole (through which the load cell shaft would pass under standard triaxial testing conditions). This would otherwise be plugged for testing without the nail. The model nail, when constructed, was fitted with a guide (23) so that it could be used for clay testing (refer component (3) Figure 5.6). This guide (23) could not be removed and so a second outer guide (22) was made to fit over it and seal the nail in the hole in the cell base (21). The two guides were securely connected together with locking screws and once in place (as shown in Figure 6.1) were anchored to the base of the cell using a locking nut (20). The nail was still free to move within the inner guide (23). The intention, for the sand tests in this apparatus, was to fix the nail rigidly to the base, i.e. prevent movement between it and the inner guide (23). This was achieved using an o-ring and clamping washer (24) with a chamfered inner edge which was screwed down into the guide, thus crushing the o-ring between the model nail and the chamfered face. This simple technique allowed the nail to be held in position very effectively.

At the upper end of the sample the boundary conditions dictated that the soil should be able to move freely past the nail. Therefore the nail had to pass through the axial stress membrane (4) and steel plate (6) keeping the pressure system sealed whilst still allowing movement of the membrane relative to the nail. This was achieved using a guide (8) which could move freely over the nail. The lower part of the guide (8) was pushed through a hole cut in the centre of the membrane (4) (there were also four small holes for the projecting screw threads), and a plate (3) placed over the end to sandwich the membrane. This plate (3) was clamped in place using four nuts, tightened sufficiently to hold and seal the membrane (4) without extruding or distorting it. Details of this arrangement are shown in Figure 6.2. The main length of the guide (8) was then slid through the hole in the plate (6), sealed with an o-ring and its movement kept vertical by an outer guide (7) which was screwed into the top of the plate. The lip of the membrane was then pulled over the edge of the plate (6) and the whole assembly bolted to the top cap (10). These final stages of equipment assembly were carried out with the components in a container of deaired water to ensure no air was trapped in the system. Once thus bolted to the top cap, the axial pressure source was an independent sealed unit.

6.2.3 Controlling boundary stresses

The general arrangement for the boundary stress control is shown in Figure 6.3.
Pressures to both the axial and radial membranes in contact with the sand were applied through standard Imperial College design 100 cc volume gauges. It was thus possible to measure axial and radial sample volume changes during the testing as well as pressure changes, monitored using standard 150 psi Bell and Howell transducers.

Outputs from the transducers were relayed through an MSL (Measurement Systems Limited) a/d converter to a minicomputer. The converted data were monitored and saved using the program TRIAX, written and developed within the Soil Mechanics Section at Imperial College by Dr. David Toll. Voltage outputs from the devices were further converted to engineering units using regressions obtained from pre-testing calibration data, (see Chapter 7 for typical calibrations and regressions). TRIAX has a versatile control capability which allows most practicable stress paths to be followed. Control was achieved by assessing the outputs relayed back to the computer, using values derived from these outputs to check adherence to the defined control equation and making the necessary adjustments to the controlling pressure sources. Adjustments to the pressure sources were achieved by sending electronic pulses to stepper motors which controlled air manostats within the control boxes. These air manostats controlled the supply to the air-water interfaces of the volume gauges.

The control equations for the sand testing were not complex, the first stage was to hold the axial and radial stresses constant at 250 kPa once the sample had been isotropically consolidated (which was usually done manually) and a shearing stage in which the radial stress was held constant at 250 kPa whilst the axial stress was decreased by 100 kPa per hour. Note that the initial isotropic stress state and the stress path followed during shearing were identical to those for the clay testing. There were two main differences in the testing of the two materials. First, the sand tests were sheared under stress control rather than strain control, this led to a lack of control in the later stages of some of the tests. Secondly, the sand was tested with air as the pore fluid which has been assumed to be at atmospheric pressure; because of this and the high porosity of the granular material, shearing could be carried out at a fast rate. The main time consideration for the shearing stage was to allow sufficient time for adequate readings and nail instrumentation response. Shearing at 100 kPa/hour and scanning every 30 seconds allowed volume readings to be taken at every 0.6 kPa change in axial stress which provided adequate coverage throughout the whole stage.

A further control stage was incorporated so that when either of the volume gauges reached the end of their travel, the program maintained the current stresses. This allowed time for the volume gauges to be reset prior to restarting shearing. The volume gauges were always reset and shearing restarted with the least delay in order to minimise creep effects which could be significant towards the end of the test, if the sample was on the verge of yielding.
During refilling of the volume gauges, stresses to the sample were temporarily isolated. Refilling was always carried out in a controlled manner using a hand pump. Volume gauge readings were taken before and after refilling and before stresses were reapplied to the sample to enable accurate resetting of the volume gauge regression coefficients to be made. The pressure supplied by the volume gauges for applying boundary stresses were checked at this point prior to reopening the valves.

6.2.4 Membrane for applying axial stress

The membrane for applying axial stress was tailor-made to fit in the space available in the top of the cell (i.e. within the confines of rings 12 and 13 in Figure 6.1). It had to be flexible enough to allow a moderate range of axial movement (at least 5 mm to give ~2.2 % axial strain) whilst maintaining almost full contact with the upper surface of the sand.

It was manufactured and tested in-house to minimise time and cost constraints. The basic material used was liquid latex (trade name Revoltex) which when dry forms a reasonably elastic impermeable material. A machined pre-former perspex mould was made to the required diameter and depth to give a possible range of 10 mm contraction from its initial shape and a similar extension, (depending on the thickness of the membrane). A small step was also machined into the top of the former to enable the sealing lip to be formed: a diagram of the former is given in Figure 6.4a.

Various methods of coating the former with the latex were tried including: dipping, pouring and painting. The best method to give a uniform thickness without bubbles was found to be by pouring and painting. The membranes initially made were found to be weak at the edges of the faces, where the latex had thinned at the rounded corners of the former. It was therefore decided to reinforce the side face with overlaps at the edges. Two materials were tried: cotton lint bandage (2 inch width) and a net curtain material! The former material tended to drastically reduce the flexibility of the membrane. The net curtain material was however found to be ideal: it was very thin and therefore easy to coat with the latex and secondly it was found to be anisotropic in its plane. Threads in one direction were elastic, those in the other were thin cotton. Placing the material with the cotton running in a circumferential sense meant that the membrane would keep its original shape but could deform considerably in the axial sense. Using the netting also meant that thinner layers of latex could be used and greater uniformity of thickness achieved (necessary when clamping the sealing plate (6) in Figure 6.1).

The steps taken in producing the final membranes used are shown in Figure 6.4b with the associated annotation describing the processes. Two such membranes were made, one for testing with a model nail and one without. They were both found to be durable and lasted throughout the testing programme without leaks.
A test was performed to check the pressure required to achieve full surface contact with the membrane extended axially ~5 mm beyond its initial shape. At ~15 kPa, approximately 95% of the surface area (i.e. ~97% of the diameter) of the contact face of the membrane was pressed against the sand.

6.2.5 Estimated resolution of measured axial and radial strain

Changes in sample dimensions were calculated using the measurements made with the volume gauges that supplied the axial and radial membranes. The resolution of the Imperial College design 100 cc capacity volume gauges is about 0.02 cc.

The strain that this volume change relates to is dependent on the membrane and sample dimensions. The sample height was typically 320 mm and the diameter of the axial membrane 162 mm. The lowest resolution of axial strain is therefore estimated to be 0.0003%.

In the case of the radial membrane, which was about 330 mm in height and about 175 mm diameter when drawn against the sides of the cell, 0.02 cc of flow into it corresponds to a radial strain of about 0.0001%

These values are estimated to be the lowest reasonable values of axial and radial strain that can be considered in the evaluation of the test results. They are based on the assumption that the sample deforms as a right-cylinder.

6.3 TESTING SAND SAMPLES

6.3.1 Establishing minimum and maximum densities

Prior to testing the sands their limiting dry densities were established using a measuring cylinder technique similar to that described by Kolbuszewski (1948a). About 500 grammes of sand were placed in a measuring cylinder and a bung placed in the top. The tube was inverted two or three times and then carefully placed on the bench and the volume of sand measured using the gradations on the cylinder. This was repeated three times to obtain an average minimum density.

The maximum density was found by tamping the sand in approximately 100 cc layers and ‘vibrating’ the cylinder by hand. The resulting density is not as great as might be obtained with a mechanical vibrator. However, the procedure provided an acceptable upper limit to the density in relation to the method of sample preparation used (use of a mechanical vibrator was precluded because of the proximity of the instrumented nail). An average value from four tests was taken.

The values recorded for the different sands are given in Chapter 8 with other measured properties.
6.3.2 Preparing the apparatus prior to each test

During the initial tests a standard thickness radial membrane was used which tended to rupture in the final stages of shearing. As a result it was often necessary to start the setting up procedure by replacing the radial membrane. This was fitted as described in Section 6.2.2 and the clamping ring tightened. A vacuum was then applied to the vent line (A in Figure 6.1) to draw the membrane hard against the inner side of the perspex with the plastic mesh sandwiched between. The purpose of the mesh was to ensure that the vacuum was applied to the whole area of the membrane and it also acted as a spacer to hold the sand slightly away from the side of the cell perspex. The valve adjacent to the cell was closed to isolate the vacuum source and the membrane left for a couple hours to check for leaks.

The space between the membrane and the cell was then filled with a thin film of water to replace the air. This was done by opening the valve to line D (Figure 6.1) and allowing flow from the volume gauge under essentially atmospheric pressure with the void still under vacuum. Usually 200 to 300 cc of water was required to almost fill the void. Filling was not continued to the upper level of the perspex as the membrane tended to start to sag slightly at the base. At this point the valve on line B was closed and the vacuum reapplied via line A sucking all excess water out (caught in a water trap) and once again causing the membrane to be pulled hard against the inner wall of the perspex. With the valve to A closed (isolating the vacuum pump) the radial pressure system was ready for the sand to be placed. The intention behind removing all air from the system and replacing it with water was so that radial volume changes could be accurately measured at the start of the isotropic compression stage, as is described in Section 6.3.4.

The radial membrane was installed without the nail in place in case of leakages. The nail, at this stage, was thoroughly cleaned of sand grains, inserted and secured in position with the locking nut (20, Figure 6.1).

The final preparations were to clean the axial pressure membrane of any loose sand from the preceding test and very lightly grease its outer edge to assist its positioning. All nuts and bolts clamping the membrane were tightened to take up creep slack and finally the hole in the guide through which the nail passed (8, Figure 6.1) was thoroughly cleaned to ensure that it was free of sand and grease which might otherwise have lead to friction where the nail was supposed to move freely.

6.3.3 Setting up the sample

After apparatus preparations, about 14 kg of dry sand was weighed so that the final mass of sand used was known. The cell was filled using a length of rigid plastic tube (dimensions: \(L = 510 \text{ mm}, \text{i.d.} = 42.0 \text{ mm}, \text{o.d.} = 48.4 \text{ mm}\)). The lower end of the tube was rested on the base
of the cell (and sand surface afterwards) and the tube filled with a scoop and funnel until full. It was then swiftly raised a short distance allowing the sand to rush out. In this way the deposition rate and energy was maximised whilst keeping the fall distance to a minimum, to produce a very loose sample, (the principle is discussed by Kolbuszewski, 1948b). Sample densities determined at the end of filling were usually comparable to those established using the measuring cylinder method. The cell was filled in layers, using this method, to just below the underside of the adapter ring (13, Figure 6.1).

When the density of the sand was to be increased, the cell was tapped with a soft hammer a set number of times around its base after placing the sand loose, to give a medium dense sample. Maximum densities were obtained by extensively tamping each layer; working inwards from the side and then tapping the base. Densities comparable to those established using the measuring cylinder, were achieved using this procedure. The effects of sand deposition on the structure of the sample are discussed in Chapter 8.

Once the sand was roughly at the required level, filling was stopped and the surface of the sand planed off. The average distance from the top of the cell to the sand surface was then determined by taking between 20 and 25 measurements over the sample area. The internal dimensions of the cell were established at the start of testing, taking averages both of the height and diameter of the cell to account for irregularities at the base and from the mesh. The diameter measurements were made with a vacuum applied as during filling. The remaining sand was weighed to give the mass of sand in the cell so that density and voids ratio could be calculated and checked with the state required.

A small disc of paper with a hole in it was ‘slid’ down the projecting length of the nail to prevent grains of sand working their way into the gap between the nail and the guide. Finally the top cap with axial pressure source was carefully lowered onto the nail and the membrane manoeuvred into the clamping ring (12, Figure 6.1). Care was required during this operation to avoid disturbing the upper surface of the sand and also to allow air to escape. The top was then bolted down.

6.3.4 Isotropic compression stage

Having filled the cell with the sand at the required density and sealed the whole system, the first step was to reset the volume gauges to the end of their travel and then set the pressures controlled by them to a low nominal value of about 20 kPa. The valve to the axial membrane was opened first, the volume required to fill it noted and then the system flushed with deaired water (usually there were no signs of air). It is assumed in determining volume changes that there is negligible axial volume change (denoted \( v_{ma} \)) in applying this small initial stress. Interpreting how much flow was required to expand the membrane to its full shape and how
much went into deforming the sample would also be difficult to quantify.

The valve into the radial membrane was then opened (with the nominal axial stress still applied) and the volume change recorded. This volume change (denoted $v_{fr}$) represents that taking place as the membrane relaxed as the vacuum holding it back was released and replaced with the small positive pressure. Once the $v_{fr}$ value was determined the membrane was flushed; usually considerable air was expelled during this operation.

The volume gauges were reset prior to isotropic compression. Increments of stress were then applied simultaneously to the axial and radial membranes at about 30 kPa per minute to the final value of $\sigma' = 250$ kPa. At this final value the control was switched on to hold the pressure constant. The sample was left overnight to allow any creeping of the sand to take place (often noted by the instrumentation output), to allow any remaining air in the radial system to dissolve or stabilise and to check for leaks in either system. Small quantities of air might have been present in the radial pressure system as a result of the filling technique and frequency of replacing the membrane. However, the effect on measured volume changes during shearing should have been negligible because the pressure in the radial stress system remained constant, so air volumes should have been essentially the same. Also the rate of air dissolving after the overnight period should have been minimal.

Overnight the radial stresses on the instrumented nail were generally found to have decreased but stabilised and a very small decrease in volume to have taken place, (typically less than 5 cc in either pressure source and usually less than 1 cc/hour during the final hours).

6.3.5 Shearing stage

Prior to shearing, the volume gauges were reset at opposite ends of travel to allow for the stress path to be followed, where axial stress was decreased under a constant radial stress thus tending to cause flow into the radial and out of the axial membranes.

A mechanical dial gauge, accurate to within 0.0001 inch was set up on the top of the inner guide (component 8, Figure 6.1) to check movements of the central part of the membrane and particularly of the guide past the nail.

Shearing was started with the control holding the radial stress constant and decreasing the axial stress by 100 kPa per hour. The shearing stage without volume gauge resets would usually take about two and a half hours. Depending on the density of the sand, one to four volume gauge resets were necessary. As explained in Section 6.2.3 there was a holding stage to allow for resetting when either volume gauge was about to reach the end of its travel. The sample was fully isolated during these resets.

In most tests shearing was stopped when the axial stress reached about 15 kPa, at which level stresses were still stable; the lower pressure limit of the manostats being 10 kPa. Most of
the final nail tests carried out with the thicker membranes had one or two reload-shear stages to see whether any further stresses could be induced in the nail. These stages involved reapplying the axial stress back to the initial value of 250 kPa and then shearing again in the same way as before. Generally stresses did not exceed those previously reached. The results from these reload-shear cycles are not presented in Chapter 8 because of the negligible influence and the uncertainty concerning sample uniformity following the first shearing stage.

6.3.6 Reference tests without a nail

Most of the preceding descriptions relate to tests with a model nail installed. A series of reference tests were also performed at similar densities without a nail to assess its benefit. The set-up and test procedure for these tests were essentially the same. Two basic alterations to the apparatus were necessary. The hole in the base of the cell, through which the nail otherwise passed, was plugged and the axial membrane replaced with one without a guide fitting and the hole in its clamping plate (component 6 in Figure 6.1) was plugged.

6.4 DATA PROCESSING

6.4.1 Initial measurements

The following quantities (relating to the sample boundaries) were measured or determined during the test set-up.

a.) After initial set-up:

- mass of sand, \( M_i \);
- initial sample height, \( H_0 = 374.1 \) - average distance from top of cell to sand;
- initial sample volume, \( V_0 = H_0(D_0)^2\pi/4 \) where \( D_0 \) was taken 175.2 mm or 175.0 mm for the thick membranes: these values being averages over the height and around the circumference of the perspex;
- initial sample bulk density, \( \rho_0 = M/V_0 \);
- initial voids ratio, \( e_0 = (V_0G/p_0)/M - 1 \), with appropriate value of \( G_i \).

b.) After isotropic compression:

- sample radial volume change during initial stress application (i.e. at \( RS = 20 \) kPa prior to flushing) = \( v_{fr} \);
- similar axial volume change, \( v_{fa} \), assumed as zero;
- sample volume changes during application of isotropic stress and creep period:

  radially, \( v_{riso} \)
  axially, \( v_{ansi} \)

c.) During the shearing stage:

- average radial stress, \( RS_{she} \).
- average axial stress, $AS_{thr}$;
- total average radial volume changes, $v_{rsFhr}$;
- total average axial volume changes, $v_{axFhr}$.

Measurements from the instrumentation on the model nail were taken at the same frequency as sample boundary parameters for all stages, (discussed in Chapters 7 and 8).

### 6.4.2 Volume corrections

The sample mass, $M$, height, $H$, volume, $V$, bulk density, $\rho$, and voids ratio, $e$, were determined after filling the cell, as described in Section 6.3.3. During initial flushing and then the isotropic compression of the sample, volume changes took place which are applied to the initial volume to establish the volume, $V$, at the start of shearing. This volume is given by

$$V_i = V_0 - v_a - v_{fan} - v_{fs} - v_{ax} - v_{rsiso}$$

Volume changes $v_{fan}$, $v_{fs}$, $v_{ax}$, and $v_{rsiso}$ are defined in the previous section and $v_a$ is the volume of the nail. $v_a$ was determined from careful measurement along the length of the nail in the sample, several readings being taken at each level to obtain average values.

The sample dimensions at the start of shearing can now be determined. At the start of shearing:

$$H_i = H_0 - \frac{(\rho_{ax} \times 1000)}{(162.2^2 \times \frac{\pi}{4})}$$

($\rho_{ax}$ multiplied by $10^3$ to convert into mm$^3$; note that internal diameter of axial membrane = 162.2 mm), and

$$R_i = \sqrt{\frac{V_i}{\pi \times H_i}}$$

Theoretically membrane corrections should have been applied in the determination of $V$, they were not for two main reasons. All of the data presented in Chapter 8 relate to tests performed using the thicker membranes, (no correction is required for the axial membrane which was about 1 mm thick). It was noted during testing (as seen through the perspex) that there was little penetration at the final stress of 250 kPa: it was certainly not discernable for the fine and medium grained sands. Secondly, the percentage of the quantity $v_{rsiso}$ attributed to this membrane correction would be very small in relation to radial volume changes between filling and shearing and the total sample volume. A final point is that the degree of membrane penetration would not change during shearing as the radial total pressure remains constant.

The sample volume $V_i$ was used to determine the voids ratio at the start of shearing,
which are used to compare sample densities between reference and nail tests in Chapter 8.

6.4.3 Strain determinations

Sample strains were calculated in the usual way with standard sign convention (for soil mechanics) where positive strains indicate a contraction and negative a dilation or increase in length dimension.

The identities used were as follows.

Volume strain:

\[ \varepsilon_p = \frac{\Delta V}{V_i} \times 100 \]

where \( \Delta V = \Delta v_{ax} + \Delta v_r \), and \( \Delta v_{ax} \) and \( \Delta v_{r} \) are the axial and radial sample volume changes during shearing. The subscript \( p \) is used for consistency with the presentation of results in Chapter 8. Axial strain:

\[ \varepsilon_a = \frac{\Delta H}{H_i} \times 100 \]

where

\[ \Delta H = \frac{\pi}{4} \times 162.2^2 \times \Delta v_{ax} \]

for the reference tests. In the case of the nail tests, the sample area has been calculated using \((162.2^2 - 14.0^2)\) to take the area of the guide that passes through the membrane into account.

Radial strain:

\[ \varepsilon_r = \frac{\Delta R}{R_i} \times 100 \]

where

\[ \Delta R = R_i - \left[ R_i^2 - \frac{\Delta v_r}{H_i^2} \right]^{\frac{1}{2}} \]

Note that \( H_i \) was used rather than \( H \) (i.e. the current height) in determining \( \Delta R \). The errors involved in this simplification are considered to be negligible, (less than 0.04 % in \( \varepsilon_r \), using typical values at an extreme axial strain of 5 %).

All strains determined are implicitly average values. The average axial strain, for example, does not take into account the restraining effect of the nail on the central portion of the sample. Similarly, the radial strain at the base of the sample, where the nail was fixed, was
probably less than that at the top where there was greater sample distortion in the vicinity of the free end of the nail. This subject is discussed further in Chapters 8 and 10 in relation to the results from the sand tests.

6.4.4 Use of spreadsheets

All raw data, in volts, were downloaded from the computer used to control and store readings from the test, and transferred to the software spreadsheet package Excel. Data were initially converted into engineering units using regressions from pre-test calibrations. Axial and radial stresses were obtained directly from transducer outputs and invariant quantities determined. Volume changes were converted to displacements and hence strains using the identities described in the previous section. The data were then processed graphically by loading the respective spreadsheets into the software package Stanford Graphics. It is these plots that are presented in Chapter 8.
CHAPTER 7

The design and development of an instrumented model soil nail

This chapter describes the instrumentation that was designed to measure axial force and radial stress on a model nail under experimental conditions. Interface shear stresses can be determined between points at which axial forces are measured, thus the progressive mobilisation of stresses along the nail boundary can be examined. Measuring nail boundary radial stress allows the influencing factors such as the angle of interface shearing resistance and effects of phenomena such as restrained dilation to be investigated.

The measurement of radial stress and especially axial force has been successfully achieved in model piles both in the field and the laboratory. The challenge for instrumentation development in the context of this present research stemmed from the size constraints dictated by the numerical parametric analyses (as discussed in Chapter 4). The nail dimensions therefore had to be no more than 8 mm diameter and 150 mm in length.

The design and development of the instrumented model nail was a key stage in the research described in this thesis. Four prototypes were developed during the course of the study, in order to avoid confusion, only the last of these is described in detail in this chapter. Details of the first three prototypes are given in Appendices 7.1 to 7.3. Prototype 4 was used for the tests carried out on sands. The clay tests were carried out with prototype 3. Some of the experimentation with the early models and the main developments made as the nail designs progressed are summarised within this chapter prior to describing the fourth prototype.

7.1 THE INSTRUMENTATION OF MODEL REINFORCING ELEMENTS

In order to develop more rational pile design approaches, a better understanding of how stresses are mobilised along the length of a pile during installation and loading is necessary. This need initiated development of pile instrumentation which started in the mid 1950s; since then considerable advances have been made in the measurement of force and stress acting on field and laboratory piles. Bond (1989) gives a comprehensive summary of such instrumented piles dating from 1955 to 1989 (his Appendix 5). Developments on the Imperial College instrumented pile were initiated by Jardine (1985) and further modifications were then carried out by Bond. The latest version of this instrumented pile can measure axial force, radial stress and shear stress at three to five points along its length.

A summary is given of some of the basic principles involved with different load or stress measurements made using some of the smaller scale piles. Details are then given of how these
principles were incorporated in the design and development of the final prototype model nail.

7.1.1 Previous experience in the measurement of axial force on a pile

The measurement of axial force in instrumented piles does not seem to have presented extensive problems. Usually strain gauges are bonded to a specially reduced section of the pile, i.e. on a thinned wall or an internal necked section and an electrical output is obtained from supplying an energisation voltage through a Wheatstone bridge circuit. The reduced section is adopted so that exaggerated strains are induced from pile loading. Consequent changes in electrical output from the gauges can then be converted to axial force using predetermined calibrations. As long as the section is able to withstand buckling there are not usually problems with such an arrangement. A highly repeatable linear output is achieved in most cases without affecting the pile behaviour in any way, (e.g. by axial deformation).

7.1.2 Previous experience in the measurement of radial stress on a pile

The three main methods are: (i) vibrating-wire devices, (ii) strain-gauged mechanisms and (iii) enclosed fluid reservoir systems.

Vibrating-wire devices

A thin supported wire is connected across a diaphragm whose outer face is in contact with the soil. The wire is vibrated by means of a d.c. plucking pulse from a receiver unit. The oscillation of the wire occurs within a magnetic field generated by an electromagnet, which induces in its coil an electrical oscillation of the same frequency. This is transmitted back to the receiver. As earth pressure deflects the diaphragm the resulting tightening or slackening of the wire changes its frequency. The frequency change is translated to units of stress using predetermined calibrations (further details are given by Dunncliffe, 1988).

These devices can have high resolution and are generally stable over long time periods. Their disadvantage, when being considered for pile use, is that the diaphragm has to be of a certain area so that the resultant earth pressure force acting on it sufficient to deflect it. At the same time deflections have to be minimised to avoid cell-action effects. This type of device was not considered further because of these limitations which could not be overcome in relation to the small scale of the model nail.

Strain-gauged mechanisms

Strain gauges are usually placed on elements of the pile which are directly strained by the earth pressure. Occasionally a diaphragm or membrane is gauged and then indirectly strained; this latter method can reduce cell-action effects.

In the same manner as for the axial devices, the electrical output of the gauges changes in response to straining of the element from externally applied stresses. This output is converted into engineering units of stress using regressions from pretest calibrations. The gauges are
usually orientated such that half the bridge is passive and the other half active thus allowing compensation for influencing factors such as bending and temperature.

The versatility and scope for contriving mechanisms to measure stress induced strains is much greater with this system of measurement than for the vibrating-wire type devices. Although strain-gauged circuits are much more prone to be affected by factors such as electrical noise and temperature, these can be minimised by careful strain gauge design and selection. Such aspects are discussed in Section 7.2.

**Enclosed fluid reservoir systems**

A thin reservoir of fluid, such as water or oil is sealed between a solid part of the pile and a thin outer membrane (e.g. as in hydraulic earth pressure cells). The fluid is connected to some form of hydraulic or pneumatic electrical transducer, for measuring its pressure, thus forming a closed system. A direct measurement of earth pressure is obtained as the soil bears on the thin membrane transmitting its pressure to the fluid.

Cell-action effects with this type of device are negligible providing the fluid is fully deaired. The system is not robust however, because the outer membrane is necessarily thin.

**7.1.3 Measuring axial force on the model nail**

Measurement of axial force proved to be comparatively straightforward with a strain-gauged thin-walled (sometimes referred to as necked) section. Various factors were considered in selecting the diameter of the necked section and its wall thickness. The wall had to be thick enough to sustain tensile and compressive loads without excessive deformations, but thin enough to show a reasonable response from the strain gauges. It is also desirable to maximise the radius of curvature of the backing surface to which the gauge is stuck, otherwise its output needs further correction. How these factors were considered for the earlier prototypes and for prototype 4 is discussed in Appendices 7.2 and 7.3 and Section 7.2 respectively. The basic methodology and calculations for checking maximum load prior to yielding and stresses at which buckling is likely to occur are given in Appendices 7.4 and 7.5 respectively.

**7.1.4 Measuring radial stress on the model nail**

Accurate measurement of radial stress is more difficult than that of axial force, but has been achieved in recent years in model piles. The more successful devices, where cell-action effects (Section 7.5) were minimal without, for example, the need for applying compensatory pressure to limit deflections, were the enclosed reservoir type devices (e.g. the piezolateral cell developed at MIT, see Baligh et al., 1985; and a device developed by Martins, 1983) and strain-gauged mechanisms, the principal one being that of Bond (1989).

Prior to the conclusion of the numerical parametric studies, when it was realised that the nail dimensions would be quite limited, efforts were made to develop an improved enclosed
reservoir device. An initial outer membrane diameter of 15 mm was used, even at this size it was difficult to de-air the thin reservoir; the problem would become more acute with decreasing diameter. Details concerning the trials with this type of device are given in Appendix 7.6.

Once the maximum diameter had been established it was decided that using a strain-gauged mechanism was the most practicable and economic method.

**Strain-gauged thin-walled shell**

The first type of strain-gauged device for measuring radial stress relied on the deformation of a thin-walled cylindrical shell simply supported internally at its ends (see Appendix 7.2, Figure A7.2.1, component 1). The idea was to mount the strain gauges on the inner surface of the shell, with the active gauges measuring circumferential strain at the quarter points of the length of the cylinder, and the passive gauges measuring axial strain as close to the midpoint as possible. Standard elastic solutions show that for a cylindrical shell subjected to an external radial stress, the resulting axial deformations at the midpoint theoretically should be zero; at the quarter points the axial deformations should be about ten times less than the circumferential (over approximately one third of the circumference: typically that covered by a strain gauge) and so gauges acting in the axial sense should be essentially passive. The formulae used, which are based on solutions from elasticity, for determining the deformations in a thin-walled shell and the calculations made are given in Appendix 7.7.

The outer surface of the shell was lubricated with very light silicon oil and then covered in a coat of grout which was in contact with the soil. Thus the average all-round radial stress was applied through the grout to the shell. The silicon oil was applied to try to avoid transmission of axial force and shear stress to the shell which could adversely affect measurements of radial stress. Further details concerning cross-effects are given in Section 7.4. In practice there were several difficulties with this design of device caused mainly by temperature and cross-effects. Devices for measuring radial stress, based on strain-gauged thin-walled shells, were used in the first prototypes described in Appendices 7.1 and 7.2. This device was modelled using numerical analyses to assess cell-action effects and the effect of the grout bonding to the thin-walled shell. The analyses are described in Section 4.6.

**Strain-gauged simply-supported segmental beam**

In this system a slender strip which is supported at its ends is positioned in a slot along part of the nail length; in section there are two faces, one flat and the other of the same curvature of the nail. This strip has strain gauges on the flat side. As radial stress is applied to the outer curved surface so the element deflects as a beam under a uniformly distributed load. This mechanism, shown in Figures 7.1 and 7.2, proved more robust and reliable than previous ones. Further details are given in Section 7.2.4. The theory, assumptions and calculations for
determining approximate deformations of the beam are given in Appendix 7.8.

7.2 DEVELOPMENT OF INSTRUMENTED MODEL SOIL NAIL PROTOTYPE 4

The model nails were designed and made as a series of components that were joined together during the final assembly, the instrumented components being those to measure either axial force or radial stress. Figures 7.1 and 7.2 show the components of prototype 4.

The devices measuring axial force used the operating principles described in Section 7.1.3. In the first two models, the necked sections for measuring axial force were located within the radial devices which were variations of the thin-walled shell described previously (refer to Appendices 7.1 and 7.2, Figures A7.1.1 and A7.2.1 for details). Forces in the nail, generated by mobilisation of shear stress on its surface, were transmitted to the inner core via shear keys at the end of each instrumented section. The advantage of this system is that axial force and corresponding radial (normal) stress are measured over the same length of nail, enabling actions at discrete points to be examined. In addition, having the two devices in parallel allows measurements to be made at more points along a fixed length of nail, e.g. each pair being 25 mm long gave six study points along a 150-mm length.

The first prototype was made from brass for ease of machining. Stainless steel was adopted thereafter for several reasons. Although stainless steel requires greater skill and care in machining, it is possible to work to a finer tolerance and achieve a better finish as it is much stiffer and more durable than brass. As it is stiffer, its deformations will be smaller for the same stress but its behaviour is more likely to be elastic. Stainless steel is also less temperature sensitive; its linear thermal coefficient of expansion is approximately half that of brass. It is easier, therefore, to obtain strain gauges with a compatible backing (this subject is discussed further in Section 7.2.1 and Appendix 7.11). The components of prototypes 2 to 4 were machined either from stainless steel of solid bar form or from cold-drawn tubing.

The method of connecting components together in the first prototype was achieved using threaded sections. There were difficulties ensuring that wires had not been trapped or twisted and that the sections did become unscrewed. The type of coupling used to connect the components was therefore changed to interference-fit push joints. The idea behind such joints is that the ends of the two components to be coupled are machined to so close a tolerance that they almost fit together but such that considerable force is needed to make the final connection, (the parts have in effect, to expand or contract by a small amount). This results in essentially a cold weld joint. If the components are accurately machined, these joints can be very strong. Prior to machining the specific components of the model nail, the strength and creep potential of several such joints were checked and found to be satisfactory; the results from these tests are
given in Appendix 7.9.

The first prototype nail made was not installed within a sample but was used to test for electrical stability, transducer response, hysteretic behaviour and creep within a chamber pressurised by air to avoid water leakage problems. Information and experience gained from the first prototype were utilised in the design of the second which was installed and tested in a 100 mm diameter sample of kaolin, this test is denoted TNAIL3 (note that the first two 'TNAIL' trial tests were used for checking the apparatus and grouting systems).

Conclusions drawn from the TNAIL3 test and the calibrations and monitoring carried out afterwards are summarised below.

(i) The stability and output from the axial force devices were good.
(ii) The radial devices (strain-gauged thin-walled shells) were adversely affected by small changes in temperature.
(iii) Stability of the devices was affected by electrical/magnetic noise even though care had been taken to place the strain gauge bridges and circuitry in a box attached directly to the nail so that wire lengths were minimised.
(iv) There seemed to be a residual strain in the radial strain device after each load-unload cycle: this is thought to have been caused either by the preformed latex film enclosing the gaps at the ends of the radial sleeves or by the grout covering the nail.
(v) There were cross-effects between the two types of device (e.g. variations in applied radial stress causing changes to the output from the axial force device under a constant applied force). These needed correction using regressions from calibrations carried out to assess these effects.

In view of the above conclusions some further, more specific, studies were carried out to improve different aspects of the instrumentation in the model nail design.

Attempts were made at numerically correcting the radial device outputs for changes in axial force, which had a significant influence (changes in axial device outputs induced by varying radial stress were negligible). The implemented corrections tended to cause a change in output from an apparent over-registration to a significant under-registration, moreover the outputs from the two radial devices in prototype 2, which should have been reading the same, diverged when the correction was applied. Temperature effects were also significant causing typically an apparent diurnal fluctuation of 50 kPa. Details of the temperature effects and monitoring carried out during the TNAIL3 test are given in Appendix 7.10.

In order to eliminate possible temperature induced differential movements between the gauges and the model nail material, rigorous consideration was given to the strain gauge selection. At the same time investigations into different strain gauge circuits and methods of reducing electrical noise were carried out in an attempt to minimise the problems encountered
with the first two prototypes. The original intention with the strain gauge selection and the experimentation was to improve the thin-walled shell radial stress measuring devices. The exercise was equally applicable to the transducers designed for the fourth prototype model nail.

7.2.1 Selection of strain gauges

When the first two prototype nails were made, the main criterion in the selection of the strain gauges was the size, especially for the radial devices on which it was much more difficult to mount strain gauges. It becomes evident from the points discussed in this section that there are many influencing factors to take into account which often have a conflicting effect on each other. Selecting the correct type of strain gauge is important when considering the following:
- optimising the gauge performance for specified environmental and operating conditions;
- obtaining accurate and reliable strain measurements;
- ease of installation;
- minimising the total cost of gauge installation.

The following summary (and many of the notes in Appendix 7.11) is based on a series of technical notes produced by Measurements Group Inc. who are strain gauge manufacturers.

The installation and operating characteristics of a strain gauge are affected by the following factors: (i) strain sensitive alloy; (ii) backing material (carrier); (iii) gauge length; (iv) gauge pattern; (v) self-temperature-compensation (s-t-c) number; and (vi) grid resistance. The gauge selection process involves determining a combination of parameters compatible with the environment and other operating conditions which best satisfies the installation and operating constraints. These constraints are generally expressed in the form of requirements such as: accuracy; stability; temperature; elongation; test duration; cyclic endurance and ease of installation. The selection of gauge characteristics for any of these constraints may have a detrimental effect on others: it is therefore very much a compromise.

The five main parameters to choose are described by the gauge designation code: e.g. MA-06-T010P-10C. The first letters MA define the gauge series; 06 the self temperature compensation (s-t-c) number; T010P the gauge length and pattern and 10C the resistance. These parameters are usually chosen in the following order for the reasons given:

i) gauge length and pattern - dependent on the space available for gauge mounting and the nature of the stress field in terms of biaxiality and expected strain gradients;

ii) gauge series - determines the foil (i.e. the strain sensitive alloy) and backing combination;

iii) s-t-c number - this is chosen to optimise temperature characteristics between gauge and substrate materials;

iv) resistance - the higher the resistance, the higher the voltage that can be applied to provide greater output without creating excessive self-heating effects.
The selection of the strain gauges is detailed in Appendix 7.11. The gauges chosen were for the radial devices type SK-09-125TB-10C and for the axial type SK-09-100TG-10C. These were fully encapsulated K-alloy gauges with solder dots for the wiring and of high 1000 \( \Omega \) resistance. Each gauge has two orthogonal grids for sensing active and passive strains. In the case of an axial device, the passive grid was orientated to measure circumferential strain. For a radial device it was placed as close to the midpoint of the radial sleeve as possible where the axial strains to be measured are minimal. In practice when installing the gauges inside the radial sleeve, they had to be placed at about the quarter-point so that the solder dots could be reached with the soldering iron. Although in theory the radial strains should still have been predominant by an order of magnitude (Appendix 7.7), it was found when the output from the passive and active grids were monitored independently (see Section 7.2.2) that both were essentially active.

The strain gauges for the axial sections were found to be far too stiff and brittle to bend around the small radius of the necked section. Encapsulating the gauges results in a much stiffer than expected composite. Although the prototype 2 design was subsequently abandoned, it was possible to incorporate the gauges in the radial stress measuring device used in the prototype 4 model.

### 7.2.2 Assessing the strain gauge circuit

The basic Wheatstone bridge circuit and gauge wiring arrangements used in the original model nail prototype 2 are shown in Figure 7.3. Note that different energisation voltages were used for the two devices: 5V for the axial devices with 1000 \( \Omega \) gauges and 1.25 V for the radial devices with 350 \( \Omega \) gauges. These different voltages were to minimise heat effects in the gauges; this subject and the question of electrical noise are discussed in Appendix 7.12. The conclusions were that the energisation voltage should not exceed 5 V and that noise effects would not be significant with adequate shielding.

A pair of the new gauges were tested to find out whether the strain gauge circuit used for the radial devices could be improved to achieve better stability and electrical output while also reducing noise and temperature effects. Details of the circuits used and monitoring carried out are given in Appendix 7.13. The following is a summary of the findings.

#### Use of amplifiers and independent circuits for monitoring gauge responses

In the first instance, an amplifier (741 Op amp) was used to increase the device outputs to a gain of 10. During this exercise the active and passive gauges were connected with high-precision resistors to form two independent full bridges so that their individual outputs could be monitored. A full bridge rather than half and quarter bridges should be implemented as it gives greater possible output and compensation against temperature and bending effects. Substantially increased output voltages were obtained from using the amplifier (approximately 250 \( \mu \)V/kPa)
but the temperature effects were equally magnified.

Independent monitoring of the active and passive gauges revealed that their outputs were essentially the same for both changes in applied radial stress and also temperature. The significance of temperature effects can be assessed by considering the readings from one period of overnight monitoring where a change of about 1.5°C was observed to cause an apparent change in output equivalent to about 150 kPa in both gauges (representing an average temperature influence of 100 kPa/°C). The gauges were 'bedded-in' by applying cycles of radial stress in an attempt to improve stability; this had little effect on their behaviour.

Some other checks were made. The passive gauges were replaced with high-sensitivity resistors to check the stability of the gauges in the original circuit, these were found to be stable. A similar operation was carried out without amplification and the outputs from the active gauges were again found to be strongly temperature dependent. Note that without the amplification the response of the active gauges dropped drastically to 0.60 µV/kPa which is significant as 1 µV/kPa realistically represents the lowest measurable unit output (i.e. the electrical resolution).

Frictional effects on the active and passive gauges

Efforts were made to minimise frictional effects induced by the end conditions of the radial sleeve and the latex membrane surrounding it. Independent monitoring of the active and passive gauges as described above showed that the output from the passive gauges decreased considerably in relation to that from the active gauges. The output remained low for the active gauges at 0.72 µV/kPa with a high average temperature influence as before (50 kPa/°C).

Noise reduction was achieved to a limited degree by tidying up all cables and connections and housing them in a box fixed to the top of the nail. However, the magnitude of the adverse effects from temperature far exceeded those from noise thus making this a secondary concern.

7.2.3 Conclusions from testing the early model nail prototypes

Despite many efforts to design and test them, the strain gauges and their associated circuits as set up on the radial sleeve had not achieved satisfactory results. The main reason for the lack of success was the end conditions of the radial sleeve and gap in that vicinity. Attempts were made to check these conditions (and factors such as cell-action effects) using a detailed finite element simulation of the prototype 2 instrumented model nail under the proposed test conditions. Results from these analyses are discussed in Chapter 4; while they did not indicate such adverse conditions, the analyses could not model the end conditions exactly or take temperature into account.

As a consequence, the radial sleeve design was abandoned. In order to advance the clay testing programme, a model nail was made which measured only axial force. It was constructed
from a single length of tube with three necked sections, using the same principle of operation applied to the axial devices that had already been tried and tested (details of the prototype 3 are given in Appendix 7.3, see also Figure 9.19).

7.2.4 The fourth prototype

The fourth prototype nail design had the same type of axial load cells as for prototype 3 but the radial device was a completely new mechanism: details of the components are given in Figures 7.1 and 7.2. The model nail is, like prototype 2, made up of a series of components joined end to end using the same type of interference-fit cold-weld joints.

The thin-walled sections of the axial devices are made to the same dimensions and of the same tubing as in the prototype 3 model with shear keys provided immediately either side.

The radial devices comprise a main section made from grade AISI 316 stainless steel rod with a flat face milled into it over most of its length as shown. The ends of this part of the device are precision machined so that they form an interference fit when forced into the axial devices. (Note that in this version, three very shallow V-grooves are cut into this shoulder to key in the Loctite when making the joint). An eccentric hole is bored through the remaining solid part of the section to carry the wires from the strain gauges. The active part of this radial device is a slender segmental ‘beam’ specially machined to leave two supporting projections at each end. This beam is made from the same stainless steel rod as the main body. Care was taken in milling it with its curved face supported to ensure that an even cut was made across its length without bending it. The flat underside of the beam is machined with a good finish to achieve high quality strain gauge bonding. Four gauges of type MA-06-T010P-10C are used on each beam with the active gauges placed in the centre as shown in Figure 7.2b.

There are several advantages to the design of this mechanism. First, the general mode of deformation is far simpler than that of the previous thin shell. The dimensions of the segment were initially chosen to suit size and practical machining constraints, but the deformations of the beam under working stresses were checked by calculation and are given in Appendix 7.8. These indicate maximum deformations of ~0.03 mm at the middle of the beam and zero at the ends when a radial stress of 250 kPa is applied (the operating cell pressure used during the test). This indicates a relatively stiff device (see notes on cell-action effects in Section 7.5 for a quantitative assessment), but with sufficient flexibility to give a reasonable output of, typically, 10 μV/kPa. The depth of the beam is sufficiently thick for it to act as a good heat sink, so the gauges are not adversely affected by temperature (when compared with the radial sleeve device). Monitoring over several days showed no deviation outside a 10μV range for the whole time and in fact the axial devices were more temperature prone.

The segmental beam rests in the slot in the main body of the device with 0.5 mm gaps
either end, so that the axial forces either during assembly or operation are minimised (there are some cross-effects which are discussed in Section 7.4.3). The fact that it is independent of the main body means that it is also possible to check the strain gauges and wiring after assembly (unlike the prototype 2).

The gap between the underside of the beam and the flat surface of the main body was specified as 1 mm to allow adequate clearance of the gauges, wiring and working deflections. This gap could probably be halved. In a refined design of this device the sides of the slot should be left intact so that there is adequate clearance for the beam to deflect but so that the remaining gap is small enough to prevent the thin covering membrane being forced into it (see Figure 7.2c). A photograph showing the components of a refined design of instrumented model soil nail is shown in Figure 7.2e. It is necessary to cover the device with a very thin membrane to prevent grout intrusion into any of the gaps. Problems were encountered with membrane intrusion into the 1 mm gap running along the sides of the beam during trials when the device was being pressure tested. These were overcome by building up the sides of the flat surface of the main body with Araldite and machining off excess material to the same diameter as the nail. The final section is shown in Figure 7.2d. Even with this measure, the membrane continued to catch in these gaps causing apparent drifts and stick-slip behaviour over random stress ranges. An attempt was made to coat the devices with grout (over a very thin membrane) but this was found to result in a drastic reduction in sensitivity and cause continuous creeping effects. The grout was therefore completely removed and experiments carried out with several reinforced membranes with the aim of minimising gap problems whilst providing a good output.

The coating adopted comprised a thin preformed membrane stretched over the device with a reinforcing layer of thin netting material, (as used for the axial membrane in the sand testing equipment, see Section 6.2.4), which was held in place by a thin coat of latex painted on it so as to cover the netting and provide a smooth outer surface. With this covering the output of the devices is stable and the response is still of the order of 10\(\mu\)V/kPa. The device behaviour is hysteretic but does always follow the same path. This subject is discussed in detail in Section 7.4.2.

The hysteretic behaviour is thought to be caused mainly by friction building up on the projecting ends of the beam which support it. The calculation of the maximum deflection of the beam at the centre is estimated using elasticity theory for a simply supported beam where deflection at the ends is zero (Appendix 7.8). Assuming the beam to be simply supported is reasonable for determining its maximum deflection. There is a tendency for rotation around the corner D because of this deflection, (as marked in Figure 7.4). As face DC tries to rotate outwards, B might tend to lose contact with the main body of the nail and corner C become a
pivot point. It is thought that in practice B never loses contact with the surface on which it rests. Considering a maximum applied pressure of 500 kPa acting on the beam, its maximum deflection would be 0.054 mm. If half of the beam is considered as a completely rigid body, such a movement would cause an uplift of 0.007 mm at B. First, this is minute, secondly, in practice the beam deflects rather than rotates (reducing the value further) and, thirdly, the surface of the beam above BC is also subjected to the applied pressure. However, the tendency for rotation about D does mean that there will be a build up of friction along BC which will inhibit the deflection of the beam during loading. When the beam is then unloaded it relaxes, but not fully as there are still shear stresses acting on the face BC which are not equivalent to those during loading because they have changed sense. This ‘locking-up’ of the frictional force on reversal of the shear stress is not an unusual phenomenon. With these devices it gradually decreases as the applied pressure, acting as a normal stress on BC, reduces. In all cases when fully unloaded, the output from the devices returned to its previous value before loading.

Although it is possible to take this hysteretic behaviour into account when converting the voltage outputs into engineering units (discussed in detail in Section 7.4.2) it would be more convenient and accurate if this effect could be eliminated. Probably the most efficient way to minimise friction would be to cut BC at an angle so that the beam rested only on the edge B.

The final nail constructed was made up of three axial components and three radial devices; a full-scale diagram of this nail is shown in Figure 7.2a. The latter were positioned at approximately 120° to each other to minimise any possible cell-action effects along the side of the nail. The axial devices were coated in grout and the radial devices with the reinforced membrane described above.

It was initially intended to perform a series of sand tests with the nail in this condition (i.e. without further grouting). In order to reduce cell-action effects with the coarser sand grain size, the radial membranes were painted with M-coat, which when dry forms a thin shellac surface, intended to minimise grain penetration into the membrane.

The instrumented nail was then calibrated, as discussed in the next section, and used to perform an extensive set of sand tests, the results from which are discussed in Chapter 8.

7.3 CALIBRATION OF THE INSTRUMENTED MODEL NAILS

In order to obtain meaningful results from voltage outputs from the devices in the model nail, it was necessary to perform extensive calibrations both before and after tests. In the case of the sand tests four calibrations were performed; before, twice during and then after the programme. Although the details of the nail (e.g. the coverings on the devices) were not changed, there were small drifts in the outputs, mainly of the axial devices and it was important
to check the overall behaviour with time.

The post-test calibrations were certainly necessary for the nails installed in clay samples as there would be minor variations in grout thickness and stiffness for each test.

In all calibrations, it was essential to check for cross-effects, e.g. to see how the application of radial stress would affect the output from the axial devices under a constant axial force. These points are discussed in Section 7.4.

7.3.1 Initial cycling

Following construction of instrumented nails comprising strain-gauged devices it is always important to cycle the gauged components to 'bed-in' the strain gauges. ‘Bedding-in’ is the process where stress or force considerably in excess of the anticipated working stress or force is applied to the devices in order to strain all components of the system, i.e. substrate, glue, backing material and grid, such that any yielding of the materials likely to occur takes place before calibration or testing. In this way working strains will always be below those corresponding to yield and hence creep and hysteresis effects will be minimised.

Usually between fifty and one hundred cycles were applied to bed-in the gauges. Loads were applied either manually using dead weights (for cycling axial devices) or by using one of the system pressure sources (e.g. cell pressure).

Cycling in axial extension needed careful control: to bed-in the gauges a load considerably in excess of anticipated working loads had to be applied but not so high as to dislocate any of the push joints. There was therefore a fine line between the two limits: care was taken to apply the dead weights without jarring the system or causing any dynamic effects.

7.3.2 General comments on calibration of instrumentation

All calibrations performed comprised a series of load-unload cycles to check for hysteresis and creep. In the case of prototype 4 which had devices for measuring both axial force and radial stress, three readings from each device were taken at each load increment. In the first two calibrations (denoted calibrations A and B) three readings were taken after one minute and then again after two minutes to assess creep effects in the transducers. The devices were found to stabilise within about 30 seconds in fact and there were negligible differences between the two sets of readings, thereafter only one set was taken after 1 minute (i.e. for calibrations C and D). A reading was also always taken of the applied pressure using a pressure transducer connected to the source.

7.3.3 Calibrating for axial force

Axial force calibrations for the prototype 3 nail were made in two ways, both of which yielded the same results. In the first method (used for the model nail prototypes 2 and 3) the calibrations were performed in the triaxial cell used for clay testing. This was convenient as it
was possible to calibrate immediately after the clay test, having removed the clay from around the nail and cleaned its grouted surface. Two specially made clamps were fixed to the grouted surface either end of the instrumented length of nail as shown in Figure 7.5. These clamps held the nail rigid in relation to the sample platens in the apparatus. The calibration was performed by unloading the ram pressure in steps whilst under a constant cell pressure (acting as a radial stress on the nail), thus increasing the tensile axial force in the nail. This was typically carried out at three levels of radial stress in the range 200 to 450 kPa to identify its influence on the measurement of axial force. The true applied axial force in the nail was measured by the load cell (component 19 in Figure 5.2).

Another similar calibration was performed for one of the prototype 3 nails: conditions were the same as described except that the nail was clamped at its ends (using the nail locking device and an external clamp) rather than on the grout. The object of this was to check whether there would be any differences in the calibration curve for the two cases (it was felt that in theory there should not be) and also there was some concern that the clamping action on the grout was deforming the tube therefore spuriously changing the outputs from the strain gauges.

The axial calibrations yielded linear outputs in all cases. There are small kinks in the calibration lines near the start of the loading (shown in Figure 7.7). These correspond to points where the triaxial apparatus load cell, measuring axial force, passes through its zero which occurs when loading sense changes from compression to tension. These kinks occur as a consequence of the design of this type of load cell. Slopes of the nail transducer calibration lines either side of the kinks were the same. The axial force devices in the model nail do not have a similar ‘zero point’, readings from a compression to tension load sense are continuous.

The calibration lines for the cases where the nail was clamped on the grout and on its main body were of the same slope but with different offsets. Following from this a different, more direct approach was used for calibrating the prototype 4 nail.

The later calibrations for axial force were performed by applying dead weights to the end of the nail as shown in Figure 7.6. The upper part of the nail projecting from the chamber was clamped in place and the dead weights were applied to a hanger connected to the lower end. Load cycles were then carried out at various applied radial stresses. Note that the O-ring at the base of the nail was well greased to minimise friction, checks to establish the force required to push the nail through this sealing ring showed it to be negligible.

7.3.4 Calibrating for radial stress

In the initial calibrations (i.e. for prototype 2), radial stress was applied by pressurising the triaxial cell with the nail set up within it. In order to minimise risks of water ingress into the instrumentation, air pressure was used. Two main disadvantages with this are: the explosion
potential of a system under air pressure (because of its compressibility compared to that of water which essentially provides a dead system); and the time required to build up pressure for the same reason. Also, because it is necessary to pump in or release large volumes of air to effect small pressure changes in a large chamber such as the triaxial cell (large compared to the size of air-water interfaces generally used in the laboratory), the stability of such a pressure source is never as good as a stiff water system.

Using the triaxial system allows any combination of radial stress and axial force to be applied to the model nail by varying the ram and cell pressures in the same way as in a conventional triaxial stress path test. Typically a calibration exercise would involve two stages: one where the nail is held at a constant axial force and the radial stress varied in steps of say 50 kPa, this would be carried out for three different values of axial load; and then conversely holding the radial stress constant and varying the axial force in say 50 N steps, this would be carried out for three different values of radial stress.

A specially made brass chamber was used to calibrate the prototype 4 nail. The volume of air required to pressurise the system was greatly reduced compared to the triaxial system and axial force was applied to the nail using dead weights (see Figure 7.6). This proved to be a great improvement on the triaxial system: air pressures could be applied safely, quickly and accurately to the devices and considerable time was saved in the radial stress cycling.

The behaviour of the radial devices (prototype 4) was not linear and was hysteretic as discussed in Section 7.2.4. The calibration sequences, data and regressions adopted are now discussed.

7.3.5 Calibrations performed on the model nail prototype 4 for sand testing

The sand testing programme (discussed in Chapters 6 and 8) was carried out as soon as the prototype 4 instrumented model nail was complete with coverings set around the devices as described in Section 7.2.4. After initial cycling and stability checks, the four calibrations were made.

Calibration A (19/2/1993)

This was a trial calibration to assess the devices for hysteresis and creep over the expected working stress and force range. It comprised five radial stress calibrations from 0 to 600 kPa with an increment of 50 kPa applied at axial loads of about 0, 20, 40, 60 and 80 lbs (exact force values with hanger: 0, −100.6, −189.7, −278.7, −367.8 N).

The calibration data were initially regressed without detailed correction for cross-effects, the regression equations input into the TRIAX program and a trial performed with the sand testing equipment. The trial being successful, a rigorous calibration was made.

Calibration B (25/2/1993)
This was the main pretesting calibration. Although carried out under six values of axial force, it was principally a radial stress calibration. Cycles were as for calibration A at six values of axial force (0, −100.6, −189.7, −278.7, −367.8, −456.9 N) but also five additional sub-cycles were performed at each loading except the last which had two. These were to assess the hysteresis of the load-unload cycles. The radial stress cycles were from zero to 600, 450, 300, 250, 200 and 150 kPa.

The results and regressions from these radial stress calibrations were those principally used in processing data from the radial stress devices in the sand testing.

*Calibration C* (12/3/1993)

This was carried out in a gap in the testing (whilst awaiting thicker cell radial stress membranes) to check for changes in the device after the testing performed and also to concentrate on the calibration of the axial devices. There was in fact more concern about the axial devices drifting.

Four cycles of axial force from 0 to 100 lbf with 10 lbf increments were applied at radial stresses of 0, 50, 150 and 250 kPa and two cycles of radial stress from 0 to 300 kPa with 50 kPa increments were applied at axial forces of 0 and −456.9 N (−100 lbf and weight of hanger).

The results and regressions from these axial force calibrations were those principally used in processing data from the axial force devices in the sand testing.

*Calibration D* (27/3/1993)

This was carried out at the end of the sand testing as a closing calibration check. The same cycles were performed as for calibration C with two additional axial force cycles at radial stresses of 100 and 200 kPa.

The results from this calibration were compared with the previous ones and were not used in the processing of results.

### 7.4 REGRESSIONS ADOPTED FOR CONVERTING INSTRUMENT OUTPUT INTO ENGINEERING UNITS

The calibration data and regressions discussed in this section mostly relate to the prototype 4 model nail, where the full scope of radial and axial cross-effects could be examined. All the sand test results are calculated using regressions from prototype 4 nail.

Calibration data and the regressions for prototype 3 nail, used in the clay tests, are included in Section 7.4.1.

#### 7.4.1 Calibration data from the axial devices

Typical sets of calibration curves for each of the three devices are shown in Figure 7.7.
These are the four load cycles from calibration C. The data over most of the range is linear and of constant slope. There is an offset between the loading and unloading lines varying from 15 to 25 N. The offset is usually constant after the first two loads have been removed in moving up the unloading cycle. In all cases the intercept values of the lines defining each cycle are greater for the unloading cycle. Also, the intercept value increases for both loading and unloading lines as the applied radial stress increases. This behaviour is shown schematically in Figure 7.8; the end-of-cycle effects (i.e. on the initial reversal) are ignored. The average slopes adopted for the three devices and the corresponding level of output are given in Table 7.1.

<table>
<thead>
<tr>
<th>Device</th>
<th>Regression slope (N/Vdc)</th>
<th>Output (μV/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AxF1</td>
<td>-126105.8</td>
<td>7.9</td>
</tr>
<tr>
<td>AxF2</td>
<td>-132219.2</td>
<td>7.6</td>
</tr>
<tr>
<td>AxF3</td>
<td>-132081.4</td>
<td>7.6</td>
</tr>
</tbody>
</table>

Table 7.1. Regressions for prototype 4 axial force devices (from Calibration C).

Similar calibration values were obtained for the prototype 3 nail using the triaxial set-up (Section 7.3.3). For the post NAIL2 calibration (i.e. with grout), the regression slopes and outputs from the devices are given in Table 7.2. Note that the output from AxF3 is different as during nail installation one of the wires was detached and therefore a quarter-bridge was formed which did not have a comparable output. Similarity in outputs from the two prototypes is to be expected as the dimensions of the necked sections, gauge types and grout covering were the same.

<table>
<thead>
<tr>
<th>Device</th>
<th>Regression slope (N/Vdc)</th>
<th>Output (μV/N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AxF1</td>
<td>126564.9</td>
<td>7.9</td>
</tr>
<tr>
<td>AxF2</td>
<td>-130195.9</td>
<td>7.7</td>
</tr>
<tr>
<td>AxF3</td>
<td>-237135.6</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Table 7.2. Regressions for prototype 3 axial force devices (carried out after test NAIL 2 with grout).
In processing the data from the prototype 3 nail, the loading cycle was used for linear regression, the chosen offset being the zero reading at the start of installation. Some of the test data were also processed using the output readings at the start of a particular stage, e.g. shearing, so that shear stresses generated due to shearing alone could be obtained.

7.4.2 Calibration data from the radial devices

Typical sets of calibration curves from calibration B for each of the three devices are shown in Figure 7.9. These radial stress calibrations were performed at six applied axial forces, with sub-cycles at each (explained in Section 7.3.5); on each plot there are data from 33 cycles. The data are well-conditioned regardless of applied axial force.

Lines representing the loading and unloading cycles diverge with increasing radial stress. When unloading from any level of a load cycle, the unloading curves are approached and regained within a stress increment of about 10% of the value from which unloading takes place (e.g. unloading from 450 kPa, the unload points plot on the well defined curve by about 400 kPa). The curves for loading and unloading roughly approximate to straight lines over certain ranges. They were modelled, however, using fourth order regressions determined using the method of least squares. Such regressions generally provided accuracies of within ±1.0 % for the ranges above 50 kPa. The outputs from the three devices varied; typical values over the central part of the range are between 7 and 10μV/kPa (devices RS2 and RS1 respectively). These variations probably result from differences in the dimensions of the segmental beam elements (measured thicknesses varied from 1.34 to 1.55 mm with an average of 1.43 mm) and, perhaps, thickness of coatings.

7.4.3 Correcting for cross-effects

Readings taken during the calibration of the axial and radial devices were used to check cross-effects.

Correcting axial force device data for changes in radial stress

Figure 7.10 shows the effect of radial stress on the output of the axial devices under a constant force (results are from calibration B). Although there is little change in outputs (not more than about 200μV, equivalent to ~25 N, for a radial stress range of 600 kPa), some trends are evident. The output increases with increasing radial stress, but at higher values (say greater than 400 kPa) this effect becomes slightly less significant. The effect becomes more noticeable when the devices are subjected to higher values of applied axial force.

These effects are noted in the discussion of the calibration results in Section 7.4.1 with Figure 7.7. The approach adopted for correcting the outputs was to model the device behaviour using a linear equation, with the slope based on the average slope of the regressions of the main
range of the cycles (i.e. ignoring data from the ends of the cycles). As implied above there are actually variations in the slope, depending on the applied radial stress, but these are not significant. The intercept values were carefully chosen by first considering the voltage output at the start of the individual test with no applied load (i.e. prior to placing the sand around the nail in the tests) and then applying an offset to this value relating to the radial stress acting at the relevant time. The offset values were based on those observed during calibration C. Consider, for example, device $AxF_1$, its response is defined by a linear relationship with slope value of $-126106 \text{ N/Vdc}$. If, at the start of the test before there were any axial force in the nail, the device output were $-0.001300 \text{ Vdc}$, the initial intercept would be set to $-164 \text{ N}$ to ensure that the equation defining its response, $AxF_1 = C_0 + C_1 (Vdc)_{AxF_1}$ was zero at this stage, ($C_0$ is the intercept and $C_1$ the slope). If the nail were subjected to a radial stress ($RS$) of 200 kPa at, say, the end of isotropic compression, the intercept value to be applied to the subsequent shearing stage initially should be offset by say $+14 \text{ N}$. This corresponds to the difference between $RS = 0 \text{ kPa}$ and $RS = 200 \text{ kPa}$ lines at an $AxF_1$ output of $-0.001300 \text{ Vdc}$ in the upper plot of Figure 7.7. The appropriate equation to use would therefore be

$$AxF_1 = -150 - 126106 (Vdc)_{AxF_1}.$$

Corrections to take cross-effects into account, as described above, were applied to the output data from the axial devices to improve the accuracy of derived values. However, values calculated without correction would not be significantly different.

**Correcting radial stress device data for changes in axial force**

The cross-effect on the radial devices from the application of axial force is shown in Figure 7.11. Cycles of axial force under four levels of radial stress for each device during calibration C are shown. Some trends in behaviour are evident, for example the cross-effect of axial force on radial stress becomes more significant as the radial stress increases, there being a greater hysteretic effect on loading and unloading.

The plots indicate that as long as applied radial stresses are less than 50 kPa and axial forces are greater than $-50 \text{ N}$ cross-effects are negligible. Also the lines representing unloading cycles, for all radial stress levels, are approximately vertical, implying no cross-effect. However, during increased application of tensile axial force there is an increase in voltage output (which stabilises at about $-150 \text{ N}$) while the radial stress actually remains constant. This offset corresponds closely to the offset between the loading and unloading curves seen in Figure 7.9. This effect is shown in the schematic representation given in Figure 7.12 and was used to correct radial stress device output data for changes in axial force.

The regressions for the loading curves (Figure 7.9) were used to convert voltages to units of force while the axial force was above $-50 \text{ N}$ or the radial stress below 50 kPa. The plots in
Figure 7.11 indicate that the loading line had stabilised once an axial force of $-150 \text{ N}$ had been reached and so in converting the data, the regressions for the unloading curves were used for converting the voltage outputs from the radial devices beyond this value. The two regressions were joined using a straight line between these two cut-off values of axial force ($-50 \text{ N}$ and $-150 \text{ N}$). This was possible because in all tests the axial force and radial stress steadily increased during shearing (when these corrections are relevant). Figure 7.13 shows the way in which the correction was implemented. In practice, point (2) was sometimes below (1) and so the first point on the unloading curve above (1) was used resulting in an almost horizontal straight line between (1) and (2).

The derived values of radial stress, so corrected, should be close to those applied to the devices, certainly within 10 kPa. The only remaining factor to be considered with the nail instrumentation was the possibility of under-registration of radial stress due to cell action effects. This factor turned out to be critical for certain soil types.

7.5 CELL-ACTION EFFECTS

Almost all devices which measure any form of force or stress require some finite deflection to activate the measuring system. For example, when a pressure is applied to an enclosed reservoir device (which is probably intrinsically a stiff device), the fluid might compress slightly and the diaphragm of the transducer measuring the pressure changes also deforms. Any air in the system will also compress. These deformations and volume changes have the effect of causing a deflection of the face at which the pressure is applied. In the strain-gauged segmental beam for measuring radial stress in the prototype 4 nail, the beam must deflect by some finite amount in order for the strain gauges to register a response.

Such deflections of the face against which the pressure acts are not important if the pressure comes from a continuous source, e.g. the air supply when calibrating the instrumented nail (Section 7.3.4 and Figure 7.6). However, in the case of soil, even minute deflections are significant. They cause the soil to deform with a consequent decrease in the stress in the soil acting against the face. The effect is caused principally by arching resulting from a stress rotation from the direction perpendicular to the instrument face to a direction parallel to it. For the segmental beams on the model nail, they deflect inwards as the radial stress activates them causing the soil to arch around them; thus a proportion of the radial stress is transmitted into a hoop stress circumferentially around the nail.

These deflections lead to an under-registration of the true applied stress (i.e. the radial stress in the case of the model nail); such effects are known as cell-action effects. Their magnitude depends on several factors. In designing instrumentation for measuring soil stresses,
the devices should be as stiff as possible to minimise deflections. At the same time the device should be able to deflect sufficiently to give reasonable output (i.e. its sensitivity should be maximised). Ideally the optimum device stiffness is therefore such that cell-action effects are minimised whilst sensitivity is maximised. The other main factor to consider is the soil stiffness: the stiffer the soil, the more significant is any given deflection and the greater the under-registration of applied stress.

Determining the magnitude of cell-action effects is not straightforward, even for regularly shaped earth pressure cells. There are further complications with the radial stress measuring device used in the prototype 4 nail. These can be summarised as follows.
i) The device has a curved outer face of small radius.
ii) The segmental beam is small and not of uniform shape, therefore the exact way in which it deforms under an applied stress is not known. Its deformation is estimated by assuming that it acts as a beam of regular section with simply supported ends.
iii) The whole device is covered with a reinforced latex membrane. This should not have adverse effects because it is essentially an incompressible material and so should transmit all-round normal stresses applied to it directly through to the measuring element. However, in practice, there are sometimes small air bubbles trapped within the latex during manufacture.
iv) The small but finite gaps around the element, though bridged by the reinforced membrane, may cause non-uniformities of stress in its vicinity.

In addition to these factors, there are several points to consider regarding the soil being tested, cell-action effects being strongly dependent on the nature of the soil. For the sand tests the following points were considered.
i) Soil stiffness significantly influences the magnitude of cell-action effects; stiffness is not easy to define as a specific value as it varies with sand type, density, level of strain and level of stress.
ii) Poisson's ratio is also an influencing factor but to a much lesser degree than soil stiffness.
iii) The grain size is significant; the likelihood of obtaining a uniform stress condition over the measuring device decreases with increasing grain size.
iv) Soils with larger grain size have a greater propensity to arch. This behaviour is directly related to cell-action effects.
v) There may be problems with grain penetration into the reinforced membrane with larger particle sizes.

In order to obtain some idea of the magnitude of cell-action effects two approaches have been used; both involve elastic solutions with various assumptions. These methods are now discussed.
7.5.1 Approximate estimation of cell-action effects - Method 1

In this approach the elastic solutions for stresses and deformations in a long thick-walled cylindrical pipe are used (from Roark and Young, 1975 p.504). Expressions are given for the radial deformations at different radii, resulting from different boundary stresses, e.g. an applied external stress acting on the outer surface of the cylinder and an internal stress acting on the inner surface of the hole running through the centre of the cylinder.

The approach used was to calculate the radial deformation of a boundary which is taken to correspond to the interface between the model nail and the soil for the three different conditions (shown in Figure 7.14). The expressions used and the calculation of these deformations are given in Appendix 7.14. The conditions considered are as follows.

1) The cylinder is assumed to represent the soil sample with the nail hole running down its centre but without the nail. An external stress (corresponding to the radial boundary stress) is applied, denoted $\Delta \sigma_{r,op}$. It is relevant to use soil properties in the calculations for this case.

$\Delta b_1$ is the resulting inward deformation of the internal boundary of the soil from $\Delta \sigma_{r,op}$.

2) The cylinder is assumed to represent the nail with its central hollow core (2 mm radius). An external stress, denoted $\Delta \sigma_{r,nai}$, is applied to the nail. Its magnitude is unknown but represents the stress transmitted to the nail as consequence of the applied boundary stress detailed in (1): note that radial stress is generated because of the finite stiffness of the nail, if the nail stiffness were zero then radial stresses at this boundary would also be zero. Steel properties are relevant for this case.

$\Delta b_2$ is the resulting inward deformation of the external boundary of the nail from $\Delta \sigma_{r,nai}$.

3) The cylinder is again assumed to represent the soil sample with the hole but no nail (as for (1)). An internal stress of magnitude $\Delta \sigma_{r,nai}$ is applied to the inner boundary. Soil properties are relevant for input to the calculations for this case.

$\Delta b_3$ is the resulting outward deformation of the internal boundary of the soil from $\Delta \sigma_{r,nai}$.

The deformation of the same boundary is to be considered for the case where one of the radial segmental beam devices deflects under a stress applied directly to the device, denoted $\Delta \sigma_{r,ins}$, which corresponds to the stress measured by the instrument, reduced from $\Delta \sigma_{r,nai}$ because of cell action effects. This deformation is calculated using the elastic beam theory given in Appendix 7.8.

$\Delta b_4$ is the resulting inward deformation of the external boundary of the instrumented part of the nail from $\Delta \sigma_{r,ins}$.

From the calculations given in Appendix 7.14, we have:

i) $\Delta b_1$ expressed in terms of $\Delta \sigma_{r,op}$ and $E_s$ (soil stiffness),

ii) $\Delta b_2$ expressed in terms of $\Delta \sigma_{r,nai}$. 
iii) $\Delta b_j$ expressed in terms of $\Delta \sigma_r^{\text{nail}}$ and $E_s$.

iv) $\Delta b_j$ expressed in terms of $\Delta \sigma_r^{\text{inst}}$.

The soil stiffness ($E_s$) is kept as a variable at this stage in order to assess its influence on the approximated cell-action effects.

It is now possible to combine these deformations using superposition to obtain relationships for $\Delta \sigma_r^{\text{nail}}$ and $\Delta \sigma_r^{\text{inst}}$ and hence determine the approximate cell-action effects. By a three part superposition:

$$\Delta b_j - \Delta b_j = \Delta b_j$$

for the uninstrumented section of the nail (Figure 7.15a).

For the instrumented section, similarly:

$$\Delta b_j - \Delta b_j' = \Delta b_j'$$

where $\Delta b_j'$ is a deformation as for $\Delta b_j$ but induced by $\Delta \sigma_r^{\text{inst}}$ rather than by $\Delta \sigma_r^{\text{nail}}$. This concept is shown in Figure 7.15b.

The error in measurements resulting from cell-action effects is given by:

$$\epsilon = \frac{\Delta \sigma_r^{\text{inst}} - \Delta \sigma_r^{\text{nail}}}{\Delta \sigma_r^{\text{nail}}}$$

Values of $\Delta \sigma_r^{\text{nail}}$, $\Delta \sigma_r^{\text{inst}}$ and $\epsilon$ are given in Table 7.3 for an applied external pressure $\Delta \sigma_r^{\text{opp}} = 250$ kPa and for three values of $E_s$ at 300, 30 and 3 MPa.

<table>
<thead>
<tr>
<th>$E_s$ (MPa)</th>
<th>300</th>
<th>30</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta \sigma_r^{\text{nail}}$ (kPa)</td>
<td>332.7</td>
<td>333.0</td>
<td>333.0</td>
</tr>
<tr>
<td>$\Delta \sigma_r^{\text{inst}}$ (kPa)</td>
<td>57.5</td>
<td>225.2</td>
<td>317.8</td>
</tr>
<tr>
<td>$\epsilon$ (%)</td>
<td>-82.7</td>
<td>-32.4</td>
<td>-4.6</td>
</tr>
</tbody>
</table>

Table 7.3. Summary of approximated cell-action effects using Method 1.

7.5.2 Approximate estimation of cell-action effects - Method 2

In this approach superposition of deformations is again used, but this time by a two-part comparison. First, the maximum deflection, $\delta_{\text{max}}$, of the centre point of the segmental beam is determined using elastic beam theory (as for $\Delta b_j$ in Method 1 and as detailed in Appendix 7.8); its magnitude is governed by the measured stress, $\Delta \sigma_r^{\text{inst}}$.

The segmental beam is then considered in another way as acting as a flat, rectangular,
flexible plate bearing on a semi-infinite soil. The deformation of the centre of the plate can be determined using an elastic solution (given by Boussinesq), and is denoted $\delta_{\text{plate}}$. The deformation of the plate is dependent on the applied stress, the soil stiffness, $E$, and its Poisson's ratio, $\nu$. Essentially the idea in the superposition is to match the deformation of the beam resulting from the $\Delta \sigma_{r}^{\text{inst}}$ applied stress (i.e. the instrument compliance) with the deformation of the plate under the relevant applied stress which is the difference between the applied stress on the nail ($\Delta \sigma_{r}^{\text{nail}}$) and that measured by the instrument ($\Delta \sigma_{r}^{\text{inst}}$). This can be thought of as the soil moving into the displaced shape of the beam, modelled here as a plate; the stress that causes that movement (or deformation) is the difference between the true stress, i.e. $\Delta \sigma_{r}^{\text{true}}$ and the stress acting on the device after the cell-action effects, i.e. the measured stress, $\Delta \sigma_{r}^{\text{inst}}$.

The elastic solutions used and the calculations are given in Appendix 7.14. From these calculations we have:

i) $\delta_{\text{inst}}$ expressed in terms of $\Delta \sigma_{r}^{\text{inst}}$,

ii) $\delta_{\text{plate}}$ expressed in terms of ($\Delta \sigma_{r}^{\text{nail}} - \Delta \sigma_{r}^{\text{inst}}$) and $E$.

By superposition we can set

$$\delta_{\text{inst}} = \delta_{\text{plate}}$$

to obtain a relationship between $\Delta \sigma_{r}^{\text{nail}}$ and $\Delta \sigma_{r}^{\text{inst}}$. The error in measurements, $\varepsilon$, can be approximated by the expression:

$$\varepsilon = \frac{\Delta \sigma_{r}^{\text{inst}} - \Delta \sigma_{r}^{\text{nail}}}{\Delta \sigma_{r}^{\text{inst}}}$$

if the numerator is small. However this is not so in this case, and it is necessary to estimate a value of $\Delta \sigma_{r}^{\text{nail}}$ so that $\varepsilon$ can be calculated as accurately as possible. It is logical to use the $\Delta \sigma_{r}^{\text{nail}}$ values determined from Method 1 for this purpose. This should not significantly influence the results from this approach and will also give a common basis for comparing the results with those from Method 1.

Again values of $\Delta \sigma_{r}^{\text{nail}}$, $\Delta \sigma_{r}^{\text{inst}}$ and $\varepsilon$ for an applied external pressure, $\Delta \sigma_{r}^{\text{app}} = 250$ kPa and for three values of soil stiffness are given, see Table 7.4.

7.5.3 Discussion of factors influencing approximated cell-action effects

In the previous section two approaches were used to calculate the cell-action effects. The assumptions made in each of them are now discussed.

Both methods rely on solutions derived from elasticity. This assumption is acceptable for the steel components but, in practice, soils are generally elasto-plastic and the elastic behaviour is highly nonlinear. The elasticity theory assumes the materials to be isotropic; in practice all soils, especially sands, have some degree of anisotropy. Both methods assume the
materials to be infinite in extent. In the case of Method 1, a plane section of an infinitely long cylinder is considered, which is not realistic as the samples have a finite length and probably significant end restraints. In Method 2, the soil is assumed to be of semi-infinite extent, this may just be reasonable considering the size of the device compared with that of the sample.

<table>
<thead>
<tr>
<th>E (MPa)</th>
<th>300</th>
<th>30</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δσ_r^naïl (kPa)</td>
<td>332.7</td>
<td>333.0</td>
<td>333.0</td>
</tr>
<tr>
<td>Δσ_r^inst (kPa)</td>
<td>75.8</td>
<td>248.7</td>
<td>322.1</td>
</tr>
<tr>
<td>ε (%)</td>
<td>-77.2</td>
<td>-25.3</td>
<td>-3.3</td>
</tr>
</tbody>
</table>

Table 7.4. Summary of approximated cell-action effects using Method 2

In Method 1, axi-symmetric conditions are assumed. It can therefore be realistically assumed that values of Δb_1, Δb_2, and Δb_3 for calculating Δσ_r^naïl are reasonable. However, the area of the segmental beam only covers about one quarter of the nail perimeter and so Δb_3' values (based on a uniform all-round stress) and their influence on estimating Δσ_r^inst are questionable. In the same method, end effects experienced in practice would inhibit sample movements and so Δb_1 and Δb_3 (or Δb_3') values would be lower, therefore calculated Δσ_r^naïl and Δσ_r^inst values would tend to be overestimated. As it is the difference of these values that is used to calculate cell-action effects, their inaccuracies should not have significant influence.

In Method 2, the superposition used to estimate a relation between Δσ_r^naïl and Δσ_r^inst relies on the assumption that the outer surface area of the segmental beam acts on the soil as a flat, flexible rectangular plate. The assumption regarding flexibility is probably reasonable, but the mode of deformation of a plate and the segmental beam are not directly comparable.

The methods provide initial approximations of the magnitudes of possible cell-action effects for a range of soil stiffnesses. Discussing the assumptions made in each approach is useful in gaining insight into their influence on the final calculated values. However, differences between the two methods might be such that effects from the assumptions are insignificant: they are certainly not readily quantifiable.

The results from the two methods show similar trends with soil stiffness. It is probably reasonable to assume that the range of approximated cell-action effects between the two approaches is fairly representative of those likely to be encountered in practice.
7.5.4 Discussions of influence of soil properties on cell-action effects

The three moduli values used, $E = (i)\ 300$, (ii) $30$ and (iii) $3$ MPa, correspond approximately to:

(i) the maximum stiffness of a dense sand determined using dynamic tests at very low strains,
(ii) a reasonable estimate of dense sand stiffness at a strain of about $0.1\%$,
(iii) a very loose deposit or soft clay stiffness.

Note that an upper value of Poisson’s ratio for the soil $\nu$ of $0.5$ was used in both methods to give a maximum influence on $\epsilon$.

Considering the $\epsilon$ values given in Tables 7.3 and 7.4 it can be seen that there are significant differences in $\epsilon$ at the very stiff end of the $E$, range between the two methods (considering the effect of their magnitude) but that the values converge as $E$ decreases. These approaches indicate that in a very stiff soil or in the initial stages of shearing the devices will seriously under-register radial stress in a range of 77.2 to 82.7 % (i.e. by 4.4 to 5.6 times).

These values are very significant. However the $E = 30$ MPa is probably far more realistic for the soils tested: the under-registration is still significant being in the range of 25.3 to 32.4 % (i.e. by 1.3 to 1.5 times). At the very low end of the $E$, range the under-registration is negligible but a stiffness as low as this is not relevant to this study.

In addition to the progressive under-registration that occurs as soil stiffness increases, there is a similar effect with increasing grain size because of arching and grain penetration into the membranes covering the devices. There are two arching conditions to consider: that of the main body of the soil around the nail (inclusion effect) and also the small localised arching that can occur around a pressure cell as it deflects. Larger grain sizes are also more prone to penetration of the reinforced membrane surrounding the radial devices in the prototype 4 nail. Although the membrane material is incompressible and has a shellac coat, as the size of the interstices increases so also will the tendency for the membrane to be forced into them as the sand grains bear on it. This will lead to under-registration and premature arching effects.

7.5.5 Conclusions regarding cell-action effects

It is evident that cell-action effects are difficult to quantify; they are influenced by many factors some of which are hard to measure and vary, for example soil stiffness which varies significantly with strain level, stress level and density. Depending on the soil stiffness the cell-action effects can range from complete under-registration (where insignificant applied stress is measured) to almost true readings of applied stress.

Determining cell-action effects for the radial devices of the prototype 4 nail was further complicated by their unusual shape. The instrument deformations that have been used to calculate cell-actions are based on approximations using elastic beam theory.
The soils tested in this study comprise clay and fine, medium and coarse sand. Typically stiffness values of these soils at about 0.1% strain are in the range of 20 to 40 MPa.

Using the approximate methods described in Section 7.5.1, cell-action effects have been estimated to be about 25% (i.e. radial devices will read about 75% of the stress applied to the solid parts of the nail and so measured radial stress values should be factored by ~1.3). The ε value will probably increase for the coarse grained sand and also for the dense sands: it will probably be much smaller for the clay (for which the devices were originally intended).
But now, it is possible to study in the further interiors of the great deserts the free interplay of wind and sand, uncomplicated by the effects of moisture, vegetation, or of fauna, and to observe the results of that interplay extended over great periods of time.

Here, instead of finding chaos and disorder, the observer never fails to be amazed at a simplicity of form, an exactitude of repetition and a geometric order unknown in nature on a scale larger than that of crystalline structure. In places vast accumulations of sand weighing millions of tons move inexorably, in regular formation, over the surface of the country, growing, retaining their shape, even breeding in a manner which, by its grotesque imitation of life, is vaguely disturbing to an imaginative mind. Elsewhere the dunes are cut to another pattern - lined up in parallel ridges, peak following peak in regular succession like the teeth of a monstrous saw for scores, even hundreds of miles, without a break and without a change of direction, over a landscape so flat that their formation cannot be influenced by any local geographical features. Or again we find smaller forms, rare among the coastal sand hills, consisting of rows of coarse grained ridges even more regular than the dunes. Over large areas of accumulated sand the loose, dry, uncemented grains are so firmly packed that a loaded lorry driven across the surface makes tracks less than an inch in depth. Then, without the slightest visual indication of a change, the substance only a few inches ahead is found to be a dry quicksand through which no vehicle can force its way. At times, especially on a still evening after a windy day, the dunes emit, suddenly, spontaneously, and for many minutes, a low-pitched sound so penetrating that normal speech can be heard only with difficulty.

These are some of the phenomena of desert dunes, and no satisfactory explanation of any one of them has so far been forthcoming.
CHAPTER 8

Sand test results

8.1 INTRODUCTION

In this chapter the results from a series of tests on three sands representing fine, medium and coarse grain sizes are presented. The testing of each sand was carried out at three densities to investigate relations between grain size and loose, medium dense and dense states.

The basic characterisation and properties of the sands are given first by considering the origin of the materials, grading curves and specific gravities. Relative density is used to express the sample state quantitatively: determining minimum and maximum densities is covered.

The results from a series of direct shear box tests performed on the fine and medium grained sands are presented. Interface tests provide information on the mobilisation of the angle of interface shearing resistance and conventional tests, comparative sand to sand behaviour. The results also give insight into the volumetric changes at different degrees of shearing under varying normal loads. However, in view of the limitations of the direct shearbox, discussion of the results are kept brief and simple.

Chapter 6 describes the apparatus and procedures used for the element tests performed to simulate conditions on a single nail in a nailed-soil excavation. To gauge the effectiveness of the reinforcement, complementary reference tests were carried out for comparison. The sands were deposited, densified where necessary and tested in a dry state. Testing was under axi-symmetric triaxial conditions with an isotropic compression stage followed by shearing in axial extension to simulate stress relief due to excavation. The range of particle sizes and states are covered by eighteen tests.

Presentation and discussion of test results is first dealt with in terms of boundary stresses and strains. In order to compare results between reference and nail tests, stress and strain 'invariants' are used in the same way as they are conventionally for triaxial element tests. The limitations of assuming uniform stresses and strains are discussed in the context of the boundary conditions before assessing the test results. The idea of expressing the benefit of the nail using work is explained and applied.

Conditions around the model nail are then considered using the output from the nail instrumentation. To help with the interpretation of the nail interface behaviour, idealised models were formulated to simulate test conditions, these are described before discussing the nail data in detail. Measurements from the instrumentation are presented using three approaches and
discussed.

In the final section an appraisal is made of the overall sample behaviour taking into account grain size, density, interface behaviour and mechanisms such as rigid-body inclusion effects, arching and restrained dilation. This sets the scene for the discussion of the test results in relation to their application to field conditions and design, the subject of chapter 10.

8.2 BASIC PROPERTIES OF SANDS TESTED

The three sands tested were of fine, medium and coarse grain size of sub-rounded, sub-rounded to sub-angular and sub-angular shape respectively. Their grading curves, obtained using a dry sieving procedure in accordance with Clause 9 of BS1377:Part 2:1990, are shown in Figure 8.1a. Photographs of the three grain types at a magnification of ten are presented in Figure 8.1b. Values of average grain size ($D_{50}$) are given in Table 8.1.

<table>
<thead>
<tr>
<th>sand type</th>
<th>fine</th>
<th>HRS</th>
<th>coarse</th>
</tr>
</thead>
<tbody>
<tr>
<td>average particle size, $D_{50}$ (mm)</td>
<td>0.10</td>
<td>0.28</td>
<td>1.40</td>
</tr>
<tr>
<td>specific gravity</td>
<td>2.654</td>
<td>2.672</td>
<td>2.654</td>
</tr>
<tr>
<td>grading ratio</td>
<td>100/170</td>
<td>-</td>
<td>7/14</td>
</tr>
<tr>
<td>$\rho_{\text{max}}$ (t/m$^3$)</td>
<td>1.589*</td>
<td>1.695*</td>
<td>1.707</td>
</tr>
<tr>
<td>$\rho_{\text{min}}$ (t/m$^3$)</td>
<td>1.329</td>
<td>1.443</td>
<td>1.449</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>0.670</td>
<td>0.576</td>
<td>0.555</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>0.997</td>
<td>0.852</td>
<td>0.832</td>
</tr>
</tbody>
</table>

*Maximum densities determined from shearbox testing, prior to applying any normal load.

Table 8.1. Basic characteristics of sands tested.

The specific gravities of the sands were determined using a water displacement method (with small pyknometers) as described in Clause 8 of BS1377:Part 2:1990. The fine and coarse grained sands both have values of 2.65 and the medium sand 2.67. These values reflect that all three sands are predominantly formed of quartz grains (quartz having a specific gravity of 2.66).

The fine and coarse sands are graded from the same deposit of Cretaceous Greensand and are described by the suppliers to be predominantly of silica. The medium grained sand is the new Ham River sand (HRS), chosen by the Soils Section at Imperial College to match the grading of the original stock material which is now depleted. The original material was tested extensively within the Soils Section: further information on its characterisation by triaxial testing
can be found in Daramola (1978) and Ovando-Shelley (1986). In order to clarify future discussion, the medium grained sand is always referred to as ‘HRS’. This is to avoid duplication of ‘medium’ which is also used in the context of an intermediate ‘medium dense’ state.

Minimum and maximum densities were obtained using a measuring cylinder technique similar to that described by Kolbuszewski (1948a). About 500 grammes of sand was placed in a 1000 cc measuring cylinder and a bung placed in the top. The tube was inverted two or three times and the volume of sand measured using the graduations on the cylinder. This was repeated three times to obtain an average minimum density. The maximum density was obtained by vibrating and tapping the cylinder by hand to replicate conditions in the test set up. In fact, greater densities were obtained in setting up the shearbox tests and these are the values presented in Table 8.1.

The minimum and maximum values of voids ratio given in Table 8.1, required for calculation of relative density, have been determined using the appropriate specific gravity.

### 8.2.1 Direct shearbox testing

A series of direct shearbox tests were performed on the fine sand and HRS to establish both their angles of internal and interface shearing resistance. The shearbox tests can also provide information on the volumetric behaviour of the sands, in particular the magnitude of the dilation. The author is indebted to Tim Connolly for kindly offering to do the tests and for performing them so carefully.

The tests were carried out in accordance with clause 4 of BS 1377:Part 7:1990. The sands were placed in loose, medium dense and dense states to match conditions in the nail tests carried out in the main testing programme. Interface and corresponding reference tests were undertaken. The tests were performed under three normal stresses of 100, 250 and 400 kPa to cover a range of values measured by the radial stress instrumentation on the model nail during the triaxial testing. The rate of shearing was on average about 0.5 mm/min. The initial testing on the loose sands was at a slower speed, about 0.1 mm/min. Comparative tests were performed on the loose fine sand at the upper normal stress loads which indicated negligible difference in results (see Figure 8.3a: tests A and B). For this reason the faster rate was adopted for the majority of tests (and is in conformance with BS1377:Part 7:1990).

It is known from sand testing using the triaxial apparatus (e.g. Hellings, 1989; Ponce and Bell, 1971) that sand, when sheared at low stress levels (e.g. < 50 kPa) has an increased angle of internal shearing resistance. Shearing in the nail tests took place under steadily decreasing stress levels (as sheared in axial extension) and so it would have been useful to examine conditions at low stresses. However, Kuwano (1996) has shown that results from shearbox tests carried out under low normal stress are not reliable because of the boundary conditions of the
test. Principally, side friction causes under- and over-estimates of strength with contractant and dilatant materials respectively. For this reason the lowest normal stress used in the shear box tests was set at 100 kPa.

Research into the effects of the boundary conditions of the shearbox test (e.g. Shibuya et al. 1993) has shown that the gap dimension between the two halves of the box has a significant influence on the results obtained from the test. Increased strength and dilatancy effects are obtained with decreasing gap thickness, but with larger gaps there may be a reduction in strength from the loss of sand. It has been estimated that the shear zone in shearbox testing is about twenty times that of the average grain size and that ideally a gap of similar dimension to the shear zone should be allowed for. In practice, realistic results are obtained as long as the gap is four to five times the average grain size of the sand being tested. A conventional shearbox was used for the testing in which the gap was 0.5 to 1.0 mm. Thus gap requirements for the fine sand and HRS should have been satisfied.

Shearbox tests were not performed on the coarse sand partly for this reason as a gap of at least 5 mm would have been required. It was also felt that it would not be possible to obtain conditions of uniform density for the coarse grain size within the box of 60 mm side dimension. BS 1377:Part 7:1990 also recommends that the size of largest grain size should not exceed one-tenth of the sample height.

8.2.2 Work concepts applied to the direct shear box test

The philosophy of using expressions for work to analyse and interpret conditions within a boundary value problem are introduced at this stage for the shearbox test. The work concept involves the principle of conservation of energy whereby external and internal work are considered to balance each other. The idea of using work as a variable for interpreting data is adopted later as a means of expressing the benefit of a nail in reducing strains. It is useful to introduce the subject here with the simplified (idealised) conditions of the shearbox apparatus.

Taylor (1948) is the reference that is generally quoted when considering work in the context of the shearbox apparatus. Wood (1990) also covers the subject comprehensively; the following comprises a summary of the salient points.

The work concept applied to the shearbox apparatus was originally used to interpret the effects of dilation during shearing of dense sands. A saw-tooth analogy of a series of inclined planes with rough surfaces is used to model the shearing zone within the sample, the teeth having an incline of $\psi$, the angle of dilation, as shown in Figure 8.2a. It is assumed that sliding takes place on the inclined planes and that the angle of friction resisting sliding is $\phi'_{cr}$. An apparent externally mobilised angle of friction, $\phi'_m$ acting on a horizontal plane is then introduced, this being the plane of shearing and that in which measurements are made. It can
be seen (Figure 8.2a) from the orientation of the resultant force acting on the rough surface, in relation to the inclined and horizontal planes that:

\[ \phi'_m = \phi'_{ca} + \psi \]  

(8.1)

The forces and displacements within the apparatus are considered, as shown in Figure 8.2b, with normal and shear loads \( P \) and \( Q \) respectively and corresponding displacements \( y \) and \( x \). The work done by the external loads in inducing incremental displacements \( \delta y \) and \( \delta x \) is given by:

\[ \delta W = P \delta y + Q \delta x \]  

(8.2)

The \( Q \delta x \) component represents the work done in shearing the sample and \( P \delta y \) that resulting from sample volume changes acting in combination with the normal load. In the case of a dense sand being sheared, when dilation occurs, the displacement \( \delta y \) is upwards working against the normal load \( P \). This effect occurs because in the dense sample the interlocking between particles is so tight that during shearing apart from sliding and rolling against each other, the only way deformations can occur is if some of the particles can ride up on others to break the interlocking.

Thus the \( Q \delta x \) component of work does not all go into shearing the sample, some of it is expended in moving the normal load. This quantity is represented by the \( P \delta y \) component which is negative for a dilating sand. Most of the nett work going into the sample, represented by \( \delta W \) in equation 8.2 is expended in overcoming friction in the sliding and rolling between particles. Part of it, albeit small, goes into elastically deforming the particles and even smaller quantities into generating heat from frictional movement.

Taylor (1948) assumes that all the nett work goes into overcoming friction and then considers that the internal dissipation of energy through friction is controlled by the normal load and a frictional constant, \( \mu \).

\[ \delta W = \mu \ P \delta x \]  

(8.3)

The two expressions for external and internal work can then be equated to give:

\[ \frac{Q}{P} + \frac{\delta y}{\delta x} = \mu \]  

(8.4)

The \( Q/P \) term is equivalent to \( \tan \phi'_m \) i.e. the externally mobilised friction on a horizontal plane (\( \phi'_m \) being the angle of internal shearing resistance usually determined from the shearbox test). The \( \delta y/\delta x \) term represents the dilation of the sample and is equivalent to \(-\tan\psi\) (the negative sign introduced to give positive values of \( \psi \) for dilation). Equation 8.4 can therefore be written:
\[ \tan \phi'_m = \mu + \tan \psi \]  

(8.5)

This implies that the angle of shearing resistance usually determined from a shearbox test is a combination of angles of friction and dilation. Similarities between equation 8.1 and 8.5 imply a relation between \( \phi'_c \) and \( \mu \); Wood (1991) does not try to relate them because of the various assumptions involved.

In the following sections where the results from the shearbox tests on fine sand and HRS are presented, some of the expressions and relations described above are used and further discussed. The concept of using work as a means of interpreting test data is developed further in a later section and applied to the triaxial test results.

8.2.3 Results from the reference shear box tests

The data from the shearbox testing are summarised using five graphs for the sets of tests performed on each sand. In Figure 8.3a \( \Delta H/H \) is plotted against \( \Delta L/L \) using basic measured readings from the tests. \( H \) is the sample height and \( L \) its length. \( \Delta H \) is change in sample height and \( \Delta L \) the displacement of the lower half of the shear box. The graph gives information on volumetric changes: positive values contractant, negative dilatant. On each of the graphs there are nine curves representing the three states (loose etc.) at the three normal loads used. The quantity \( \Delta L/L \) is always taken for the abscissa as it effectively represents the progress of shearing, the absolute displacement \( \Delta L \) is given on the upper part of the graph.

Changing voids ratio \( e \) with displacement \( (\Delta L/L) \) is plotted in Figure 8.3b. The voids ratio calculated is an average value determined from the sample volume, mass and specific gravity \( (e = G_r \rho_v/\rho_v - 1) \). Local changes in void ratio in the sheared zone are not known but the graph is useful for assessing initial conditions and comparing behaviour during shearing of the samples at the same density but different normal loads. The average initial relative density of the samples at each state is also marked.

The remaining three graphs in Figure 8.3 illustrate quantities discussed in the previous section. In Figure 8.3c what shall be referred to as \( \phi'_m \) versus \( \Delta L/L \) is given, \( \phi'_m \) being set equal to \( \tan^{-1}(\pi/\sigma_n) \) which is equivalent to \( \tan^{-1}(Q/P) \). Note that no correction for the changing plan area of the sample has been made, as recommended in clause 4 of BS 1377:Part 7:1990. Petley (1966) looked into the significance of the difference, he proposed a correction to offset it but concluded that the overall effect is small as it affects the shear and normal stress in equal proportions. Rate of sample volume changes with respect to displacement \( \Delta L \) is given by \( \psi = -\tan^{-1}(\Delta H/\Delta L) \) and plotted against \( \Delta L/L \) in Figure 8.3d. Although \( \psi \) is generally reserved for the angle of dilation it is used here for contractant and dilatant samples as this component of behaviour forms an essential part of the work concepts just discussed: negative \( \psi \) values indicate
contractant behaviour. The final quantity plotted, as shown in Figure 8.3e, is the frictional constant $\mu$ given by equation 8.4.

The results from the shearing of the fine sand and HRS samples are presented in Figures 8.3 and 8.4. Viewing the graphs (a) and (b) for both sands it can be seen that for the loose samples there is a greater contraction with the finer grain size (although its relative density is lower). With the fine sand there is a relation between volume change and normal load where the degree of contraction increases with increasing normal load. The difference in results between the series A and B tests carried out for the fine sand is negligible. A similar relation between volume change and normal load is not reliably evident with the HRS. Considering the dense samples, there is greater dilation with the HRS than the fine sand although the HRS had a slightly higher relative density (1.06 compared with an average value of 0.96); this may be related to the greater angularity of the medium size grains. The results from both sand samples indicate that the magnitude of dilation decreases with increasing normal load. In the case of the fine sand, the sample with $\sigma_n = 401$ kPa has dilated more than that with $\sigma_n = 253$ kPa but this probably results from the higher relative density of the former sample (see Figure 8.3b). The behaviour of the medium dense samples tends to be slightly dilatant in both cases with a similar trend as the dense sand with the degree of dilation decreasing with increasing normal load.

The fact that only a small part of the sample is affected by the shearing is evident from Figures 8.3b and 8.4b. There is hardly any change in the overall voids ratio volume, indicating that only the localised shearing zone reaches a critical voids ratio.

The influence of density, normal load and grain size on the angles of mobilised friction, $\phi'_m$ and dilation $\psi$ can clearly be seen in graphs (c) and (d) of Figures 8.3 and 8.4. Shearing of the loose samples results in steadily increasing values of $\phi'_m$ and $\psi$ with no peak, just a levelling out as conditions of shearing at constant volume are approached. At this point values of $\psi$ are approximately zero. As the samples are contracting, values of $\psi$ for the loose samples are negative. The Ham River sand reaches conditions of constant volume more quickly than the fine sand for which $\phi'_m$ is still increasing slightly at the end of the test. Note that for all densities, values of $\psi$ have not been plotted in the initial stages as there is considerable scatter arising from the small volume changes during take-up of slack within the apparatus.

The brittle behaviour of the dense samples is evident in the $\phi'_m$ and $\psi$ plots, particularly with the more angular HRS. The maximum angle of dilation decreases with increasing normal load, as would be expected as $\psi$ is based on the changing volume as shown in Figures 8.3a and 8.4a. The variation in $\phi'_m$ representing the relationship between the mobilised shear stress and the applied normal, can be seen with $\phi'_m$ decreasing with increasing $\sigma_n$ in a similar fashion to $\psi$ (the exception again being the fine sand at $\sigma_n = 401$ kPa). The effect is much more marked
with the HRS, the behaviour provides an example of increasing internal shearing resistance with decreasing stress level (as mentioned in Section 8.2.1). In fact, at all densities for both sands, \( \phi'_{\text{m}} \) has the greatest maximum values for the tests with the lowest applied normal load. At the two higher applied loads for the loose and medium dense sand there is not a clear trend perhaps confirming that the failure envelope (in \( r-\sigma_n \) space) is curved only for a certain range in the low stress level region, after which it becomes straight. The results from the dense samples indicate that the envelope is curved over a greater range of applied normal stress than for the loose samples. The effect is clearly seen for the more brittle HRS, as can be seen in Figure 8.5. Note that the medium dense samples also exhibit dilation but to a much lesser extent; the relation with applied normal load is again evident.

The variations of the frictional constant, \( \mu \) with displacement are given in Figures 8.3e and 8.4e. During the early stages of the tests when \( \phi'_{\text{m}} \) is being mobilised, values of \( \mu \) gradually increase. Once shearing approaches conditions of constant volume, when \( \psi \) is negligible, \( \mu \) becomes constant as would be expected. The fact that \( \mu \) is not constant throughout shearing implies that additional energy used in dilating the sample is not entirely accounted for by the increases in angle of internal friction above the constant volume value. It therefore seems that a small, but finite, amount of energy is used in elastically deforming the particles and generating frictional heat.

The test results have so far only been discussed qualitatively; peak values of \( \phi'_{\text{m}} \) and \( \psi \) and values of \( \phi'_{\text{m}} \) towards the end of shearing (\( \phi'_{\text{mcv}} \)) with corresponding \( dL/L \) and frictional constant \( \mu \) are given in Table 8.2. Because of the scatter in the \( \psi \) and \( \mu \) plots, values have been determined by bounding the scatter zone for each test and constructing an average line (these plots are not presented). In some cases the \( \phi'_{\text{mcv}} \) values given are those prior to significant top cap rotations occurring (a note was always made when the top cap had reached a set degree of rotation, measured with a small spirit level). However, in many of the tests, the top cap started rotating after peak dilation had occurred in which case a value was chosen to correspond to soon after \( \psi = 0 \) was reached. This might explain why a relationship still exists between \( \phi'_{\text{mcv}} \), initial relative density (also given in Table 8.2) and applied normal stress, \( \sigma_n \).

Assessing the effects of top cap rotation is not straightforward, if the effect is ignored completely then, from Figures 8.3.c and 8.4.c, \( \phi'_{\text{mcv}} \) can be taken to be \( 33.5^\circ \pm 1.5^\circ \) and \( 31.5^\circ \pm 1.5^\circ \) for the fine sand and HRS respectively. In general, higher \( \phi'_{\text{mcv}} \) values are associated with the dense samples, indicating perhaps that shearing to conditions of constant volume is not complete.

Considering the numerical values given in Table 8.2 it is worth commenting that the maximum \( \phi'_{\text{m}} \) and \( \psi \) values occurred for the dense sands at low normal stresses. Maximum
values measured were for the fine sand, \( \phi'_m = 35.5^\circ \) and \( \psi = 12.7^\circ \) and for the HRS, \( \phi'_m = 40.3^\circ \) and \( \psi = 20.0^\circ \). Values of \( dL/L \) are given to provide an estimate of the shearing displacement required to mobilise \( \phi'_m \) and \( \psi \); generally \( \psi_{max} \) was mobilised just before or at the same time as \( \phi'_m \). The Ham River sand mobilised shearing resistance at lower displacements than the fine sand and its behaviour was much more brittle (in fact there is not much of a peak in the plots for the fine sand). Note that absolute displacements in mm can be determined from the \( dL/L \) values by multiplying them by the length of the shearbox, 60 mm.

Values of the frictional constant, \( \mu \) towards the end of shearing (i.e. when \( dL/L > 0.1 \)) are in ranges of about \( \mu = 0.65 \) to 0.70 for the fine sand and \( \mu = 0.59 \) to 0.67 for the medium sand. Clearly these ranges represent \( \tan \phi'_m \) values, as by this stage \( \psi \to 0 \) (refer to equation 8.5).

8.2.4 Results from the shearbox interface tests

In order to increase an understanding of the behaviour of the sands being sheared against the model nail in the triaxial test series, shearbox interface tests were performed. The interface was prepared in the same way as the protective coverings for the radial stress measuring devices (see Section 7.2.4). The netting material was stretched over a metal block, of same area dimensions as the shearbox, with the elasticated threads across the block (so as to be perpendicular to the direction of shearing). It was overlapped around the edges of the block and a single thin uniform coat of latex applied to the top and sides to hold the net in place. Once the latex has dried, a final protective shellac layer was applied.

The interface was set in the lower half of the shearbox so as to be slightly proud of its upper surface to allow for any compression during application of the normal loads. The tests were performed in the same way as the sand-sand reference tests.

The results from the interface tests are presented in Figures 8.6 and 8.7. They are in the same format as the reference tests except that \( \phi'_m \) is replaced by the angle of interface shearing resistance \( \delta' \).

Changes in sample volume and density during shearing are shown in the (a) and (b) plots of Figures 8.6 and 8.7. Trends in behaviour similar to those in the reference tests can be seen with a greater contraction of the loose samples with increasing applied normal stress, the effect being much more pronounced with the fine sand. The fine sand sheared against the interface contracts more than in the sand-sand tests (by about 50 % for the low values of applied normal stress) even though its average relative density (as marked in Figures 8.3.b and 8.6.b) is higher. The converse occurs with the HRS which starts dilating after about 1 mm displacement, regardless of applied normal stress. Therefore all the Ham River sand interface tests show
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<th>$\phi_m^{\prime}$ (degs.)</th>
<th>$\psi_{\max}$ (degs.)</th>
<th>$dL/L$</th>
<th>$\phi_{\psi_{\max}}$ (degs.)</th>
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<th>$\mu$</th>
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Table 8.2. Values of $D_n$, $\phi_m^{\prime}_{\psi_{\max}}$, $\psi_{\max}$, and $\mu_{cv}$, with corresponding values of $\mu$ and $dL/L$ for fine and Ham River sand reference shearbox tests.
predominantly a dilatant behaviour, although once again the relative densities of the interface tests are generally higher than those in the reference samples.

The *medium dense* samples have similar behaviour to that observed in the reference tests with a small degree of initial contraction followed by dilation, the magnitude of which decreases with increasing applied normal stress. However, the onset of dilation starts sooner than for the reference tests and has associated volume changes between 2 and 4 times greater. These effects could again be partly attributable to the higher relative densities at the start of shearing.

Behaviour of the *dense* samples when sheared against the interface is dilatant from almost the very beginning of shearing but the well-defined trend of increasing dilation with decreasing applied load witnessed in the sand-sand tests is not evident. With the fine sand the converse occurs after 1.0 to 1.5 mm displacement (i.e. dilation increases with increasing applied normal stress). The dense HRS exhibits the behaviour seen in the sand-sand tests at the very beginning of shearing (i.e. displacement less than 1 mm) and then again at larger displacements of about 5 mm.

The effects of sample top cap rotation are evident in both sands after 3 to 5 mm displacement for many of the tests, especially those at higher applied normal stresses. Volumetric results beyond these points, marked by a rapid change in \(dH/H\), are not considered reliable. The influence of top cap rotation with regard to calculated \(\delta'\) values is discussed when referring to Table 8.3. Errors associated with rotation are more pronounced in the interface tests and occur sooner than for the reference tests: probably because of the thinner sample thickness.

The average values of relative density are also marked for each state. A comparison between the relative densities at the start of the shearing for all tests is shown in Figure 8.8. In all but one case relative density appears to be higher for the interface tests, the effect is partly related to a compression of the interface itself and is more noticeable with the Ham River sand where particles are more able to embed themselves. On the subject of compression of the interface material, this effect along the leading side of the steel block during shearing has been assumed negligible, it could possibly lead to slight over-estimations of \(dL/L\) but should be similar for most tests.

The relative densities shown in Figure 8.8, although covering an adequate range, appear to be in the upper region for each state, especially the dense sample which for the Ham River sand always have values greater than unity (for reference and interface tests). The reason for this is that the \(e_{\text{min}}\) and \(e_{\text{max}}\) values used to calculate \(D\), are determined under atmospheric conditions. At the start of shearing the normal stress has been applied resulting in further particle reorientation and elastic compression.

The influence of density, normal load and grain size on the interface friction angle, \(\delta'\),
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<th>$\delta'_{\text{max}}$ (degs.)</th>
<th>$dL/L$</th>
<th>$\psi_{\text{max}}$ (degs.)</th>
<th>$dL/L$</th>
<th>$\mu$</th>
<th>$\delta'_{\text{cv}}$ (degs.)</th>
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Table 8.3. Values of $D_r$, $\delta'_{\text{max}}$, $\psi_{\text{max}}$, and $\delta'_{\text{cv}}$ with corresponding values of $\mu$ and $dL/L$ for fine and Ham River sand interface shearbox tests.
and angle of dilation, $\psi$, is shown in plots (c) and (d) of Figures 8.6 and 8.7. Similar trends in behaviour, as seen in the reference test results, are evident in the mobilisation of $\delta'$ which continues to increase with shearing of the loose samples as it gradually approaches $\delta''$, but which reaches a peak before dropping off towards $\delta''$ with the dense samples. The peaks are not as well defined as with the reference tests indicating more ductile behaviour, also the increasing $\delta'$ values with the loose and medium dense samples can be seen to step up after a certain displacement in many of the tests.

The stepping is caused by rotation of the top cap and the boundary conditions of the apparatus. If the angle of rotation of the cap is taken as $\theta = 10^\circ$ and the system applying the normal load is influenced such that only $P\cos \theta$ is transmitted to the horizontal plane, the resulting increase in $\delta' = \tan^{-1} (Q/P\cos \theta)$ is $1.5\%$, corresponding to $0.4^\circ$ for a value $\delta' = 30^\circ$. The increase in $\delta'$ values at the step is generally of the order of degrees indicating that top cap rotation is only partly responsible. Although the exact reason for the step is unknown, values of $\delta'$ after the step are considered to be valid because the lines for most of the tests converge to a constant volume value within $2^\circ$ to $3^\circ$. The values are included in Table 8.3 (and Table 8.2 although the effects were not as dramatic in the reference tests).

As a final point, if the upper half of the shear box rotates slightly as well as the top cap, then errors from the $P \cos \theta$ component will be countered by the shearing component $Q \cos \theta$ (assuming it rotates by the same amount as the top cap).

Plots for the angle of dilation $\psi$ have considerable scatter, particularly with the fine sand tests, which is a direct reflection of the $dL/L$ versus $dH/H$ plots discussed above. The contractant behaviour observed in the loose reference tests is far less pronounced for both grain sizes. Angles of dilation are positive for most tests after about 1 mm displacement indicating predominantly dilational volume changes for the interface tests. The $\psi$ values for the dense samples are considerably reduced from those observed in the sand-sand tests, the brittleness has also increased with a sharp fall-off after peak, especially with the medium grained HRS.

Relationships between applied normal stress, $\sigma_n$ and $\delta'$ and $\psi$ are generally similar to but less pronounced than those for the reference tests with both $\delta'_{\text{max}}$ and $\psi_{\text{max}}$ increasing with decreasing $\sigma_n$. The exception is the dense fine sand which exhibits the converse with respect to $\delta'$ for unknown reasons.

Values of the frictional constant, $\mu$, have more scatter than the sand-sand tests but similar trends with shearing, many of the plots have been curtailed to reduce scatter. The increased scatter compared to the reference tests almost certainly arises because of the compression of the interface.

The values of $\delta'_{\text{max}}$, $\delta'_{\text{cr}}$, $\psi_{\text{max}}$, $\mu$ and corresponding $dL/L$ values for the interface tests
are given in Table 8.3, values have been determined using the same approach adopted for the sand-sand tests, most of the comments made in Section 8.2.3 concerning the values given in Table 8.2 are equally applicable here. For the fine sand and HRS, \( \delta'_{\text{max}} = 36.1^\circ \), \( \psi_{\text{max}} = 9.2^\circ \), and \( \delta''_{\text{max}} = 38.4^\circ \), \( \psi_{\text{max}} = 14^\circ \) respectively. These values are only slightly lower than the \( \phi'_{\text{maxz}} \) and \( \psi_{\text{max}} \) values observed for the sand-sand tests, they are generally mobilised at similar or slightly smaller displacements. There are also similar \( \delta'_{\text{e}} \) values to those of \( \phi'_{\text{mcv}} \) being \( 32.0^\circ \pm 1.0^\circ \) and \( 30.2^\circ \pm 1.7^\circ \) for the fine sand and HRS respectively, i.e. about \( 1.5^\circ \) lower. Because of the top cap rotations it is possible that constant volume conditions have not been reached in every case, especially where \( \psi \) values are still finite at the end of the test.

Peak and constant volume values of \( \phi'_{\text{m}}, \delta' \), and \( \psi \) are presented for the reference and interface tests in Figures 8.9 and 8.10 (fine sand and HRS respectively) as a means of summarising and comparing the sand behaviour in the two types of test.

### 8.2.5 Shearbox tests performed by Everton (1991)

Everton (1991) investigated the shearing properties of a wide range of grain sizes using the shearbox, shearing soil to soil and against various interfaces. The coarsest sand tested was from the same 7/14 batch used in this study for coarse sand tests. Potential misinterpretations can arise from the testing of large grain sizes in the conventional size shearbox as mentioned in Section 8.2.1 and BS 1377:Part 7:1990 (compare grain size \( D_{50} = 1.40 \text{ mm} \) with shearbox side dimension of 60 mm and typical gap width less than the \( D_{50} \) value). Nonetheless, the results provide a useful aid to the interpretation of conditions at the interface during the nail tests performed in coarse sand under triaxial conditions (discussed in Section 8.4.3).

The tests were performed at three densities under normal stresses of \( \sigma_n \) of 50 kPa and 100 kPa and sheared soil to soil and against a steel interface. The results are summarised in Table 8.4. Reported values of \( \phi'_{\text{mmax}} \) and \( \phi'_{\text{mcv}} \) are significantly larger than those from the Ham River sand when tested at similar densities and normal stress. The effect of testing under lower normal stress is also evident with peak values increasing by 3 to 4 \( ^\circ \) and constant volume values by 4 to 5 \( ^\circ \).

Results from the interface tests against steel indicate a much greater reduction in frictional angles \( \delta'_{\text{max}} \) and \( \delta'_{\text{e}} \), compared with values from the soil to soil tests (i.e. \( \phi'_{\text{mmax}} \) and \( \phi'_{\text{mcv}} \)) than was observed for the membrane interface used in this study. Peak interface angles reduce by 40 to 50 \% compared with those from soil to soil tests. Corresponding reductions with the membrane interface used in this study were always less than 10 \%.

### 8.2.6 Summary and conclusions of shearbox testing

The results from a comprehensive series of shearbox tests have been discussed in detail in Sections 8.2.4 and 8.2.5. The behaviour between sand-sand tests and sand-membrane
interface tests have been discussed and compared for the fine sand and medium grained HRS in loose, medium dense and dense states.

<table>
<thead>
<tr>
<th>State</th>
<th>$\sigma_\text{n}'$ (kPa)</th>
<th>Initial rel. density, $D_r$</th>
<th>$\phi_\text{max}$ (degs.)</th>
<th>$\phi_\text{r}$ (degs.)</th>
<th>Displacement $dL$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>loose</td>
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<td>41.7</td>
<td>39.9</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.25</td>
<td>37.7</td>
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<td>1.3</td>
</tr>
<tr>
<td>medium</td>
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<td>46.5</td>
<td>40.2</td>
<td>1.2</td>
</tr>
<tr>
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<td>100</td>
<td>0.52</td>
<td>42.1</td>
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<td>0.7</td>
</tr>
<tr>
<td>dense</td>
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<td>0.76</td>
<td>53.8</td>
<td>39.5</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.76</td>
<td>50.4</td>
<td>35.3</td>
<td>0.9</td>
</tr>
</tbody>
</table>

a.) Soil to soil tests.

<table>
<thead>
<tr>
<th>State</th>
<th>$\sigma_\text{n}'$ (kPa)</th>
<th>Initial rel. density, $D_r$</th>
<th>$\delta_\text{max}$ (degs.)</th>
<th>$\delta_\text{r}$ (degs.)</th>
<th>Displacement $dL$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>loose</td>
<td>50</td>
<td>0.29</td>
<td>24.4</td>
<td>22.7</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>0.29</td>
<td>22.8</td>
<td>22.8</td>
<td>0.3</td>
</tr>
<tr>
<td>medium</td>
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<td>0.49</td>
<td>25.6</td>
<td>22.7</td>
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<tr>
<td>dense</td>
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<td>1.2</td>
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<tr>
<td>dense</td>
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<tr>
<td></td>
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<td>0.76</td>
<td>25.3</td>
<td>24.2</td>
<td>0.35</td>
</tr>
</tbody>
</table>

b.) Steel to soil interface tests.

Table 8.4. Results from shearbox tests performed by Everton (1991) on coarse 7/14 sand.

The test data as shearing progressed are presented in Figures 8.3 and 8.4 for the reference tests and Figures 8.6 and 8.7 for the interface tests. The behaviour at peak and constant volume conditions are summarised in Figures 8.9 and 8.10 with the numerical values given in Tables 8.2 and 8.3.

A summary of results from shearbox tests on coarse sand performed by Everton (1991) are included in Table 8.4 to supplement those from the fine sand and HRS.

The results from the shearbox tests and the trends observed are used in Sections 8.4 and 8.5 where conditions at the nail interface, observed during the model nail element tests, are discussed.
8.3 RESULTS FROM REFERENCE AND MODEL NAIL ELEMENT TESTS PERFORMED UNDER TRIAXIAL CONDITIONS

In this section the results from tests performed using the apparatus described in Chapter 6 are discussed in relation to boundary stresses and displacements. Tests were performed using the three sands previously described in Section 8.2 of fine sand, HRS and coarse sand each at loose, medium dense and dense states.

The samples were prepared and tested in a dry state as described in Chapter 6, with an isotropic compression stage to $\sigma' = p' = 250$ kPa followed by shearing in axial extension. For each nail test performed, a corresponding reference test (with no nail) was performed so that direct comparisons can be made. In this way the nail benefit is examined in terms of volumetric behaviour and stiffness.

Results from the instrumented model nail are discussed in Section 8.4. The overall behaviour, linking conditions at the external boundaries with those at the nail interface, is covered in Section 8.5.

Prior to discussing the comparative behaviours between the nail and reference tests, some comments are made concerning the sample boundaries, the use of invariants and the approach taken to make direct comparisons.

8.3.1 Sample boundaries and the use of invariants

When analysing the results from conventional triaxial tests it is normal to express the sample behaviour in terms of invariants of stress and strain, so defined as to satisfy the work equations (see Atkinson and Bransby 1978, pp 52-59).

The components that make up the invariants (e.g. $\sigma_{xx}$, $\sigma_{yy}$ and $\sigma_z$ for stress invariants under triaxial conditions) are considered to act under conditions of uniform stress and strain. This is a reasonable assumption in conventional triaxial testing with lubricated ends or over the central length of the sample, up to the point when a failure plane develops (although non-uniform stresses may also result from the rigidity of the sample platens).

Conditions of stress in the reference tests performed might reasonably be considered as uniform as the boundaries are flexible (except the base) and stresses are applied directly to them by water pressure. However, the membranes through which stresses are applied are in practice likely to suffer some restraint at their ends and corners. This effect combined with the difficulty in preparing a sand sample of uniform density almost certainly leads to varying strain levels in the sample.

In the case of the nail tests, significant additional conditions of non-uniform stress and strain are imposed because of the interaction taking place between the sand and nail. The implication of this is that measured and deduced boundary stresses and displacements are
average values acting over the whole boundary being considered. Strains determined from these average displacements are clearly averages as well, while in reality, large strain gradients would be expected within the sample, e.g. in the immediate vicinity of the nail.

The same invariants used to express results from conventional triaxial tests are used for both the nail and reference test results as a means of comparison. It is important to be aware though that conditions within the nail samples are not uniform. For this reason, invariants used to express overall conditions within the nail samples, are given a superscript \( n \). Otherwise the invariant quantities are calculated using boundary measurements in the normal manner. This approach is considered acceptable as it is clearly necessary to make overall comparisons in a quantitative manner.

The conditions being modelled are those relating to a single nail within an excavation. In practice, it is the overall soil behaviour that has to be considered when analysing/designing a nailed-soil excavation. However, understanding conditions at the nail-soil interface is important for assessing the benefit of the nail and the displacements required to mobilise it. Quantifying the non-uniformity of stresses around the nail is possible to some degree by the measurements from the nail instrumentation. Localised strain variations are harder to assess. Further discussion on assessing localised stresses and strains is given in Section 8.4.

A final point on the subject of boundaries is that no corrections have been made for the effect of membrane penetration. There are two reasons for this: during the shearing stage there is no change in applied radial stress and so any penetration that has occurred remains the same. The membrane through which the axial stress is applied occupies a much smaller area and is thicker: penetration effects are therefore taken to be negligible. The magnitude of volume change associated with membrane penetration is considered to be negligible compared to the overall sample volume changes taking place (given the sample dimensions: diameter \( \approx 175 \text{ mm} \) and height \( \approx 320 \text{ mm} \)).

### 8.3.2 Behaviour of nail and reference test during the isotropic compression stage

Although the principal stage during testing is that of shearing, it is necessary to consider the conditions and behaviour of the sample from setting up through isotropic compression to appreciate the relative state of the samples at the start of the shearing stage.

Set procedures were employed to achieve comparative sample densities between nail and reference tests, as described in Chapter 6. Despite this, the reference samples in all cases but one started in a denser state than the nail test samples as can be seen in Figure 8.11, which shows relative density \( D_r \), plotted against mean effective stress at the beginning and end of the isotropic compression stage. The numerical values of \( D_r \) are given in Table 8.5 (values of \( e_{mn} \) and \( e_{ms} \) used to determine \( D_r \) are given in Table 8.1). The final values given include creep
<table>
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<th>Medium dense</th>
<th>Dense</th>
</tr>
</thead>
<tbody>
<tr>
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<td>end</td>
<td>start</td>
</tr>
<tr>
<td>Fine</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reference test</td>
<td>0.926</td>
<td>0.873</td>
<td>0.841</td>
</tr>
<tr>
<td>Nail test</td>
<td>0.947</td>
<td>0.899</td>
<td>0.880</td>
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<tr>
<td>HRS</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Reference test</td>
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<td>Nail test</td>
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<td>0.732</td>
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<td></td>
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<tr>
<td>Reference test</td>
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<td>0.708</td>
<td>0.650</td>
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<tr>
<td>Nail test</td>
<td>0.747</td>
<td>0.715</td>
<td>0.679</td>
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</table>

<table>
<thead>
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<th>Dense</th>
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<td>end</td>
<td>start</td>
</tr>
<tr>
<td>Fine</td>
<td></td>
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<tr>
<td>Reference test</td>
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<td>0.379</td>
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<tr>
<td>Nail test</td>
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<tr>
<td>HRS</td>
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<tr>
<td>Reference test</td>
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<td>0.446</td>
<td>0.605</td>
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<tr>
<td>Nail test</td>
<td>0.232</td>
<td>0.351</td>
<td>0.435</td>
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<tr>
<td>Coarse</td>
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<td></td>
</tr>
<tr>
<td>Reference test</td>
<td>0.332</td>
<td>0.448</td>
<td>0.657</td>
</tr>
<tr>
<td>Nail test</td>
<td>0.307</td>
<td>0.422</td>
<td>0.552</td>
</tr>
</tbody>
</table>

Table 8.5. Values of voids ratio, $e$, and relative density, $D_r$, at start and end of isotropic compression stage.
which took place overnight after application of isotropic stress. It can be seen from Figure 8.11 that the increase in $D_r$ between setting up and the start of shearing is about the same for each pair of nail and reference-test samples. In general this increase in $D_r$ is greater with decreasing density. Another trend that is evident is that for smaller grain sizes lower relative densities can be achieved and conversely denser samples with coarser grain sizes.

Figure 8.12 shows the measured relationships between volumetric strain $\varepsilon_v$ and $p'$ or $p''$ during isotropic compression. It can be seen that the looser the sample the greater the volumetric strain, as is to be expected. Also, the magnitude of volumetric strain increases with decreasing grain size (although this might be partly due to the looser initial state of the finer grained sands, as mentioned above). The shape of the curves indicates that sample stiffnesses increase with mean effective stress. Generally, by the end of the isotropic compression stage, the volumetric strains of the samples with nails are greater than those without, the exception being the dense and medium dense fine grained sand samples. The inconsistency with the fine sand indicates perhaps that it is the presence of the nail that is influencing the volumetric behaviour rather than the lower relative densities associated with the nail test samples.

To investigate the presence of the nail further in terms of boundary strains, the axial and radial components of strain are plotted against mean effective stress in Figures 8.13 and 8.14 respectively. It can be seen, from Figure 8.13, that in all cases the axial strains occurring in the reinforced samples are greater than those of the reference tests by the end of the compression stage. The small degree of axial extension seen during the initial compression of some of the samples probably results from slight differences between the application of radial and axial stress, which were applied manually.

Trends in the radial strains between nailed and reference samples shown in Figure 8.14 are not so clear. In some instances those of the nailed samples are less than those unreinforced, indicating that it is the radial component of strain that gives rise to the inconsistency in overall volumetric strains observed in Figure 8.12.

The magnitudes of radial strain are about double those of axial strain, the scale used for radial strain in Figure 8.14 being twice that used for axial strain in Figures 8.13. Greater differences are observed with decreasing grain size. The structure of the samples is therefore anisotropic, with greater stiffness axially than radially, probably as a result of the method of deposition. In general there is greater radial restraint with increasing density. The low radial stiffness might also be partly responsible for the small initial axial expansion that occurs with the HRS and coarse sand reference test samples.

8.3.3 Comparing nail and reference test results from the shearing stage

The shearing stage simulates conditions of stress relief at the face during a nailed-soil
excavation. Results from the shearing stage of the reference and nail triaxial tests are compared in terms of axial and shear stiffnesses and work. When considering stiffnesses the results from the nail tests and their corresponding reference tests are plotted on the same graphs, these are presented in Section 8.3.4.

The idea of expressing test results in terms of work is considered in Section 8.2.2 for the shearbox tests. An advantage of using work as a quantity is that it includes both volumetric and distortional components. The results are presented in Section 8.3.6 as differences in work expended between nailed and reference samples.

The common parameter used to make the direct comparisons between the reference and nail triaxial test results is stress ratio \( \eta = q/p' \). The relationships between \( \eta, q, p', \phi, \phi' \) for the shearing stage stress path are shown in Figure 8.15. Stress ratio was chosen because it is composed of invariant quantities, indicates the degree of shearing that has taken place from the end of the isotropic stage and because nail and reference tests were run under similar conditions and rates of stress control, increments of stress ratio between readings were generally equal for pairs of tests. This latter point facilitated the processing of the data where results from each pair of tests had to be lined-up prior to computing differences. Having completed the process for all test results, differences in axial and volumetric strains were calculated as well as work, using the same common points.

8.3.4 Behaviour of nail and reference test samples during shearing in terms of stiffness

Nail and reference test results are discussed in this section in three ways. The components of axial, shear and volumetric strain for each type of test are considered and the derived secant axial and shear stiffnesses determined. Comparisons of volumetric stiffness, \( i.e. \) bulk modulus, are not made because in the cases of the loose and medium dense samples there is a change from dilatant to contractant behaviour during the initial stage of shearing. This change in sense of volumetric strain, results in very high, unrealistic, values of bulk modulus being calculated as the volumetric strain passes through its zero axis. Also it is not usual to calculate values of bulk modulus during shearing. Three sets of figures are therefore presented for each sand expressing the different components of strain and stiffness where appropriate, so that the parallel nail and reference test data can be compared.

Axial stiffness

Axial strain and stiffness for the fine sand tests are shown in Figure 8.16. The upper graphs, a and b, express axial strain and secant stiffness against stress ratio. Secant stiffness is plotted against axial strain given on a logarithmic scale in graph c, which is a conventional way of expressing changes in stiffness with strain. Graphs of tangent stiffness were also produced but were found to be of only limited value because of their degree of scatter, they are therefore
The first two graphs indicate that the stiffnesses of the reference samples are greater than those of the nailed samples down to stress ratio values of about −0.6. The stiffnesses of the reference and nailed samples are more or less the same from this point to stress levels of about −1.0 after which the nailed samples generally start to become stiffer. At this stress ratio the axial strains are in a range of about −1.0 to −2.5 %.

Further information about the relative reference and nailed sample behaviours at small strains can be obtained from the stiffness curves, expressed against a logarithmic scale of axial strain as shown in Figure 8.16c. The shape of the curves implies that at the very start of shearing some of the reinforced samples are stiffer than those of the reference tests. In general, this is only so at very small axial strains less than about −0.01 %. Note that from Section 6.2.5, the smallest reliable value of axial strain is estimated to be 0.001 %.

Similar trends in behaviour are also seen with the HRS and coarse sand test results shown in Figures 8.17 and 8.18. General observations may be made on the results from all three sand grain sizes. In most cases the axial stiffness of the nailed samples is greater than that of the corresponding reference sample at small strains of less than −0.005 %. During the small strain range (i.e. up to about −0.1 % axial strain) the stiffnesses of nailed samples decrease rapidly, whilst those of the reference samples reduce more gradually, particularly for the HRS and coarse sand. As a consequence, by axial strains of about −0.1 %, the stiffnesses of the nail samples are generally lower than those of the reference samples. This is evident both in plots of stiffness versus stress level and axial strain (graphs b and c). Beyond this strain level, there is a marked decrease in the stiffness with strain of the reference samples. In contrast, their degradation in stiffness then becomes faster than that of the reinforced samples, consequently at strain levels of −1.0 to −1.5 %, stiffnesses are more or less the same. In some cases, the nailed samples become stiffer than the reference samples with further increased axial strain.

The axial strain versus secant stiffness plots given in Figures 8.16c to 8.18c indicate that beyond axial strains of about −0.1 % the stiffnesses of the reference samples are almost the same regardless of density for all three sands. This behaviour is contrary to that observed by other researchers (Kuwano 1997) and that seen in the shearbox tests where stiffness increased with density. The stiffnesses decrease at any given strain level with increasing grain size. The upper limit of $E_{sec}$ at $\epsilon_s = −0.01\%$ ranges from 50 MPa, through 38 MPa to 35 MPa for the fine sand, HRS and coarse sand respectively.

These stiffness values are about two to three times lower than those obtained from similar sand tested at corresponding strain and stress levels in conventional triaxial test apparatus with rigid end platens, (Kuwano 1997). There are several reasons that could be responsible for
this difference. In the apparatus used, the upper axial boundary was flexible and the sample dimensions larger. It is also worth noting that the tests performed by Kuwano were with local instrumentation, whereas in this case the strains measured were average strains. The behaviour may also be partly attributable to the effects of the phenomenon of arching, which is discussed in relation to the test results in Section 8.4.6.

Another factor which may explain the difference in magnitude of stiffness between tests is that the comparisons made are between dry and fully saturated sands. Skinner (1969) showed that for glass ballotini the angle of interparticle friction, $\phi_\text{p}$, can be increased by factors of between 3.5 and 30 by flooding a dry sample. Although he concludes that the $\phi_\text{m}$ and $\phi_\text{v}$ values in shearing are unaffected by this result, as they are mainly governed by the mechanics of particle interlocking, it does seem plausible that the interparticle friction could have an effect on stiffness in the small strain range. This is best justified by considering results from two comparative shearbox tests performed on dry and wet glass ballotini ($D_\text{p}=1 \text{ mm and initial porosity of 34.9 \% with a low applied normal stress of only about 25 kPa}$) expressed as shear load versus horizontal displacement. Skinner observes that there are load fluctuations in the wet high $\phi_\text{p}$ tests that are about five times greater than those occurring with the dry ballotini and attributes these to different shearing processes. Movement between particles during shearing taking place by sliding in the case of the dry low $\phi_\text{p}$ grains and rolling for the wet high $\phi_\text{p}$ grains. It is interesting to note that the fluctuations do not commence until almost peak shear load, implying perhaps that $\phi_\text{p}$ is being mobilised in the small strain region. If this is so, variations in stiffness would be expected and are in fact evident in the graphs presented in Skinner’s work.

In addition to the differences that are likely to arise because of the wet and dry condition of the sand in the different tests, another factor is that the samples tested by Kuwano were sheared undrained. Although results have been compared in terms of shear stiffness, thus eliminating influences from pore water pressure, differences in behaviour under drained and undrained conditions are not necessarily expected to be the same.

The differences in stiffnesses discussed above do not affect the comparison between the nail and reference tests as both were performed under the same conditions.

In summary, the stiffnesses, in terms of axial strains, of some of the reinforced samples are greater than those of the reference samples at the very beginning: this might be related to the interface stress reversal along the nail after the isotropic compression stage. At slightly higher strains ($-0.005\% > \varepsilon_\text{r} > -0.1\%$) the reference samples are stiffer; possibly while frictional forces are being developed along the nail, after which the stiffnesses of the reinforced samples are sometimes slightly greater than those of the reference test samples but generally they are more or less the same. It will be possible to comment further on the stiffness behaviour after
assessing the stresses on the nail in Section 8.4.

Another significant observation relating to the test results, is that very high values of stress ratio are reached in both the reference and nailed samples. According to the critical state equation plotted in Figure 8.15d, values of the angle of internal friction in extension, $\phi'_e$, corresponding to the stress ratios reached are in a range of 55° to 65°. Although these values are very high and seem unrealistic, similarly high values of $\phi'_e$ have been recorded when testing at low stress levels under conditions of passive stress relief (Hellings, 1989 and Hellings and Burland, 1997).

Shear stiffness

Relationships between shear strain, secant shear stiffness and stress ratio for the three sands are shown in Figures 8.19 to 8.21. Relative behaviours between nail and reference test samples are very similar to those observed in terms of axial stiffness. Reasons for the similarity can be deduced from Figure 8.22 which shows the development of radial and axial strain during the shearing stage. It can be seen that the relationships between axial and radial strains are almost linear, the plots fall within a narrow range for nail and reference tests. Further trends evident from Figure 8.22 are that radial strains increase with decreasing grain size and density.

Volumetric strains

Comparative behaviours between reinforced and reference samples, are now discussed in terms of volumetric strains. Volumetric stiffnesses are not presented for the reasons given at the beginning of the section. Graphs of the variation of volumetric strain with stress ratio are shown in Figures 8.23 to 8.25 for the three sands. The differences in behaviour with state (i.e. loose to dense) and between nail and reference samples is immediately evident with loose samples contracting and dense dilating in all cases. The degree of contraction increases with decreasing grain size, and the converse occurs when considering dilatancy. The relative density $D_r$ of the samples at the start of shearing are marked alongside the relevant curves for ease of comparison between nail and reference samples. Regardless of initial density all samples dilate initially up to stress ratios between $-0.1$ and $-0.2$. In most cases the presence of the nail inhibits volumetric strains. It should be recalled that the relative densities of the nailed samples are always less than those of the corresponding reference test samples. If they had been the same the differences in volumetric strains between the nailed and reference samples would have been even greater.

A marked reversal in volume strain in the later stages of shearing can be seen in the $\varepsilon_r$ versus $\eta$ curves for the loose and medium dense fine sand and HRS shown in Figures 8.23 and 8.24. This occurs typically when stress ratio, $\eta < -1$. This reversal represents the onset of a renewed sample dilation, it indicates that overall, the sample cannot contract any further. The
grains have reached a sufficiently densely packed state that they consequently start to dilate. The amount of volumetric strain that takes place before the onset of dilation increases as sample density (or $D_r$) decreases.

The coarse sand test results from the reference samples, shown in Figure 8.25, indicate that some form of 'failure', associated with large volume changes, occurs towards the end of the shearing stage. This rapid increase in volumetric strain almost certainly results from excessive sample distortions causing the radial membrane to rupture. This effect is inhibited with the dense and medium dense nailed samples. In the case of the loose nailed sample, the magnitude of deviatoric stresses that can be sustained before its onset, is greater than that of the reference sample.

8.3.5 Conclusions from comparing nail and reference sample behaviour in terms of stiffness

Results from nail and reference test samples have been expressed and compared in terms of axial, shear and volumetric strains and stiffnesses. In terms of axial stiffness and strain, the nail samples are stiffer than the nailed samples at axial strains greater than $-1.0$ to $-1.5\%$.

The data expressed in terms of shear strain and shear stiffness indicate similar trends to axial stiffnesses, principally because radial and axial strains are roughly linearly related.

The influence of the nail within the samples seems most noticeable when the results are considered in terms of volumetric strains. These are significantly inhibited during shearing of the reinforced nail samples.

Very high values of stress ratio are sustained towards the end of the shearing stage. In many cases these values correspond to angles of internal friction in excess of $55^\circ$. Similar values of $\phi'$ have been reported by other researchers during triaxial tests at very low stress levels.

8.3.6 Comparisons between nail and reference test samples using differences in work and strain

In the previous section it was shown that the presence of the nail provided more benefit in relation to volumetric strain than for axial or deviatoric strains where it was even detrimental much of the time. It was felt that another more global way of assessing the benefit of the nail might be through the consideration of work which incorporates both volumetric and deviatoric components of strain.

The work expression was developed for the shearbox conditions in Section 8.2.2 (see equation 8.2). A similar expression encompassing distortional and volumetric components of work can be written for triaxial conditions (see Wood, 1991, p. 232).
\[ \delta W = q \delta \varepsilon_q + p' \delta \varepsilon_p \quad (8.6) \]

By definition the work expended during shearing a sample from \( \eta = 0 \) to \( \eta = \eta_n \) is therefore given by:

\[ W = \int_0^{\eta_n} q \, dq + \int_0^{\eta_n} p' \, dp \quad (8.7) \]

\( W \) represents the work done from the start of the shearing stage. The components within each integral represent the areas under the respective \( q - \varepsilon_q \) and \( p' - \varepsilon_p \) curves up to a degree of shearing expressed by \( \eta_n \). Values of the cumulative work expended during the tests on both nailed and reference tests are shown in Figures 8.26 to 8.28.

It is clear that the softer the soil the greater will be the work done for a given increment of stress. Thus, if a nail has the effect of producing a globally stiffer sample, less work will be done than for the corresponding reference sample. Thus, the benefit of the nail is defined as \( W^{\text{ref}} - W^{\text{nail}} \), which is termed the 'work difference'. When this quantity is positive the effect of the nail is beneficial.

Differences in strain levels during shearing between corresponding nail and reference test samples have also been computed at the same corresponding stress ratio values. From these plots, ranges of \( \eta \) over which the nail is beneficial can be identified. Comparisons are made only in terms of axial and volumetric strains as it was established in Section 8.3.4 that shear and radial strains are closely related to axial strains.

When considering differences in axial strain, \( \varepsilon_{a,\text{ref}} - \varepsilon_{a,\text{nail}} \), it must be remembered that as all the tests were performed in extension, axial strain is always negative. Therefore if \( \varepsilon_{a,\text{ref}} - \varepsilon_{a,\text{nail}} \) is negative, the axial strains in the reference sample are greater than those in the nail sample and so the nail is beneficial in reducing axial strains. Conversely, when the difference \( \varepsilon_{a,\text{ref}} - \varepsilon_{a,\text{nail}} \) is positive, this implies that in the sense of restraining axial strains, the nail is detrimental.

The same sign convention only applies to differences in volumetric strain \( \varepsilon_v^{\text{ref}} - \varepsilon_v^{\text{nail}} \) if the samples are dilating and the strains negative. The converse applies when the samples are contracting, therefore:

(i) if \( \varepsilon_v^{\text{ref}} - \varepsilon_v^{\text{nail}} \) is negative, the nail is only beneficial in reducing volumetric strains if the samples are dilating; otherwise for contractant samples its presence is detrimental;

(ii) if \( \varepsilon_v^{\text{ref}} - \varepsilon_v^{\text{nail}} \) is positive, the converse applies: the nail is beneficial in contracting samples and detrimental in dilatant samples.

The implication of the differences in work and strains between reference and nail test samples are summarised in the sketches in Figure 8.29 for quick reference when assessing plots.
8.3.7 Discussion on the 'benefit' of the nail

In this section the 'benefit' of the nail is discussed in terms of the differences in work and strain as defined above. Figures 8.30(a) to 8.32(a) show the relationships between $\eta$ and the work difference ($W^{\eta'} - W^{\eta''}$) for the fine to coarse sands respectively. In the figures (b) and (c) the results are presented in terms of differences in axial and volumetric strain.

Prior to discussing the differences, it should be recalled that generally the relative densities of the reference samples were greater than those of the nailed samples (refer to Figure 8.11 and Table 8.5). In the case of the medium dense samples this is particularly so because of the difficulty in controlling the sand state at this intermediate density. There are further complications with the interpretation of the results from the medium dense samples in that, depending on whether the density is above or below the critical density, the sample behaviour can be dilatant or contractant. Therefore if reference and nail sample densities fall either side of the critical density, their behaviours can be expected to be different. This effect can be seen in Figures 8.24 and 8.25 which show volumetric changes during shearing for the HRS and coarse sand. Because of the potentially misleading nature of the results from the medium dense samples (expressed in terms of differences) they are not discussed here although the plots are included in the figures for completeness.

Assessing the work plots in Figures 8.30a to 8.32a, the loose samples generally derive benefit from the nail at earlier stages of shearing than the dense samples with the exception of the coarse sand which is the only case where the density of the nailed sample is greater than that of the reference sample. Axial strains at which nail benefit starts to commence are typically within a range of $-0.2$ to $-1.2$ % (cross-referencing with Figures 8.16b to 8.18b). The differences in work for all sands in the loose and dense states continue to increase steadily with shearing indicating further benefit with displacement. The effect diminishes in the later stages of shearing (i.e. stress ratio $\eta < -1.2$) when sample failure is being approached.

If work is considered to constitute an overall means of assessing nail benefit it could be said that under working conditions (represented by a range of values of stress ratio of about $-0.6$ to $-1.0$, which has a corresponding axial strain range of about $-1$ to $-2$ %), the nail is most beneficial for loose fine sands, the effect decreasing with increasing grain size. There appears to be an improved benefit with the dense samples for the coarse sand.

In terms of differences in axial strain (Figures 8.30b to 8.32b) greater degrees of shearing and hence displacements are required before the nail starts to restrain movements axially (compared with those for work). For the fine and coarse sands, the dense samples mobilise nail forces first with degrees of restraint of axial strain of about $-1$ % being achieved by the end of shearing. In the case of the dense coarse sand, where the density of the nail test
sample was greater than that of the reference sample, the nail benefit starts at the very beginning of shearing and continues to increase throughout the stage. This might imply that if the relative density of the corresponding nail and reference test samples were the same at the start of shearing for all the tests performed, the nail benefit would come into play sooner. The differences in density that exist at the start of shearing have been attributed to the presence of the nail during the flushing and isotropic compression stages of the test (see Section 8.3.2).

The magnitude of volumetric strains occurring during shearing are comparatively small as the dilatant axial strains are countered by contractant radial strains. As a result, differences in volumetric strain between reference and nail test samples (see Figure 8.30c to 8.32c) are small and no particular trends can be readily identified. Nevertheless, referring to Figure 8.29, the nail can generally be seen to be beneficial in reducing overall volumetric strains after some degree of shearing (see also conclusions in Section 8.3.5).

8.3.8 Conclusions on nail benefit expressed in terms of boundary stresses and strains

Two methods are used to express nail benefit: the first using conventional stiffness parameters and secondly by assessing differences in work and strain throughout the shearing process.

The two approaches indicate that for most of the tests performed some degree of shearing is required before the nail starts to restrain displacement. Typically the nail is beneficial at and beyond stress ratios of $\eta = -0.6$ to $-1.0$ which roughly correspond to axial strains of $-1.0$ to $-2.0\%$. Thereafter, the degree of benefit of the nail generally increases steadily with shearing. Considering the overall restraint of deformations (i.e. as expressed in terms of differences in work) and differences in relative density between reference and nail test samples at the start of shearing, the loose sands derive the most benefit from the nail, the effect decreasing with increasing grain size.

Although the nail seems to be detrimental to a small degree in the early stages of shearing this probably results from the change in stress path from isotropic compression to axial extension.

It should be noted that the results from the medium dense samples have not been considered in this appraisal because differences in relative density between comparative tests are large. This causes misleading results when their states fall either side of the critical density (as defined by Schofield and Wroth, 1968) causing dilatant or contractant behaviour.

8.4 RESULTS FROM THE INTERNAL MEASUREMENTS MADE WITH THE INSTRUMENTED MODEL NAIL

In the previous section the influence of the nail on sample behaviour is discussed by
considering stresses and displacements measured at the sample boundaries. The results from the instrumented nail are now considered to assess how interface shear stresses and axial forces along the nail are mobilised and how radial stress is distributed.

Initially the layout of nail instrumentation is described and the parameters that can be determined from them explained. The sign conventions used are also defined.

In order to gain insight into the distribution of interface stresses along the nail at different degrees of shearing, a series of idealised models was formulated for different loading conditions and soil behaviour. These are explained and form a useful reference framework for comparison with the test results.

The data from the nail instrumentation are expressed in three ways. Most of the discussion concentrates on plots showing the distribution of force and stress along the length of the nail as a series of curves for different stages of shearing. Continuous plots of device outputs versus stress ratio are also presented; with the data in this form comparisons with the measured boundary stresses and strains can be made and additional trends emerge. The third approach is to plot ‘stress paths’ of shear and radial stress on the nail boundary, normalised with respect to the initial measured radial stress. This approach is useful for assessing the overall nail behaviour, it has been adopted as a means of expressing information from instrumented pile tests (e.g. Chow 1997).

The results from the isotropic compression and shearing stages are presented and discussed. In the light of this work, phenomena such as rigid-body inclusion effects, arching, and restrained dilation are investigated.

### 8.4.1 Measurements with the instrumented model nail

The design and calibration of the instrumented nail is covered in Chapter 7, no further details are given in this section: the data are presented as processed values of axial force in Newtons and radial stress in kiloPascals (kPa). No corrections are applied for cell-action effects, which tend to cause under-registration of the radial stress measuring devices. The magnitudes of these effects are quantified in Chapter 7 and are only referred to here in the discussion of the results.

The three axial force transducers positioned along the length of the nail (refer to Figures 7.1 and 7.2) allow the average interface shear stresses to be determined between the two consecutive pairs and between the outer devices using expressions as given below (see also Figure 8.33).

\[
\tau_{mi} = \frac{F_{ai}(i) - F_{ai}(i+1)}{A(i \rightarrow i+1)} \quad (8.8)
\]

\( F_{ai}(i) \) represents the axial force measurement from device \( i \) and \( A(i \rightarrow i+1) \) the surface area of the
nail between the midpoints of devices $i$ and $i+1$. A fourth average value of $\tau_{imat}$ can be determined between the upper device and the point at the top of the sample, where the nail passes freely through the platen, if it is assumed that the axial force there is zero.

There are three radial stress measuring devices, two of which are positioned between the axial force transducers enabling an angle of interface friction to be determined using the relevant calculated $\tau_{imat}$ values.

$$\delta' = \tan^{-1} \left( \frac{\tau_{imat}}{\sigma_{r,imat}} \right)$$  (8.9)

The sign conventions adopted are that compressive axial forces are positive (usual soil mechanics convention). The convention for soil shear stresses, namely those acting at the interface, are as defined by Atkinson and Bransby (1978 p.16) where shear stresses (and strains) are positive as angles in the positive quadrants of a soil element increase. The sign of $\tau_{imat}$ is therefore dependent on whether the nail is in compression or tension and on the distribution of this force: the layout of instrumentation and sign conventions are shown schematically in Figure 8.33. Note that the sign of $\delta'$ is directly related to that of $\tau_{imat}$.

8.4.2 Idealised models to aid interpretation of results

A series of cases using idealised models have been formulated to aid with the interpretation of conditions at the nail interface which are deduced from the instrumented nail, in particular the shear stress distribution along the nail length. Eight cases are described of increasing complexity, the final two are intended to simulate conditions during the isotropic compression and shearing stages of the laboratory tests.

Most of the cases are formulated using a model that assumes a bi-linear relationship between interface shear stress and displacement of the soil relative to the surface of the nail. At a critical relative displacement, the interface shear stress ceases to increase linearly and remains constant. The nail is assumed to be incompressible throughout.

The eight cases are shown in Figure 8.34a to 8.34h. The boundary conditions for each case are shown schematically with plots of soil displacements relative to the nail, $\Delta_{imat}$, interface shear stress $\tau_{imat}$ and axial force in the nail $F_{ar}$, all versus distance along the nail. The bi-linear model is shown for each case on the $\tau_{imat}$ graph. It should be emphasised that conditions along the interface are being considered, not those within the sample. Also no account is taken of radial stresses acting on the boundary of either the sample or the nail.

The methodology used for producing the graphs for each case is as follows. The first step is to consider the applied boundary condition (e.g. a force applied to the nail or sample) and formulate the resulting displacements. These are imposed as a series of increments of small enough size that a progressive displacement ‘front’ moves down the length of the nail. For
example in case 2 (Figure 8.34b) in the first increment, the displacement imposed at the top of the sample diminishes to zero at a fifth of the way down the nail. In the second increment this progresses further and so on. The interface shear stress $\tau_{\text{int}}$ is calculated using the $\tau_{\text{int}} - \Delta_{\text{soil}}$ relationship, i.e. the soil model, and lines of $\tau_{\text{int}}$ corresponding to the displacement are plotted in the second diagram from the top. Distributions of axial force in the nail are then obtained from the information in this second figure and are plotted in the bottom graph.

The cases have been chosen to be progressively more complex leading up to cases 7 and 8 which simulate conditions during the two main stages of the laboratory tests.  

**Case 1** (see Figure 8.34a). The nail passes freely through the platens either end of the sample and is loaded directly at one end. As the nail is incompressible and is loaded directly, soil displacements are uniform along its length. Consequently shear stresses are uniform along its embedded length and increase with increments of displacement up to the point where slip occurs, after which they remain constant. The maximum axial force in the nail occurs at the loaded end and is determined from the area under the $\tau_{\text{int}}$ line. The boundary conditions dictate that there is zero force at the base of the nail. As the $\tau_{\text{int}}$ distributions are constant the $F_{ax}$ distributions are linear (by integrating $\tau_{\text{int}}$ along the length).

**Case 2** (see Figure 8.34b). The nail passes freely through the sample platens as for case 1 but this time the sample is loaded axially. As the sample is compressible, displacements along the length of the nail are progressive. Again values of $\tau_{\text{int}}$ are directly related to $\Delta_{\text{soil}}$ with the cut-off occurring after a relative displacement of five units. Because of the different loading condition, the stresses act in the opposite sense to case 1 (refer to Figure 8.33). Nail axial forces are calculated in the same way, with the maximum value determined from the area under the $\tau_{\text{int}}$ line. As there is a linear but varying distribution of $\tau_{\text{int}}$ the resulting $F_{ax}$ values are curved (second order polynomials resulting from integration of linear $\tau_{\text{int}}$ values). $F_{ax}$ values are maximum at the fixed end of the nail, at the other end they are zero. During the initial increments, the axial force only increases along the same length from the top of the sample over which $\Delta_{\text{soil}}$ advances and $\tau_{\text{int}}$ acts. Beyond this point $F_{ax}$ is constant because the nail is incompressible. Once full slip occurs and $\tau_{\text{int}}$ values become constant, the $F_{ax}$ distribution becomes linear, e.g. for the first two increments of distance along the nail at stage 7 shown in the lower diagram of Figure 8.34b.

**Case 3** (see Figure 8.34c). The nail is now fixed to the base platen and the sample axially loaded in compression as for case 2. The axial loading conditions are similar to those during isotropic compression in the laboratory tests. Displacements are again imposed to give a progressive build-up along the length of the nail. However, because of the fixity of the nail at the base, soil displacements there are always zero. Distributions of $\tau_{\text{int}}$ and $F_{ax}$ progress as for
case 2 up to increment 5 when the soil displacement just diminishes to zero at the base. This corresponds to the point beyond which slip at the top of the nail starts to occur. It is important to note that the rate of movement of the slip ‘front’ along the nail decreases as the applied increments of displacement increase (e.g. after 10, 100 and 1000 equal increments of $\Delta_{\text{cum}}$, the maximum $F_{ax}$ at the base of the nail is 37.5, 48.75 and 49.875 units respectively).

**Case 4** (see Figure 8.34d). The same boundary conditions as those in case 3 apply, the only difference is that a ‘strain-softening’ soil model has been adopted as shown in the $\tau_{\text{int}}$ plot. The resulting distributions up to increment 5 are the same as cases 2 and 3. Thereafter, the strain softening starts to occur and the peak $\tau_{\text{int}}$ value progresses down the nail, although in progressively decreasing increments because of the fixity of the nail at the base. In this case the axial force generated in the nail has a lower and more complex distribution along its length because of the strain-softened $\tau_{\text{int}}$ values. These values become constant after softening and so $F_{ax}$ along the nail, after many displacement increments, would tend to a linear distribution with a maximum value of 30 units.

**Case 5** (see Figure 8.34e). The exercise of case 4 is repeated but using a soil model with more complex peak and post-peak displacement characteristics as illustrated within the $\tau_{\text{int}}$ plot. After the ‘hardening’, the interface shear stress generated by the soil drops to a constant value. This model might provide insight into a situation where restrained dilation occurs temporarily with soil displacement resulting in a localised zone of elevated normal stresses and hence $\tau_{\text{int}}$ value, which progresses down the nail with cumulative displacement increments. Similar observations can be made as for case 4 except that here $F_{ax}$ values temporarily increase. After many displacement increments applied to the top of the sample there would be a linear distribution of $F_{ax}$ with a maximum value of 50 units, the same final distribution as for case 3.

**Case 6** (see Figure 8.34f). Almost the same conditions exist as for case 3 with the nail fixed to the base of the sample and the bi-linear soil model. The exception is that the sample is loaded in the opposite sense, in axial extension. The effect is that all the resulting distributions and absolute values of $\tau_{\text{int}}$ and $F_{ax}$ are the same but in the opposite sense, i.e. all plots are mirrored about the nail length axis. This case is intended to simulate conditions during the shearing stage of the laboratory tests.

**Case 7** (see Figure 8.34g). The boundary conditions within this case are the same as case 3, the difference is that the displacements are applied in a compressive sense after a degree of extension, in fact starting from increment 3 of case 6. The idea behind this case is to simulate conditions during the isotropic compression stage but after a small degree of extension that was often induced whilst flushing the radial membrane. Displacements have been distributed in the same way as in case 3 but added to the existing values from increment 3 case 6 (see upper graph
of Figure 8.34g. The $\tau_{\text{int}}$ and $F_{\text{int}}$ distributions are calculated using the same approaches as before. The case illustrates how the sense of shear stress distribution can change during the course of a monotonic loading as the nail passes from a tensile to a compressive state.

Case 8 (see Figure 8.34h). A similar approach is adopted here, where a combination of conditions from two previous cases are considered. The boundary loads are imposed in the same way as for case 6, i.e. by axial extension of the sample, but applied after a considerable degree of compression, namely stage 10 of case 3. Clearly the intention with this case is to simulate the conditions of the laboratory test during shearing, after isotropic compression. Again there is a reversal of interface shear stress as the nail passes from compression to tension. During this transition, a stage is reached after two of the increments (16 and 17), where different lengths of the nail are in loaded in compression and tension at the same time while the sample is being loaded monotonically.

These eight cases, although very much idealised, give insight into different mechanisms that might occur during the isotropic compression and shearing stages of the laboratory tests. The actual test results are discussed in the next sections, in the first type of presentation, the nail actions ($\tau_{\text{int}}$, $F_{\text{int}}$) are expressed in the same way as the models, as a series of curves that develop with shearing. Only part of the distributions can be shown, i.e. that relating to the instrumented length of the nail.

8.4.3 Observed interface behaviour along the length of the nail

The experimental results from the nine nail tests representing the behaviour of the fine to coarse sand over a range of states from loose to medium dense to dense are presented and discussed in this section in terms of changes along the length of the nail. The behaviour during the isotropic compression and shearing stages of each test are given in separate figures but discussed together: trends developed in the former often affecting the latter.

As a means of explaining the layout of the results, consider for example Figures 8.35 and 8.36, which show the results from the isotropic compression and shearing stages of the test on loose fine sand. Each figure includes two ‘primary’ graphs of (a) axial force and (b) radial stress measured directly at the interface, followed by a further two graphs of (c) calculated interface shear stress and (d) angle of interface friction. These quantities are all plotted at the appropriate positions along the nail. Within each graph, a selection of curves are shown, each representing progressing degrees of compression or shearing (generally given at specific values of axial displacement, $\Delta_{\text{ax}}$). The curves are numbered and the numbers tabulated with their corresponding values of stress ratio, $\eta$, average axial displacement at the top of the sample, $\Delta_{\text{ax}}$, and the mean effective stress, $p^*$. An additional graph (e) is given on most of the shearing stage figures of $\Delta_{\text{ax}}/\Delta_{\text{ax}}$ versus
stress ratio. \( \Delta_{\text{core}} \) represents approximate axial movements at the top of the core of soil immediately surrounding the nail. Measurements of \( \Delta_{\text{core}} \) were made by means of a dial gauge bearing on the upper surface of the guide through which the nail passes (component 8 in Figures 6.1 and 6.2). These measurements were primarily made to check that the system was operating freely, sometimes they were not made regularly. Nonetheless the quantity \( \Delta_{\text{core}}/\Delta_{\text{ave}} \) gives insight into differential movement between soil immediately around the nail and the mass as a whole. Note that \( \Delta_{\text{core}}/\Delta_{\text{ave}} \) is a cumulative quantity: towards the end of shearing, incremental values of \( \Delta_{\text{core}}/\Delta_{\text{ave}} \) tend to unity.

Before discussing the results in detail a brief description will be given of three phenomena that are mentioned occasionally in the text, viz., rigid-body inclusion, arching and restrained dilation.

Rigid-body inclusion effects were first mentioned in Chapter 7 when cell-action effects were being considered. In the context of the nailed sample test results, rigid-body inclusion effects give rise to radial stresses acting on the surface of the nail that are greater than those applied at the sample boundary. A detailed discussion and explanation of these effects are given in Section 8.4.5 and Appendix 8.1 in the context of the test results.

The phenomenon of arching is often encountered in a variety of geotechnical applications (see Terzaghi, 1943). Within the confines of the test apparatus, arching causes the formation of annular structures around the nail, probably induced by the confinement of the sand within two circular boundaries. These structures cause a 'locking-up' of grains such that the sand does not behave as an isotropic medium. Radial stresses applied at the sample boundaries are not uniformly distributed through the sample but instead are transmitted into circumferential stresses. As a result the radial stresses acting on the surface of the nail are lower than those applied at the sample boundary. Arching is strongly dependent on the density of the sand structure, it becomes more prevalent with increasing density. The subject is discussed further in Section 8.4.6.

The phenomenon of restrained dilation is also discussed later (Section 8.4.7), although no evidence of it was observed in most of the test results.

Fine sand

Processed data from the fine sand tests are given in Figures 8.35 to 8.40. Data from the isotropic compression stages are considered first, shown in Figures 8.35, 8.37 and 8.39. Graphs (a) indicate that the nail in all states goes into compression, the medium dense by a smaller amount than the loose and dense samples. There is little variation along the nail length, although there is slight trend for values to decrease towards the top of the nail where \( F_{\text{ae}} = 0 \).
(this is consistent with case 7 of the idealised models shown in Figure 8.34g). Note that there is a small degree of initial extension in the nail resulting from the flushing stage. The axial displacements required to reach the target stress of $p' = 250 \text{ kPa}$ decrease with increasing density. Note that values of stress ratio tend to zero as the boundary stresses are applied isotropically, progress of the compression is best judged from the tabulated $p''$ values.

The distributions of radial stress are shown in graphs (b) of Figures 8.35, 8.37 and 8.39. The magnitude and distribution of radial stress differ significantly for the three relative densities. Inferences on the effects of arching and rigid-body inclusion can be made even at this early stage. In the loose sand test (Figure 8.35b), rigid-body inclusion effects are evident during most of the stage towards the base of the sample where radial stresses measured at the nail interface exceed those applied at the sample boundary (e.g. the radial stress values measured at the lower device for curves 2 and 3 exceed those applied at the boundary). During compression of the loose sand, the presence of the nail and the friction developed along it impede a uniform 'isotropic' compression throughout, with the result that the material towards the top of the sample becomes slightly denser than that towards the base. The upper part is thus slightly more prone to circumferential arching effects which inhibit the transfer of applied radial stress through to the centre of the sample. These effects are thought to account for the distribution and magnitude of radial stress in the loose state (Figure 8.35b).

In the case of the dense fine sand (Figure 8.39b), the magnitudes of measured radial stress are considerably lower than those applied at the sample boundary (generally by at least a factor of 2). These lower radial stresses are symptomatic of the effects of arching in dense sand. The distribution is such that values measured towards the top of the sample are higher than those measured further down the length of the nail. The dense structure at the top of the sample is disrupted by the compression process, localised dilation takes place in the vicinity of the nail which reduces the degree of arching induced there. In the lower part of the sample the sand grains are still locked-up in a dense state.

The behaviour of the medium dense sand (Figure 8.37b) is somewhere between that of the loose and the dense. The radial stresses measured on the nail are less than those applied (but not to the same degree as the dense sample) and the distribution is quite uniform. This indicates that perhaps the fine sand structure in a medium dense condition is less susceptible to localised changes in state from the applied compression stresses. Arching in the context of the experimental conditions is discussed in Section 8.4.6.

The results from the graphs (c) show the deduced values of interface shear stress, $\tau_{in}$.

It can be seen that during the isotropic compression stage, values of $\tau_{in}$ only develop towards the top of the sample, indicating that this is the area in which particle movement and
reorientation is predominantly occurring. In the case of the medium dense sample the $\tau_{int}$ distribution given in Figure 8.37c is uniform and of low magnitude.

The results from the graphs (d) show the values interface friction angle $\delta'$ deduced from radial stresses acting on the nail and $\tau_{int}$. It is only possible to deduce two local values of $\delta'$ along the length of the nail. It can be seen from graphs (d) that near the end of the nail the values of $\delta'$ are positive and towards the base they become negative for all three relative densities. The distribution of $\delta'$, though only given by two points, extends to both positive and negative values reflecting the $\tau_{int}$ distribution.

The initial $\delta'$ values prior to starting the compression stage (shown by curves 1 in the figures d) are generally quite high. They are induced from the processes of depositing the sand and flushing the radial membrane prior to testing. The membrane is filled from the base upwards, thus causing a small degree of radial squeezing and axial extension progressively up the sample. This is reflected by the uniform, albeit small, tensile force developed along the instrumented length of the nail. Generally interface shear stresses have started to develop towards the top of the nail, although the initial distributions are often slightly erratic. The uneven distributions are almost certainly a consequence of the process of depositing of the sand. The high $\delta'$ values might result from the low stress levels at this time (i.e. prior to loading the sample in isotropic compression). Other researchers have observed very high $\phi'$ values at low stress levels (Hellings 1989, Hellings and Burland, 1997). Once loading starts, the values decrease with increasing applied pressure and a more uniform distribution of $\delta'$ develops.

The results from the shearing stage are given in Figures 8.36, 8.38 and 8.40, in the same format as those for isotropic compression. The graphs (a) show the distributions of axial force, $F_{ax}$ along the nail for the three relative densities. As the sample is progressively unloaded axially, values of $F_{ax}$ in the nail steadily increase. The distribution of axial force along the length of the nail is uniform during the initial stages of shearing as the nail passes from compression to tension, indicating that, at this stage, negligible shear stresses are mobilised over the instrumented length. In graphs (c) the distribution of interface shear stresses are shown. It can be seen that the development of $\tau_{int}$ at the free end of the nail increases from the start. The fact that shear stresses are gradually mobilised over progressively greater lengths of the upper part of the nail are indicated by the increasing $F_{ax}$ values.

During the progressive unloading, there is a 'front' behind which $\tau_{int}$ is being mobilised, that moves towards the base of the nail. For example, in Figure 8.36c for the loose fine sand, it is clear that at curve 5 the front has reached the upper axial load cell, and by curve 6 has progressed beyond the middle load cell. Similar idealised behaviour is seen in case 6, Figure 8.34f, where at increment 3, $\tau_{int}$ has only been mobilised over the upper half of the nail.
The progressive increase in axial force and interface shear stress continues steadily with shearing. The final curves of the loose and medium dense samples being of similar form to those idealised in case 8 (Figure 8.34h). In the case of the dense sample, mobilisation of $\tau_{int}$ remains localised in the upper part of the nail indicating that even after large displacements the nail is only working at its upper end, transferring the forces from there to its fixed base.

It can be noted from Figure 8.36a that towards the end of shearing there is a drop of force in the nail within the loose sample, this occurs at a $\Delta_{mm}$ value of over 13 mm. It is worth noting that although displacements of about 3.5 mm occur between curves 8 and 9 the decrease in $F_{ax}$ and $\tau_{int}$ is not dramatic. Maximum displacements developed by the end of the shearing stage decrease with increasing density, neither the medium dense nor dense samples show any reduction in $F_{ax}$ or $\tau_{int}$.

In graphs (b) the distributions of radial stress are shown. It can be seen that radial stresses progressively increase, the loose and the medium dense more so than the dense. It is interesting to note that the forms of the radial stress distributions along the nail length do not change much during shearing. These trends can be explained by the same reasoning given for the isotropic compression stage, i.e. by consideration of localised changes in density and the resulting grain structures.

Radial stresses in the lower part of the loose sample (Figure 8.36b) increase to values of about 320 kPa, considerably in excess of the applied boundary stress of 250 kPa. The radial stress at the nail interface, calculated using elasticity theory in Chapter 7 for the dimensions and boundary conditions of the test was 333 kPa, giving credibility to the measured values. As a check, another nail test on loose fine sand was performed which gave maximum radial stresses of about 300 kPa, the lower value probably resulting from the slightly higher relative density of the second sample. It is thought that, of the three phenomena mentioned at the start of the section, rigid-body inclusion effects are the most likely cause of the elevated radial stresses measured.

Similar results showing an increase of radial stress at the nail interface were also obtained from the numerical analyses performed to model conditions during clay testing. These results are given in Chapter 4 (see Figures 4.6 and 4.7). Again the maximum value developed is of the order of 320 to 350 kPa depending on how conditions at the interface are modelled (the higher value being associated with fully frictional conditions).

In the case of the dense fine sand (Figure 8.40b), radial stresses remain lower than those applied at the sample boundary throughout the shearing stage. This is thought to be caused by circumferential arching within the dense grain structure.

In graphs (d) of the shearing stage figures, the calculated angles of interface friction are
shown. Further insight into the mechanisms within the sample is provided by these graphs together with the relative movements between the core and the overall area of the top of the sample. The $\delta'$ values progressively increase during shearing of the loose and medium dense fine sands but the maximum values reached fall considerably short of values obtained from the interface shearbox tests (see Table 8.3: for loose fine sand $\delta'_{max} = 31^\circ$ being mobilised after 3.5 to 4.5 mm displacement). Maximum values of $\delta'$ for the loose and medium dense sand are 23° and 22° respectively; in the case of the dense sand, the values only just start becoming positive.

Plots of $\Delta_{cur}/\Delta_{max}$ versus stress ratio are shown in graphs (e). They indicate that the relative movement of the core of soil immediately surrounding the nail reduces with increasing relative density. Towards the end of shearing, when particle reorientation starts to take place more freely, the cumulative relative movements start to increase, but movement of the core is still only about half that of the average and it should be remembered that this represents movement at the top of the sample. Approximate displacements of the top of the core, $\Delta_{cur}$, towards the end of shearing are 5.0, 3.2 and 1.5 mm for the loose, medium dense and dense samples respectively. The shearbox displacements required to mobilise values of $\delta'_{max}$ are 3.6, 3.0 and 1.2 mm respectively (estimating appropriate normal stresses in Table 8.3).

The combination of the results, from the interface shearbox tests and the nail instrumentation indicates that full interface friction should be mobilised towards the top of the nail. Further down the nail, at the position of the instrumentation (i.e. roughly midway), there are insufficient displacements to mobilise the full friction angle. The percentage of $\delta'_{max}$ mobilised at this midpoint decreases with increasing density. Clearly it also diminishes towards the base where there are no relative movements between soil and nail.

The implications of this reduction in displacement of the soil along the nail interface are as follows. For the loose fine sand, the elevated values of radial stress measured towards the base of the nail are of only limited value as the resulting interface shear stresses are also controlled by $\delta'$ which only becomes significant (but is still not fully mobilised) in the later stages of shearing, after displacements of about 1 mm (referring to Figure 8.6c).

In the case of the dense fine sand, the very low $\delta'$ values calculated indicate that there is negligible displacement of the sand along the nail interface for at least 60% of its length. The $\tau_{int}$ values between the free end of the nail and the upper axial device being plotted midway along this length, in reality $\tau_{int}$ might be even more localised towards the top of the sample. Although only small displacements are necessary to mobilise $\delta'_{max}$ with the dense sand, interface shear stresses generated over this length are very small because radial stresses are reduced by the effect of arching and there is insufficient displacement to invoke the phenomenon of restrained dilation.
The results from the medium dense fine sand sample indicate that similar values of $\delta'_{\text{max}}$ are reached as for the loose sample and that there is sufficient freedom of particles to allow the smaller displacements necessary to mobilise $\delta'_{\text{max}}$ (compared to the locked-up structure of the dense sample). This is evident from the values of $\tau_{\text{in}}$, which are greater than those mobilised in the loose sample.

**Ham River (medium) sand**

The results from the nail instrumentation during tests on the medium grained Ham River sand are shown in Figures 8.41 to 8.46. They are not discussed in the same detail as those from the fine grained sand, instead comparisons are drawn to investigate the influence of grain size. The same approach is adopted when considering the coarse grained sand.

During the *isotropic compression* stage, given in Figures 8.41, 8.43 and 8.45, the development of axial force and radial stress, shown in graphs (a) and (b), follow the same trends as for the fine sand. The nail goes into axial compression and radial stresses increase with applied boundary stresses. Both quantities are reduced in magnitude compared to the measurements made in the fine sand, the force values only marginally. Again the magnitude of radial stresses developed decreases with increasing density from the effects of arching. A similar form of radial stress distributions, as observed in the fine sand tests, where $\sigma_r$ increases towards the base of the loose sample and *vice versa* for the dense sample, is again evident but much less accentuated.

It can be seen from the tabulated values of $\Delta_{\text{av}}$ given in the figures, that the average axial displacements required to achieve compression are about the same as for the fine sand. However, the resulting interface shear stresses, shown in graphs (c), towards the top of the sample are slightly larger. As radial stresses and axial forces are generally lower than in the fine sand tests, the increased $\tau_{\text{in}}$ values can be attributed to higher $\delta'_{\text{max}}$ values which are mobilised after smaller displacements. This comparative interface behaviour can be deduced from Table 8.3 where the shearbox interface test results are summarised. Calculated values of angle of interface shearing resistance, $\delta'$ at about the mid-length of the nail, given in graphs (d), indicate that little mobilisation of friction occurs in the lower half of the nail.

Similar comparisons with the fine sand tests can be made from the observations in the *shearing* stage (Figures 8.42, 8.44 and 8.46). Axial forces and radial stresses, shown in graphs (a) and (b), progressively increase with shearing but again to a lesser extent. Rigid-inclusion effects are not evident with this more granular material, (*i.e.* radial stress does not exceed applied boundary stress). The effect of arching has increased with the dense sand as can be seen in Figure 8.46b. Measured radial stresses at the interface are less than 40% of those applied at the sample boundary with very little increase throughout the whole stage.
The profiles of interface shear stress $\tau_{im}$ shown in graphs (c), follow the same trends with a front of mobilised $\tau_{im}$ moving down the nail with shearing. At the end of the stage there is a wide distribution for the loose sample (Figure 8.42c), extending over at least two-thirds of the nail length, while in the case of the dense sample (Figure 8.46c) the front remains towards the top of the sample. Its exact location is unknown as the instrumentation did not extend to the upper part of the nail.

Although $\tau_{im}$ values are lower than those for the fine sand, because of reduced axial forces, the mobilised interface friction angles, shown in graphs (d), are larger. At the end of shearing $\delta'$ at the location of the upper radial device for the loose sand is 32°. In the case of the dense sands negligible $\delta'$ values are mobilised in the lower part of the nail (reflecting $\tau_{im}$ values). This implies that soil movements along the side of the nail are negligible within the lower 200 mm of the sample, the inner core is quite inert. Values of $\delta'$ can be estimated at the upper point where $\tau_{im}$ is deduced (shown on all plots as about 270 mm), by extrapolating the radial stress curves back to the same point. Values of $\delta'$ obtained using this approach are generally about 30°, roughly as expected. $\delta'_{max}$ values from Table 8.3 are between 33° and 38° depending on the sand density and applied normal stress. After peak they reduce to $\delta'_{cv}$ values between 28° and 32°.

Movements of the sand core around the top region of the nail, shown in graphs (e), indicate similar trends as for the fine sand, decreasing as density increases. In the dense sand (Figure 8.46e) the movements are sub-millimetre up to the end of shearing. According to the interface shearbox results, these are sufficient to mobilise $\delta'_{max}$ though only at the top of the sample, but not $\delta'_{cv}$. Values of $\Delta_{core}/\Delta_{ave}$ are about four times greater in the case of the medium dense sand (Figure 8.44e), in this state $\delta'_{cv}$ might be approached towards the top of the sample.

Movements of the sand core at the positions where $\delta'$ values are determined (i.e. midway along the nail) can be estimated by assessing the displacements necessary to mobilise the friction angle $\delta'$ deduced from $\tau_{im}$ and $\sigma_{r,im}$. They are obtained from the interface shearbox $\delta'$ versus displacement curves (see Figures 8.6c and 8.7c) using the appropriate density and normal stress. This exercise was performed for the medium dense Ham River sand. $\Delta_{core}/\Delta_{ave}$ values midway along the nail are estimated to be about one tenth of those at the top for the latter part of the shearing stage.

Coarse sand

Results from the instrumented nail during the compression and shearing stages of the coarse sand tests are shown in Figures 8.47 to 8.52. Trends very similar to those of the fine sand and HRS are evident. Interface shearbox tests were not performed with the coarse sand for the reasons given in Section 8.2. However, the results of Everton (1991), presented in
Section 8.2.5, are used as a guideline. His work shows that $\phi'_{\text{max}}$ values are considerably higher than those for HRS, while $\delta'_{\text{max}}$ values are lower, only 40 to 50% of the resistance obtained from particle interlocking in the sand-sand tests being mobilised along the interface. This information provides a guide to the approximate values of $\delta'_{\text{max}}$ to expect, a value of about $30^\circ$ is estimated, given the differences in the interface materials and the higher $\phi'_{\text{max}}$ values recorded.

Measurements made with the radial stress devices are not so reliable with the coarse sand. The instruments under-register most of the time, probably as a consequence of the large grain size relative to device dimension (i.e. that of the curved area of the segmental beam as discussed in Chapter 7 and in Section 8.4.6).

Results from the isotropic compression stage, shown in Figures 8.47, 8.49 and 8.51, indicate that the nail goes into compression by slightly lower amounts than for the fine sand and HRS. The final compression is achieved after similar $\Delta_{\text{comp}}$ displacements of the top of the sample (less than 1 mm). Interface shear stress profiles derived from the axial force values, shown in graphs (c), indicate that most of the friction mobilised is towards the top of the nail.

The radial stress measuring devices indicate that values of $\sigma_{r,\text{int}}$ increase progressively during this stage but by very small amounts, see graphs (b). Maximum values reached are about 30 kPa for the loose sand, decreasing to about 10 kPa for the dense. The profiles throughout are almost uniform.

A greater degree of arching might be expected with the more granular coarse sand, which could significantly reduce radial stresses on the nail. However, the very high, slightly unrealistic angles of interface friction, shown in graphs (d), suggest that other factors are contributing to the low measured values of $\sigma_{r,\text{int}}$. The possibility of high values of $\delta'$ being generated at low stress levels during deposition of the sand and flushing the radial membrane was discussed for the case of the fine sands. Similar effects were seen with the HRS tests. In the case of the coarse sand, values greater than $70^\circ$ are indicated at the start, regardless of density, i.e. roughly twice those recorded by Everton (1991) from his tests. The low stress levels might partly explain these values, but the supposition is that the radial stress measuring devices are under-registering and not reading accurately because of the coarse sand grain size.

The effects of arching have been observed with the finer grained soils. These effects increase with grain size and density, and so they are certainly partly responsible for the low radial stresses shown in graphs (b) of the coarse sand test results. As isotropic compression progresses, the calculated $\delta'$ values reduce because of the reversal in sense of the interface shear stresses from that induced prior to the start of the stage.

Further evidence concerning the under-registration of the radial stress measuring devices in the coarse sand can be deduced from consideration of the results from the fine sand and HRS.
It was observed in the previous two sections that as compression stresses increase in the fine sand and HRS tests, the calculated $\delta'$ values reduce. In the loose fine sand tests, where arching is least expected and $\tau_{int}$ can develop most readily, $\delta'$ at the end of the stage is about $12^\circ$, i.e. about a third of $\delta'_{max}$. The results from tests on the fine sands indicate that the zone adjacent to the nail in which friction is mobilised always starts at the top of the sample. Propagation of interface shear stress down the nail is steadily inhibited as sand density and grain size increase. The implication of the above is that the possibility of mobilising $\delta'$ values, at the position of the upper two radial devices, greater than about a third of $\delta'_{max}$ is unlikely by the end of the compression stage regardless of density or grain size.

Using one third of the $\delta'$ value assigned to the coarse sand from Everton's test data, i.e. $\delta'_{max} = 30^\circ$, and the $\tau_{int}$ values at the end of compression, estimates can be made of a lower limit of the radial stresses acting on the nail. Values of radial stress approximated using this approach are greater than 140 kPa, far in excess of those measured by the radial stress devices in the coarse sand. The conclusion is that the devices are not very responsive with the coarse grain sized particles (recalling that for coarse sand $D_{50} = 1.40$ mm, c.f. width of segmental beam = 6.25 mm).

Observations during the shearing stage, shown in Figures 8.48, 8.50 and 8.52, are similar to those for the fine sand and HRS. Axial forces and radial stresses acting on the nail, shown in graphs (a) and (b), progressively increase with shearing but to a lesser extent because of the larger grain size. As discussed above, responses of the radial devices are particularly low and it is not possible to assess trends in the distributions. In the case of the loose sand, values of $\sigma_{int}$ increase to just over 100 kPa by the end of the stage, while for the dense sample there are negligible increases, all three devices indicating values of about 10 kPa.

Interface shear stress distributions during shearing, shown in graphs (c), only propagate fully to the location of the instruments in the case of the loose sand (Figure 8.48c). At increased densities there is negligible change, the distributions of $F_{as}$ being almost uniform throughout. Profiles of $F_{as}$ and $\tau_{int}$ developed in the loose sample are still similar to those of the idealised model shown in Figure 8.34h.

In the graphs (d), the under-registration of radial stress is again evident from the $\delta'$ values calculated from the measured value of $\sigma_{int}$. By the end of the shearing stage in the loose sand test, the front, behind which interface shear stresses are mobilised, has extended to the nail instruments. The values of $\delta'$ calculated at both locations have reduced to more realistic levels, that nearest the base to a steady value of about $35^\circ$. However, even at this stage the $\delta'_{max}$ values are still high.

In the case of the dense sample, the degree of under-registration of the radial devices
remains constant throughout the shearing stage (Figure 8.52b). The interface shear stresses generated are very close to the top of the nail. The fact that the sample was sheared to a stress ratio, $\eta$ of about $-0.86$ with $\Delta_{\text{ave}} = 3$ mm (curve 5) indicates that the sample itself is providing most of the resistance to shearing as the contribution from the nail is minimal.

Results from the nail instrumentation during the coarse sand tests are not discussed further because of the uncertainties with the measured radial stresses.

Output from the nail instrumentation is expressed in different forms in the next section to seek and identify additional characteristics.

**8.4.4 Continuous interlace reactions during shearing**

In the previous section the parameters obtained from the nail devices are plotted at their respective distances along the nail and joined up to give a profile of force or stress at any one time during shearing. The development of the parameters is shown by using a series of profiles at selected times during shearing. The output data in this section are presented as a continuous series of readings throughout the stage (only the shearing stage is covered). Although interpretations of mechanisms within the sample are not so easy to identify with this type of presentation, trends between the different sand types and densities at different stages of shearing do emerge.

The first series of graphs expressed in this way are of axial force $F_a$ and radial stress $\sigma_{r,\text{net}}$ versus stress ratio. The data are grouped first in terms of grain size (Figures 8.53 and 8.55 for axial force and radial stress respectively) and then density (Figures 8.54 and 8.56).

Considering the axial force plots shown in Figure 8.53 and 8.54, several trends emerge. Axial force in the devices develops steadily from the start of the shearing stage. It can be seen from Figure 8.54 that the rate of increase of axial force seems to be predominantly density related in the initial stages up to about $\eta = -0.4$ (corresponding roughly to $e^* \approx 0.5\%$). The loose samples in particular, shown at the top of Figure 8.54, have a remarkably consistent rate of increase. The rate of load development decreases with increasing density; also as the samples become more dense the influence of grain size starts to become evident. The nail in the dense coarse sand has the slowest rate and the lowest loads by the end of the shearing stage.

In each graph shown in Figures 8.53 and 8.54, there are three lines for each test representing the outputs from the three devices. The path where the three lines are roughly co-linear (i.e. having the same force in each) represents the stage when friction is being developed along the upper part of the nail above the instrumented length. Once the 'front' reaches the first (upper) instrument, the lines start to diverge because the force developed in each is that controlled by the differing lengths of nail above them where friction is gradually being mobilised. This mechanism has already been discussed in the previous sections. For the loose
and medium dense samples the force in the nail increases with depth towards its fixed base, therefore the solid lines representing the upper device have the lowest load in them.

In the case of the dense sands, although there is little variation between device outputs for each test, the upper device always has the greatest load in it, regardless of grain size (see lower plot of Figure 8.54). Divergence of nail outputs in each of the dense samples does not occur throughout the shearing, confirming that the mobilised ‘front’ has not travelled far from the top of the nail. It has perhaps just reached the upper device at the end of the test on dense fine sand. The increased load towards the top of the nail over the instrumented length, at roughly mid-section, might be explained using the final idealised case shown in Figure 8.34h. During increments 16 and 17 there are tensile and compressive forces in the nail with a varying distribution. Perhaps because of the limitations of particle movement and consequent interface shear stress development, the force in the nail is for most of the tests fluctuating between compression and tension.

It can be seen from the upper plots in Figure 8.54 that once the force outputs start to diverge, the rate of increase of \( F_{ax} \) decreases quite dramatically but does still continue. Either further frictional mobilisation is taking place or the normal stresses on the nail, represented by \( \sigma_{r,\text{int}} \), are slowly increasing.

The results from the radial stress devices during the course of shearing, shown in Figures 8.55 and 8.56 where the results are grouped in terms of grain size and density respectively, can be used to investigate corresponding variations of \( \sigma_{r,\text{int}} \). The mechanics of what is happening in terms of radial stresses is not easy to interpret and was not covered within the idealised models. Despite this lack of an overall understanding various trends and features of behaviour can be identified.

In Figure 8.55 it is very clear that as the grain size increases so the radial stresses decrease. There is not such a strong trend with density (Figure 8.56). However, the reduction might also be partly caused by the under-registration of the instrumentation, as discussed in the previous section.

The stress ratio at which the front of mobilised friction reaches the instrumented length of the nail has been clearly identified from the axial force plots discussed earlier. It can be seen that at roughly corresponding values of \( \eta \) \((\eta = -0.4)\), the radial stresses start to increase; prior to this point they are generally almost constant. This confirms that a greater transfer of radial stress from the applied boundary stresses only takes place once the sand particles reorientate themselves during interface shearing, by breaking down arching mechanisms. The rate at which the radial stresses increase is density related; higher rates being associated with looser samples. However, this observation might be influenced by the fact that the front did not reach the
instrumented length in the dense samples.

In the case of the loose samples (upper plot in Figure 8.56) a reduction in the rate of radial stress increase occurs after further shearing, at stress ratios between \( \eta = -0.6 \) to \(-1.0\). It is possible that the initial increase at \( \eta = -0.4 \) occurs when the sand immediately around the instrument reorients itself as the front reaches the level of the instruments and stress rotations start to occur in earnest. Prior to this point the stresses acting are those imposed during the isotropic compression stage. The onset of stress rotation is associated with contractant volume changes (see Menkiti, 1995 and Zdravkovic, 1996). Once these are complete the rate of increase of radial stress then drops off. These ideas and the effects of arching are discussed further in Section 8.4.6.

The final method used for presenting the nail data is by interface 'stress paths' in the form of normalised plots of interface shear stress \( \tau_{im}/\sigma_{r_{int}} \) versus radial stress \( \sigma_{r_{int}}/\sigma_{r_{int}} \), where \( \sigma_{r_{int}} \) is the radial stress at the beginning of the stage. This style of plotting is used at Imperial College for presenting the results from instrumented pile tests (see Lehane, 1992 and Chow, 1997). Values of interface friction \( \delta' \) can readily be determined from the ratio of \( \tau_{im}/\sigma_{r_{int}} \) at any stage of the test.

Results from the shearing stage of the model nail tests are given in Figures 8.57 and 8.58 for the fine sand and HRS respectively: those for the coarse are not presented because of the very low radial stresses measured. In each figure, data in the upper graph relate to measurements from the upper radial stress device, i.e. at a position just above the midpoint of the nail, and in the lower graph the corresponding data from the middle device are given. As axial force was not measured below the lower radial device, there are no corresponding values of \( \tau_{im} \) at its position. Superimposed on the plots are lines of constant \( \delta' \) from which the magnitude of friction mobilised can be assessed. The same stress paths are presented individually with values of axial strain \( \varepsilon_a \) marked against them in Figure 8.59 and 8.60.

The lack of mobilisation of interface shear stress in the case of the dense sands, despite the overall sample strains, is quite evident. So also is the fact that mobilisation of interface friction in the loose and medium dense samples occurs first at the upper device. In the case of the dense samples considerable shearing of the sample occurs (given in terms of axial strains) before the stress paths of the lower devices start to reverse and follow the same sense as the upper devices. Their initial paths indicate positive shear stress changes even though the axial force in the nail is decreasing (refer lower plot in Figure 8.54). By the end of the shearing stage each pair of paths are starting to converge (see lower plots of Figures 8.59 and 8.60). This trend in behaviour is clearer with this style of presentation than that given in Figures 8.40 and 8.46.
observations to be made on the behaviour of the sand within the sample and at the nail interface. In the previous discussions brief references have been made to the effects of rigid-body inclusions, arching and restrained dilation. These phenomena are now addressed separately and in detail.

8.4.5 Rigid-body inclusion effects

The subject of rigid body inclusion effects, within the context of the boundary conditions of the test apparatus used in this study, arose when considering cell-action effects with reference to the compliance of the instrumentation for measuring radial stress. In Section 7.1, elasticity theory was used to quantify these effects by considering closed-form solutions for the case of a long thick-walled cylinder (see also Appendix 7.14). Using the appropriate radial dimensions and realistic moduli values for the steel nail and soil sample, the solutions, implemented using the principle of superposition, indicate that when a boundary stress of 250 kPa is applied to the sample, the stress transmitted to the nail interface (i.e. \( \sigma_{rm} \)) is about 333 kPa.

In the context of this study, rigid-body inclusion effects lead to radial stresses at the nail interface being generated that are greater than those applied at the boundary of the sample, as in the case of the loose fine sand. These effects arise because of differences in relative stiffness and dimensions of the steel nail and soil sample. In Appendix 8.1 a physical explanation for this phenomenon is formulated.

8.4.6 Arching mechanisms

The phenomenon of arching is usually associated with granular materials (although it can also occur in elastic continua). It is a mechanism where particles interlock in a preferred orientation within a soil mass, forming a band or zone of increased stress level and density. The mechanism is generally induced by flow or reorientation of particles, often as a result of stress changes. The ‘band’ formed acts as an arch, transmitting stresses applied to it elsewhere in the soil mass or to external rigid boundaries.

An understanding of arching mechanisms is necessary for more realistic analyses of engineering cases such as silo and piling design. It is discussed extensively by Terzaghi (1943) who devotes a chapter to the subject and gives the following definition.

If one part of a support of a mass of soil yield while the remainder stays in place the soil adjoining the yielding part moves out of its original position between adjacent stationary masses of soil. The relative movement within the soil is opposed by a shearing resistance within the zone of contact between the yielding and the stationary masses. Since the shearing resistance tends to keep the yielding mass in its original position, it reduces the pressure on the yielding part of the support and increases the pressure on the adjoining stationary part. This transfer of pressure from a yielding mass of soil onto adjoining stationary parts is commonly called the arching effect, and the soil is said to arch over the yielding part of the support. Arching also takes place if one part of a yielding support moves out more than the adjoining parts.
In this section the phenomenon of arching around the nail is discussed. Its relevance to this study is best illustrated by discussion and example in the context of the experimental boundary conditions and results. The following four possible causes of arching and the resulting effects are:

1. that induced by the method of placement and densification;
2. that arising from the radial stress application from the outer boundary during compression,
3. that induced as a consequence of principal stress rotation in the immediate vicinity of the nail during shearing against the interface, and;
4. that occurring in the vicinity of the segmental beams of the radial stress measuring devices.

Understanding the mechanisms resulting from the first three causes is necessary for a realistic interpretation of the test results, as they control certain aspects of the behaviour of the nail-soil system. The fourth cause is relevant to the measurement of radial stress, its occurrence is directly related to the compliance of the segmental beams and is therefore also associated with cell-action effects.

**Cause 1**

The first possible cause of arching around the nail arises from setting up the sample. Initially a loose condition is achieved by releasing the sand rapidly, from a filled tube within the cell chamber, with the nail in place axially (see Section 6.3.3). At this time the radial membrane is held hard against the perspex of the cell by vacuum: the sand is therefore confined within two rigid circular boundaries (the cell and the nail). Particles flow down and spread outwards towards these boundaries from which they rebound. It is very possible that this behaviour will give rise to annular zones of varying relative density, as shown in Figure 8.61. For the tests in a loose state the sample is loaded in this condition, but for the medium dense and dense samples further densification is achieved by tapping the base of the chamber. The radii of the densified zones are probably controlled by the sand density and grain size.

**Cause 2**

Arching during compression is the next case to consider. As radial stress is applied to the boundary the particles move inwards radially. Towards the centre of the sample (considering a section through it) there is less freedom for these movements to occur, and the grains tend to lock up circumferentially. As a consequence, the magnitude of the hoop stresses starts to increase above those of the radial stresses. Once an arched structure (an annulus of soil or hoop) is initiated, progressively greater degrees of the applied stress are diverted into circumferential stress and less to radial stress within the interior of the hoop. Again the critical radii at which
the onset of this mechanism occurs is controlled by grain size and density (the dimensions of the nail and cell being fixed). The process occurs because of the circular boundaries which help induce these hooped structures. Arched structures that exist from the setting up procedure make the sample more prone to this effect. The processes involved with this type of arching are shown in Figure 8.62.

The magnitude of circumferential stress that can be sustained by the arches is dependent primarily on the internal shearing resistance of the sand and the dimensions of the arch (i.e. inner and outer radii). To illustrate this, equilibrium equations for an arched structure within the sample are given in Appendix 8.2, along with an example of the relative values of radial and circumferential stress (i.e. $\sigma_r/\sigma_w$) resulting from some typical conditions. The sand strength is taken to be governed by its passive resistance with $\sigma_w$ being set to $K_p\sigma_r$. Using this assumption, the formulation indicates that considerable radial stress gradients can be sustained either side of an arch.

**Cause 3**

A further cause of arching arises indirectly during the shearing stage, when the stress directions induced during compression, rotate as a consequence of change in stress path. Recent studies into the effects of principal stress rotation and anisotropy (Menkiti, 1995 and Zdravkovic, 1996) show that soils subjected to this action undergo contractant volume changes. This is possible in the case of dense samples because of the anisotropy inherent in most granular materials, volume reductions take place in the weaker plane. In the vicinity of the nail-soil interface, severe but gradual stress rotations occur during shearing, thus causing a reduction in the soil volume immediately around the nail. However, elsewhere within the sample, rotations take place much more quickly as there is no particle restraint, as exists against the nail. The relatively denser soil existing around the contracting zone can then develop into an arched structure and progressively smaller amounts of radial stress are transmitted to the nail. This process could also take place during the compression stage, although to a lesser degree as stresses after sample deposition are low and have not been induced in a particular direction.

The effect of the arching induced by localised contraction, combined with further shearing in axial extension, could result in stress levels within the inner zone decreasing to the point where elevated values of angle of internal shearing resistance, $\phi'_r$, are generated (see Hellings 1989 and Hellings and Burland, 1997). If similar effects occur for interface friction angles, this could partly explain the very high $\delta'$ values observed during the shearing stage of the coarse sand in a dense state.

The combined effect of causes 2 and 3 could be responsible for the very high stress ratios observed during the shearing stages of the tests when considering the boundary stresses
and strains, most of the samples had $\eta$ values between $-1.3$ and $-1.4$ by the end of the stage. According to the critical state equation plotted in Figure 8.15, this would imply that values of $\phi'$ between about $55^\circ$ to $65^\circ$ were achieved. The result might arise from conditions imposed by the apparatus but might also be explained by a redistribution of stress induced by arching. During shearing in axial extension the applied radial boundary stresses act as the 'driving stresses'. If they are not transmitted into equal components of radial and circumferential effective stress as would be expected in the case of an isotropic elastic continuum, a greater circumferential stress component is induced at the detriment of the radial stress. Consequently the calculated stress ratios, which are based on the measured applied boundary stresses, are overestimated.

**Cause 4**

The fourth incidence of arching occurs locally in the vicinity of the segmental beam of the radial stress measuring devices. As stresses are applied to the beam, its deflection can lead to arched structures forming around it (radially and longitudinally as shown in Figure 8.63). This case is similar to the yielding strip scenario discussed by Terzaghi (1943). The likelihood of it occurring increases as grain sizes become larger. This form of arching mechanism in combination with cell-action effects is mainly responsible for the under-registration of the radial stress devices in the coarse sand, especially in a dense state. The relative sizes of the grains to the segmental beam for the three sands tested are shown in Figure 8.64, reasons for the unrealistic responses with the coarse sand become immediately obvious. Apart from the arching effect there are very few points of contact between the grains and the curved surface of the beam (there are typically less than ten grain contacts across a section).

The above discussions on the causes and forms of arching within the experimental apparatus indicate that the first three causes affect the behaviour of the sample as a whole and the fourth, the measurement of radial stress. Because the first three causes have a similar effect during the shearing stage, they cannot be quantified individually. Arching around the segmental beam only has a major effect with the coarse sand grain size.

Another way of considering the phenomenon of arching, in the context of the experimental results, is as a 'negative rigid inclusion effect' or 'soft inclusion effect' because the reverse process occurs, to that described in Section 8.4.5 where rigid inclusion effects are described.

In Figure 8.65, an assessment of arching during the compression stage is made by plotting measured radial stress on the nail, $\sigma_{\text{me}}$, normalized by the applied boundary stress $\sigma_{\text{app}}$, against $\sigma_{\text{app}}$. It should be noted that the influence of rigid-body inclusion effects and restrained dilation, that tend to cause increases in radial stress at the interface, are also incorporated in
these plots. Results for the three grain sizes from each radial stress measuring device are compared in graphs a to c. The quantity $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ represents the proportion of radial stress applied at the sample boundary that is transmitted through to the nail, as $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ decreases this implies that the degree of arching increases. When $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ is zero there is no transfer of applied boundary stress to the nail; when it is at unity there is no arching effect (in the absence of rigid-body inclusion effects or restrained dilation). The effects of grain size and density are very clear: there is a greater degree of arching as density and grain size increase. The inception of the mechanisms occurs predominantly at lower stress levels ($\sigma_{r,\text{app}} < 50$ kPa) after which values of $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ decrease more slowly.

In the results from the case of the fine sand shown in Figure 8.65a, values of $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ from all tests at the beginning of shearing were greater than 0.8, with some of the devices in the loose and medium dense sand tests registering values in excess of unity, probably because of rigid-inclusion effects. The response during the dense sand test from the upper device, where displacements are greatest, indicates measured values greater than twice those applied. This is thought to be from the phenomenon of restrained dilation, which is discussed in the next section. The effect is not seen clearly elsewhere in the test programme because arching mechanisms are thought to have prevented its development. The effect diminishes rapidly as the compression stresses increase and particles start to lock up. The relevance of the position along the nail at which arching takes place is discussed in Section 8.4.3 (under fine sand), it is not discussed further here.

The trend of behaviour in the medium grained Ham River sand is well defined and the paths are within a narrow range, as shown in Figure 8.65b. Values of $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ decrease by 2 to 4 times from the start to end of compression. In the case of the coarse sand tests, values of measured radial stress are generally less than 50% of those applied, and by the end of the stage they are all less than 20%, the dense less than 10% as presented in Figure 8.65c. Although the coarse sand has a greater propensity to arch as indicated by the results, the degree indicated is certainly also affected by the local arching around the instrument, described as cause 4 above.

The conclusion from the compression stage is that for the fine sand and HRS, arching caused by the setting-up procedure is not as significant as that from the compression itself. The effect increases with grain size, although for the coarse sand it is not as severe as indicated, the results in this case reflecting an under-registration of the devices. As would be expected, the tendency for locking up of the particles to form hooped structures increases with density.

Results from the shearing stage are presented in a different form as shown in Figure 8.66. The degree of arching is still represented by $\sigma_{r,\text{inf}}/\sigma_{r,\text{app}}$ but plotted against average grain size on a logarithmic scale. A separate graph is given for each device (upper, mid and lower)
and three lines are shown for each density representing conditions at the beginning of shearing and then after 2 mm and 6 mm axial displacement of the upper sample surface.

In the discussion of the causes of arching given above, the third is attributed to stress rotation during shearing. The results presented in Figure 8.66 indicate that, despite this effect, the general trend in behaviour is for a breakdown of arching during shearing. The degree of breakdown generally increases with decreasing grain size and density. The particles of the coarse sand in a dense and medium dense state remaining essentially locked-up throughout the entire shearing stage. Many of the lines, spanning the three grain sizes tested, are linear in the semi-log plots. The lines relating to the dense and medium dense samples converge to within a narrow range of grain size of 2-3 mm when extrapolated back to the $\sigma_{r,in}/\sigma_{r,sp} = 0$ axis. This implies that there would be no transmission of applied boundary stress to the nail for samples with $D_{50} > 2$ to 3 mm for the dimensions and conditions used. The whole system (i.e. along the entire nail length in the active zone) would be locked-up, even at moderate degrees of shearing. However, scaling-up nail and grain size dimensions pro rata indicates that the effect would only occur in coarse gravels for a 150 mm diameter nail (as the scaled-up $D_{50} > 50$ mm).

There is greater freedom for arching to break down in the loose samples during shearing, regardless of grain size. This effect is seen at each instrument position. The lines relating to the different degrees of shearing do not converge with increasing grain size. Extrapolating the lines back indicates that, after 6 mm axial displacement, the grain structure would lock up with particle sizes of $D_{50} = 5$ to 6 mm. However, judging from the trend of the lines, the arched structure would break down after further shearing, even with grains of this size.

The effect that arching has on radial stresses acting on the nail-soil interface is clearly illustrated in this section and in 8.4.3 where stresses and displacements at the nail interface are discussed. Four causes of arching have been identified relating to different stages of the test, and to under-registration of the radial stress measuring devices. Although these specific causes may not occur in field soil-nails, other arching mechanisms might be induced by other means. Arching is therefore an important factor to consider in design of soil-nails in granular materials as the radial stresses control levels of mobilised interface shear stress and axial force which are relied on for augmenting overall stability. The degree of arching is controlled by density and grain size; predominantly the former. Arching in loose soils breaks down with shearing even for coarse grain sizes whereas in the case of dense sands, it appears that the arching structure remains intact for the larger particle sizes regardless of the degree of shearing.

8.4.7 Restrained dilation

Restrained dilation, sometimes termed constrained dilation, is included here as the third phenomenon that can influence the behaviour at the nail-soil interface. It is associated with
granular materials, particularly those in medium dense and dense states that have a propensity to dilate.

The effect of restrained dilation is best explained using the boundary conditions of the direct shearbox apparatus as an example. Consider a dense sand being sheared under an applied normal stress, $\sigma_n'$. Within the shearing zone, interparticle contact stresses develop up to the point when particles start to slide. The dense packing of the grains inhibits excessive sliding movements until particles start to ride up over each other. During this process there are dilatant volume changes within the shearing zone which are resisted by the normal stress $\sigma_n'$. Because $\sigma_n'$ is usually applied as a dead load acting on the top cap of the shearbox its value remains constant. If higher values of $\sigma_n'$ are applied, the shearing stress $\tau$ required to overcome $\sigma_n'$ for dilation to occur increases but the ratio of $\tau/\sigma_n'$ remains essentially constant. The conditions of the test are therefore those of free dilation.

If the same test were performed without allowing dilation (i.e. by restraining vertical movement of the top cap) by increasing the normal stress as shearing progressed, values of $\sigma_n'$ and consequently of $\tau$ would rise far in excess of those in the free dilation situation. This case constitutes one of restrained dilation. In the limit, if no dilation were allowed and the particles were in their densest state, the only way that shearing could continue would be by shearing through the grains themselves.

Similar effects can be expected during shearing against an interface, although the degree of dilation that occurs is dependent on the relative size of the grains to the asperities of the contact surface. The shearbox testing of the fine sand and HRS against the nail interface material (see Section 8.2.4) showed that for these cases, the angles of dilation generated are comparable to those from the sand-sand tests (though slightly lower: compare $D_i$ and $\psi_{max}$ values from corresponding reference and interface tests summarised in Tables 8.2 and 8.3).

The implications and benefit of the effects of restrained dilation are clear. If a nail were installed in a dense sand without excessive disturbance, and during interface shearing the dilation trying to develop were inhibited by the surrounding soil mass, the normal (radial) stresses generated on the interface would far exceed those anticipated from consideration of initial in situ stresses. The effect has been observed during pull-out tests both in the laboratory and in the field (e.g. Plumelle, 1979; Wernick, 1977).

The subject of restrained dilation has been studied experimentally and numerically by several researchers at the University of Grenoble (e.g. Boulon et al. 1986). Work carried out within the former Soviet Union is described by Sobolevsky (1995) who writes comprehensively on the subject, based on laboratory studies. The tests described by both groups take the form of direct shearbox tests where the stiffness of the restraining platen (where $\sigma_n'$ is usually
applied) can be varied. Their results and conclusions are very similar although their methods of analysing the problem differ.

Boulon et al. (1986) compare their test results to those from numerical studies which are of two types: finite element analyses applied with different models for the interface behaviour (conditions of fixed volume and of normal stress) and a simplified cavity expansion type analyses employing a pressuremeter modulus.

Attention is focused here on the work of Sobolevsky (1995) who describes conditions in terms of a Coulomb type model that takes into account restrained dilation. The additional component of normal stress induced by restrained dilation $\sigma_d'$ is incorporated into the failure criterion as given in equation 8.10 and shown in Figure 8.67.

$$\tau_u = (\sigma_{so}' + \sigma_d') \tan \phi'$$  \hspace{1cm} (8.10)

where $\sigma_{so}'$ is the normal stress acting prior to shearing. The magnitude of $\sigma_d'$ is directly proportional to the stiffness of the medium providing restraint to volume changes (e.g. the surrounding soil in situ or the apparatus platen in laboratory testing). Sobolevsky defines this quantity as the coefficient of elastic resistance (or of uniform compression) given by:

$$K_v = \frac{\Delta \sigma_d'}{\Delta \delta_u}$$  \hspace{1cm} (8.11)

the ratio of incremental dilational stress $\Delta \sigma_d'$ to the corresponding displacement $\Delta \delta_u$ (the subscript $\psi$ is used here to avoid confusion with bulk modulus $K$). The displacement $\Delta \delta_u$ is the broadening (or narrowing) of the shear band in the same sense in which $\Delta \sigma_d'$ acts. $K_v$ is empirically related to conventional elastic parameters, for axi-symmetric conditions, by:

$$E = (1 + \nu) r K_v$$  \hspace{1cm} (8.12)

where $r$ is the width of the shear surface. When using equation 8.12 values of $E$ and $\nu$ have to be selected: Sobolevsky argues that for granular materials truly elastic behaviour only occurs at very small strains but that these are adequate for the development of $\Delta \sigma_d'$. The values of $E$ that he quotes, range from 100 to 300 MPa (covering a series of compression tests on sands of varying density and grain size) the magnitude of these values suggests very low strain levels.

Values of $\tau_u$ are obtained from dilatometric apparatus which allow the stiffness of the medium beyond the shearing zone to be controlled (i.e. $K_v$). Results are plotted as lines of $\tau_u$ versus $\Delta \sigma_{so}'$ for different $K_v$ values, in those presented there is an increase in $\sigma_d'$ with $K_v$ as would be expected but also a corresponding decrease in $\phi'$. This implies a similar relationship
to Mohr-Coulomb but with varying angles of internal friction, which are dependent on the stiffness of the outer soil mass. Idealised curves showing this effect and examples from the test data in are given in Figures 8.67 and 8.68 respectively.

A convergence of the lines occurs at $\sigma_{0r}$, the critical initial normal stress, which corresponds to the conditions that describe critical density. The existence of this point at $\sigma_{0r}$ indicates that a certain initial normal stress is required to maintain a condition of no volume change (i.e. suppress dilation). The lines imply that even with no normal stress, finite values of shear stress can be generated under conditions of restrained dilation. In Figure 8.67 the idealised line for restrained dilation is extrapolated back beyond the $\sigma_{0r} = 0$ axis and the value of $\sigma_{0r}$ marked.

Sobolevsky (1995) concludes from his experimental results that the magnitude of the increase in normal stress from restrained dilation, $\sigma_{d'}$, is directly related to the stiffness of the soil mass outside of the shearing zone, the soil density and grain size and the strength of the minerals of which the grain are composed. The Mohr-Coulomb relationship generally adopted is one specific case of equation 8.10 for the condition of free dilation when $\sigma_{d'} = 0$.

The only instance of restrained dilation witnessed in this study occurred at the beginning of the compression stage for the dense fine sand when radial stresses in excess of twice those applied at the boundary were measured. The early onset of circumferential arching within the samples, particularly evident in the dense samples, is thought to have inhibited this effect.

The three phenomena described here are discussed further in the next section where they are integrated with the other methods of expressing the test results. Their relevance to analysis and design in a general sense and within the context of what has been observed in this study are covered in Chapter 10.

8.5 SUMMARY AND CONCLUSIONS FROM THE SAND TESTS

The main purpose of this chapter has been to present the results from a series of triaxial element tests that were performed to model conditions for a single nail being indirectly loaded by stress relief of the surrounding soil mass. The path followed during testing was isotropic compression followed by shearing in axial extension. Tests were performed on dry fine sand, HRS and coarse sand in loose, medium dense and dense states. Comparative reference tests with no nail were also performed.

Prior to discussing the main conclusions, the results from a series of shearbox tests carried out to obtain interface shearing parameters are summarised.

The triaxial test results have been analysed considering conditions externally in terms of applied boundary stresses and the resulting strains and internally at the nail interface by
means of the instrumentation incorporated into the model nail.

In assessing external conditions the sample responses between corresponding nail and reference tests are compared. The results have been expressed in terms of strains, stiffnesses, differences in strains and differences in work expended during shearing of the samples. These quantities have been plotted against stress ratio, which was chosen as a common parameter between nail and reference tests. Work has been adopted because it incorporates components of both volumetric and distortional stress and strain.

Assessment of internal conditions is clearly only relevant to the nail tests, the results from the nail instrumentation have led to the identification of various mechanisms and phenomena which are described prior to summarising the results.

So far the two aspects of external and internal behaviour have been discussed separately. The intention in the following paragraphs is to summarise the comparative reference and nail sample test results from the isotropic compression and shearing stages and then explain them in terms of the soil behaviour in the vicinity of the nail.

Shearbox test results

Direct sand-sand and interface shearbox tests were undertaken to characterise the shearing behaviour of the fine sand and HRS. Relationships with applied normal stress $\sigma'$ are evident: generally peak values of $\phi'$, $\delta'$ and $\psi$ increased with decreasing $\sigma'$ and were higher for HRS. The degree of contraction of the loose samples increased with applied $\sigma'$ and was greater with the fine sand. Mobilisation of the peak values of $\phi'$, $\delta'$ and $\psi$ occurred at similar displacements for both sand-sand and interface tests. Peak and constant volume values of $\delta'$ were only marginally lower than those of $\phi'$ (i.e. less than 3° for most cases), post-peak decreases in $\delta'$, for the dense samples, occurred in a more ductile manner than those of $\phi'$. Interface values of $\psi$ were lower than those of the reference tests and exhibited a more brittle behaviour, particularly with the HRS.

The results from these conventional shearbox tests are summarised in Tables 8.2 and 8.3. They act as a base reference in the interpretation of results from the more complex conditions of the triaxial reference and nail tests performed.

Phenomena influencing sample behaviour

Three phenomena have been described which can have a significant influence on conditions at the interface and consequently the overall sample behaviour.

Rigid-body inclusion effects are usually associated with elastic continua, where two materials have different relative stiffnesses. In the context of the test arrangement, stresses applied at the sample boundary would be expected to be magnified at the nail interface because of the higher stiffness of the nail.
Arching results from granular interlocking between grains which tend to cause non-uniform distributions of stress. In the nail tests, annular rings of varying relative density are thought to have been formed around the nail during sample preparation. As the sample is isotropically compressed, the relative density of the rings increases until eventually the grains lock up. As a result, greater degrees of applied radial stress are transferred into circumferential stress and the interface radial stresses on the nail are considerably reduced from those applied at the boundary.

Restrained dilation arises when interface shearing of dense granular materials results in a dilatant behaviour but the freedom for the soil to dilate is inhibited by high stresses or dense packing around the shearing zone. As a consequence high normal stresses can be generated. This phenomenon was not observed during shearing stage, it is believed to have been inhibited by the effects of arching.

**Isotropic compression**

The results from the isotropic compression stage indicate that axial and radial strains increased with decreasing density, and hence so also did the volumetric strains. In many cases strains in the nailed samples were greater than those of the reference samples.

In general, the samples were stiffer axially than radially, suggesting that an anisotropic structure was induced during sample preparation.

Results from measurements with the instrumented model nail show that in all cases the nail went into compression. The distribution of axial force over the instrumented length (middle third) was uniform in most cases, indicating that mobilisation of interface shear stress only occurred towards the free end of the nail. The magnitudes of these shear stresses were similar regardless of grain size or relative density.

Radial stresses increased with application of boundary stresses by different degrees according to the relative density of the sand and the grain size. Towards the base of the *loose fine* sand almost full transmission of applied stress to the nail occurred. In some of the early stages of the compression, measured values were greater than those applied as a result of rigid-body inclusion effects. The degree of radial stress transmitted to the nail decreased towards the top of the sample where the particles were more able to reorientate themselves which is thought to lead to local densification and a consequent onset of arching.

Similar behaviour was observed with the loose HRS and coarse sand, but the magnitude of the measured radial stresses decreased as the grain size became coarser.

The effect of arching is evident in the *dense fine* sand sample where radial stresses at the interface were considerably less than those applied at the sample boundary. The reorientation of particles towards the free end of the nail had the opposite effect to the case of
the loose sample. Particle movement is thought to have broken down the arching so that radial stresses transmitted to the top of the nail were greater than those lower down where a locked-up structure still existed. A much more uniform transfer of radial stress occurred with the medium dense sand of magnitude between those of the loose and dense samples.

The effect of grain size is again evident from the results from the dense sand tests on the HRS and coarse sand. Similar trends were observed as for the fine sand, but radial stresses decreased with coarseness of grain size. In the case of the dense coarse sand the calculated values of interface friction angle were unrealistically high, this is thought to have resulted from the radial stress measuring devices under-registering with the coarse size grains.

**Shearing in axial extension**

Conditions of stress relief at the face during a nailed-soil excavation have been simulated by shearing in axial extension. The 'external' measurements at the sample boundaries during this stage indicate that the axial stiffness of the nail test samples were generally lower than those of the reference samples up to axial strain levels of about $-0.1\%$; by about $-1\%$ they were slightly stiffer or more or less equal. Generally axial stiffness decreased as grain size increased; no definite relationships between density and stiffness were observed.

The magnitude of axial stiffnesses of both reference and nail samples was about a third of values obtained from conventional triaxial tests under similar conditions of axial extension, although the tests with which comparisons are made were sheared undrained. The difference is believed to result predominantly from the flexible sample boundaries (average displacements were used in the determination of strains), and the fact that the sands were tested dry.

The volumetric strains measured differ markedly with state, unlike the case of axial strains, loose and dense samples contracted and dilated respectively. The degree of contraction increased with decreasing grain size and *vice versa* for dilation. There were also marked differences between nail and reference tests: the onset and development of volumetric strains were inhibited by the presence of the nail. Sudden 'failure' witnessed with the coarse sand reference tests was inhibited in the nail tests.

Sample behaviour expressed in terms of stiffness, indicates that the nail was far more beneficial in a volumetric sense than axially where during the early stages of shearing it often appears to be detrimental. In order to gain insight into the benefit of the nail in an overall sense, the results have been analysed in terms of work. Differences in work, axial strain and volumetric strain between reference and nail test samples have been presented which give clear indications of the degree of shearing required before the nail benefit was mobilised.

In terms of work, the results indicate that the nail became beneficial after shearing to stress levels of $\eta = -0.6$ to $-1.0$, roughly corresponding to axial strains of $-1.0\%$ to $-2.0\%$, ...
the benefit of the nail then steadily increased with shearing. The increased restraint of volumetric strains by the nail did not contribute significantly to the work difference because their magnitudes were small compared to the axial strains (volumetric strains being composed of axial and radial strains which are dilatant and contractant respectively, thus countering each other). However, combining the two work components and presenting the overall nail benefit in this way has reduced the magnitude of the apparent detrimental behaviour of the nail and the shearing range over which it acts.

The initial detrimental behaviour is thought to have arisen principally from conditions imposed during the isotropic compression stage and the subsequent change in direction of the stress path at the onset of the shearing stage. The likelihood of this effect occurring under field conditions is uncertain.

The external measurements also indicate that very high values of stress ratio were reached towards the end of shearing in both the reference and nail test samples. According to critical state theory, values of the angle of internal friction in extension, $\phi'$, corresponding to the stress ratios reached were in a range of $55^\circ$ to $65^\circ$. Although these values are very high and seem unrealistic, similar values have been recorded by other researchers when testing at low stress levels during passive stress relief tests (e.g. Hellings, 1989 and Hellings and Burland, 1997).

The 'internal' measurements from the nail instrumentation are now considered to investigate conditions at the nail-soil interface and relate them to the overall external behaviour.

There was a reversal of axial stress with the onset of shearing, forces in the nail therefore started to decrease. Only beyond stress levels of $\eta \approx -0.2$, i.e. at axial displacements of about 1 mm, did the forces become tensile. Interface shear stresses were not mobilised at the first radial stress measuring device, about a third of the way down the nail, until after displacements of about 3 mm (corresponding to $\epsilon_r \approx -1 \%$).

In the case of the loose fine sand, the mobilisation of interface shear stresses continued steadily towards the base of the nail with values along the instrumented length increasing progressively with shearing. Radial stresses also continued to increase, those towards the base of the sample exceeding the applied boundary stresses because of rigid-body inclusion effects. The values of radial stress measured were similar to those calculated in Section 7.5 using elastic solutions.

Calculated values of interface friction over the instrumented length, determined from the interface shear and radial stresses measured at the end of shearing, represent about 75 % of the peak values measured in the shearbox. This indicates that there were only small displacements of the soil relative to the nail at this point (less than a quarter of the average values measured
at the top of the sample). Measurements of displacement of the soil core immediately adjacent to the nail at the top of the sample confirm this observation.

In the case of the loose fine sand, the benefit of the nail is clear from both external and internal conditions, there being a continued increase in benefit as shearing progresses and the front of mobilised interface stresses moves down the nail.

In the case of the dense fine sand, although the onset of mobilisation of interface stresses was registered at the instrument positions after about the same degree of shearing as the loose sample, there was little further increase in their magnitude at this location. Most of the mobilisation occurred towards the top of the sample with virtually none below the position of the upper instrument for the entire shearing stage. At the same time there was a minimal increase in radial stress and interface friction angles hardly develop. The difference in axial strains and work between reference and nail samples from the external measurements indicate the nail benefit started at stress levels similar or less than those of the loose sample.

The above observations regarding the dense sample indicate that the nail benefit implied by external considerations was either gained solely from mobilisation of interface shear stresses at the top of the nail or by some other mechanism. It is possible that the dense nature of the sand and the mobilised interface stresses at the top of the sample locked-up the structure in this vicinity where the sample was being unloaded. However, the radial devices indicate that at the same time there was little transfer of the applied boundary radial stress to the central part of the sample because of arching effects. Thus, possibly the driving stress was being translated into a more stable circumferential stress rather than deforming the sample axially. The results from the nail imply that a large proportion of the sample mass towards the base was essentially inert. Although the nail appears beneficial according to the external measurements, careful judgement is necessary when extrapolating conditions within the test apparatus to those of the field.

The behaviour of the medium dense fine sand sample was similar to that of the loose sand with a progressive mobilisation of shear stress along the nail length during shearing, similar degrees of interface friction were reached. The external measurements in this case indicate little benefit from the nail and then only during later stages of the shearing. Judging the benefit of the nail from differences in external measurements between reference and nail tests has been found to be misleading for medium dense samples. It was not easy to control the density in this state and if values for reference and nail test samples were either side of the critical density, contrary behaviours could be expected. Results from the internal measurements, if conditions were similar to those of the loose sample, would indicate that the nail in the medium dense sand was beneficial.

Results from the HRS suggest similar trends of behaviour to those of the fine sand but
with lower magnitudes of radial stress and axial force, and arching effects more accentuated. The transfer of radial stress through the sample to the nail was reduced and no rigid-body inclusion effects were evident. The distribution of radial stress along the nail had the same form as with the fine sand, with greater values towards the base of the loose sample and conversely towards the top for the dense sample.

The larger particles size caused increased arching effects compared to the fine sand and a greater interlocking of particles within the structure overall. During shearing, smaller axial displacements took place with this larger grain size. Although displacements along the nail interface were reduced, the shearbox tests indicate that higher friction angles were mobilised at smaller displacements for the HRS. This effect is evident in the test results and consequently interface shear stresses did develop, although not to the same degree as with the fine sand.

In assessing the benefit of the nail from the external measurements, the nail became beneficial in terms of work and axial strain after similar stress ratios as those for the fine sand. The nail in the loose HRS became beneficial after smaller degrees of shearing than when in a dense condition, this is more realistic given the greater degree of interlocking of grains in the dense state.

Observations regarding the misleading degree of benefit of the nail in the dense sample apply equally or more so to the HRS. Interface shear stresses are again only mobilised towards the top of the nail and the increased arching effect with grain size means that a greater proportion of the applied radial stress, acting as the driving stress, appears to have been transferred into circumferential stresses. Consequently more than half of the sample remained essentially inert.

Interpretation of internal results from the coarse sand tests is limited by the fact that the radial stress measuring devices have been shown to under-register significantly because of their size relative to that of the grains (there were very few points of contact).

Trends in behaviour are similar to those for the fine sand and HRS but the magnitude of interface stresses was considerably reduced. The mobilisation of shear stress only propagated to the position of the instruments for the loose sample, with the medium dense and dense cases frictional stresses were only generated at the top of the sample.

As arching effects develop more readily with increasing grain size, the proportion of applied radial stress being transmitted to the centre of the coarse sand samples was small and reduced further with increasing relative density.

Consideration of external boundary stresses and strains indicates that the nail in the loose coarse sample became beneficial after similar degrees of shearing as witnessed for the finer sands. The onset of its benefit in the dense coarse sand occurred from the very start of shearing,
probably because of the higher relative density of the nail to reference sample (one of the few instances where this was the case) combined with the transfer of the driving radial stress into circumferential stress, as discussed for the dense finer grained samples.

**Overview of results**

The preceding discussion of results highlights the need to assess conditions internally as well as externally when making judgements relating to the benefit of the nail. In most cases the nail appeared to be beneficial after some degree of shearing according to the external measurements (the displacements necessary to mobilise the benefit might in reality be less than implied by the results because of the generally lower relative densities of the nail samples). On examining the measurements made with the nail instrumentation, the behaviours of the loose and dense sands are seen to be quite different. In the former there is a high transfer of boundary stresses to the nail, and interface shear stresses are mobilised at progressive distances along the nail as shearing takes place. In the case of the dense samples most of the applied radial stress is thought to be transferred into circumferential stress because of arching mechanisms within the sand structure, thus the radial 'driving stress' is diverted. This effect combined with the greater degree of particle interlocking means that mobilisation of interface shear stresses only takes place at the top of the sample.

The benefit of the nail in the dense samples was partly derived as an indirect consequence of the presence of the nail. If arching effects do not occur to a similar degree under field scale conditions, perhaps the phenomenon of restrained dilation, which has not been witnessed in the shearing stages of the laboratory tests because of arching, could be initiated.

The results from the sand tests combined with those from the clay tests, discussed in the next chapter, are related where possible to field conditions in Chapter 10. Scale effects and the likelihood of phenomena such as rigid-body inclusion effects, arching and restrained dilation occurring are discussed in the context of the dimensions and boundary conditions encountered in practice.
CHAPTER 9

Clay test results

9.1 INTRODUCTION

A series of tests were performed to investigate the behaviour of a soil-nail in clay using the boundary conditions discussed in Chapter 3 and the apparatus and testing methods described in Chapter 5. The results from five tests performed on 150 mm diameter samples are described. The first, referred to as a reference test, without a nail, and the remainder with model nails installed. In three cases the nail was instrumented to enable measurements of axial force to be made at three points along its length, i.e. using the nail described as prototype 3 in Appendix 7.3.

Initially the reasons for choosing kaolin clay are given and the methods used to obtain similar samples of known composition, fabric and stress history explained. The testing techniques adopted are described in Chapter 5. Once the sample has been set up in the triaxial apparatus, there are six stages of the test. The sample is first isotropically consolidated, followed by four stages necessary to install the nail in a wished-in-place condition, and finally conditions of stress relief from excavation are simulated by shearing in axial extension.

The measurements made during each of these stages of the test are presented and discussed in terms of the conventional parameters used in the interpretation of triaxial tests. The comments concerning the use of invariants, given in Section 8.3.1, are equally applicable to the clay tests. The results from the clay tests are not as comprehensive as those with sand, because essentially a single material was tested in the same normally consolidated state. Also, no measurements of radial stress were possible with the version of the instrumented model nail available at the time of testing (the tests were performed before the final development of the prototype 4 nail).

A similar approach in presentation is adopted as for the sand test data where first the sample behaviour is discussed in terms of the external boundary stresses and strains, and then internal behaviour at the nail interface is considered.

The intention during each test was to follow the same paths to achieve repeatable and comparable results up to the shearing stage. In practice, by the beginning of the shearing stage, small variations in paths had occurred. Reasons for these differences are given and taken into account in the discussion of the results. They are also interpreted with respect to the results from the instrumented nail. Because of these differences there is less emphasis on comparing
nail test results with those from the reference test, which was one of the main approaches adopted for the interpretation of the sand tests.

The principal stage of the test is that where conditions of stress relief at the face of an excavation are modelled. Results from this stage, which is generally referred to as the shearing stage, are presented and discussed in detail. The data from the instrumented nail are presented in relation to both the point after which the nail was grouted into the sample and also to the start of the shearing stage. Further analysis of the data for this stage is made using the results from an interface shearbox test performed by Martins (1983). Estimates are made of possible ranges of radial stress acting on the nail and conversely of ranges of interface friction angles necessary to generate set radial stresses.

A seventh stage can be considered to be the unloading of the samples at the end of the tests. Particular care was needed with two of the samples which were prepared for microfabric analysis using thin-section techniques. These methods are described and the observed fabrics discussed within the context of the experimental results.

9.2 CHOICE OF CLAY SOIL TYPE

The soil used for the clay tests was Speswhite kaolin, a clay mined and processed by the English China Clay Company. There are various reasons for choosing this clay, the main ones are as follows.

(i) Kaolin has been the subject of continuous research testing for the past thirty years. Even though different types of kaolin have been used, (e.g. the early tests at Cambridge, used in the development of critical state theory, were performed with Spestone kaolin), the basic properties are very similar. Speswhite kaolin has been used in more recent research, allowing comparisons to be made with this study (e.g. Martins, 1983; Al-Tabbaa, 1987; Mahmoud, 1989).

(ii) Kaolin is a relatively coarse grained clay, meaning that sample preparation from a slurry can be made with minimal time, whilst still obtaining a material of clay properties. The properties of Speswhite kaolin are detailed by Martins (1983).

(iii) Kaolin is known to develop readily microfabric features which can be studied using thin-section techniques, (Morgenstem and Tchalenko, 1967a and b; Martins 1983; Mahmoud,1989). Preparation of thin-sections for such analysis is also facilitated because of the coarse nature of this clay.

(iv) The Spestone kaolin clay originally tested at Cambridge was that used in the development of the ‘Cam Clay’ and ‘Modified Cam Clay’ constitutive models, (Schofield and Wroth, 1968 and Roscoe and Burland, 1968). The latter model provides a means of numerically analysing the behaviour of this clay, (as discussed in Chapter 4).
The properties of Speswhite kaolin established by Martins (1983), in terms of strength and consolidation parameters, are given in Chapter 4 where they are used as input data for the numerical analyses (see Section 4.2.3).

9.2.1 Preparing samples of uniform fabric

An internal report was prepared at the beginning of the study summarising past research into the effects of slurry water contents, placement, and consolidation method on resulting sample fabric. This summary is given in Appendix 9.1.

In view of the observations in Appendix 9.1, a slurry of water content 130% was used (i.e. roughly twice the liquid limit which was established to be 63% and 57% using the cone and Casagrande apparatus respectively, a plastic limit of 33% was determined). The two principal reasons for this were that (i) the requirements necessary to obtain an initial slurry of uniform fabric are satisfied and (ii) with this initial water content, it was possible to form a sample of the required height in the consolidation apparatus used.

Since completing the report, a recommendation for the moisture content at which to prepare reconstituted samples has generally been accepted to be 1.5 times the liquid limit (Burland, 1990). Therefore a slurry at twice the liquid limit should certainly satisfy the fabric requirements.

9.3 SAMPLE BEHAVIOUR DURING THE STAGES PRIOR TO SHEARING

In this section the sample behaviour as determined from the boundary measurements of pressure and displacement are discussed. The first stage discussed is that of isotropic consolidation where the effective stresses are increased to exceed those applied during the one-dimensional compression from slurry. The other stages were necessary to install the nail in a wished-in-place condition.

Samples were formed in the large oedometer and then trimmed and set up in the triaxial apparatus as described in Sections 5.7.2 and 5.7.3 respectively. In the case of sample NAIL2 the membrane had a rubber ‘lentreset’ grid of 1cm squares applied on it to see whether any specific deformation modes could be detected during the test (this grid was kindly provided by Professor Tatsuoko of University of Tokyo). A photograph showing the sample from test NAIL2 set up in the apparatus at the start of the test prior to isotropic consolidation is given in Figure 9.1; two views are shown, one before placing the perspex around the sample and the other through the perspex and cell water. In Figure 9.2 similar photographs of the sample are shown taken at the end of shearing.

9.3.1 Isotropic consolidation

Prior to opening the drainage lines to start the consolidation stage the cell and ram
pressures were incrementally increased simultaneously until the final required boundary stresses were applied. The cell pressure at this point was 450 kPa. Plots of pore pressure equalisation, given in terms of sample effective stress, for this undrained loading are shown in Figure 9.3.

If no drying of the samples or disturbance occurred during the storage and trimming procedures, the effective stresses measured would be expected to be the same as those reached at the end of the one-dimensional consolidation stage in the oedometer. A value of $K_0$ can be estimated as 0.65, by taking an average of a value of 0.715 determined from a Modified Cam Clay prediction by Martins (1983) and one of 0.577 calculated from the Jaky equation with $\phi' = 25^\circ$. With an applied one-dimensional vertical consolidation stress of 200 kPa, the horizontal stress would therefore be 130 kPa, giving a mean effective stress, $p'$ of 153 kPa.

Generally, lower values are observed, the plots indicate initial sample effective stresses ranging from 75 to 165 kPa. The decrease in stress probably results from water being sucked into the sample from the porous stones during unloading in the oedometer. Values calculated from the mid-height measurements are generally about 15 kPa higher than those at the ends indicating that water is probably also taken in at the ends during set-up in the triaxial apparatus.

The results from test NAIL0 are not shown as there was a small leak into the sample during the initial stages of this test which subsequently sealed itself during the consolidation stage. Initial effective stresses of about 30 kPa were calculated.

Sample consolidation commenced when the drainage was opened with a back pressure of 200 kPa. Plots of axial, volumetric and deduced radial strain with time are shown in Figure 9.4. Average initial water contents, $w$, and bulk densities, $\rho$ measured during sample trimming are marked on the figure. There is quite a large variation in the volumetric strains which do not seem to be governed by either $w$, or $\rho$ but which can be related to the final stresses shown in Figure 9.3. The greater the change in effective stress required to reach the final stress level of 250 kPa, the larger the resulting strain. The effect is quite evident from the plots of strain versus mean effective stress given in Figure 9.5.

The consolidation stage generally lasted more than a week. At the end of consolidation the samples were allowed to creep until the strain rate was less than 0.005 %/day, in preparation for the undrained unloading stage, when axial height is maintained constant. Axial and volumetric strain curves shown in Figure 9.4 are essentially flat prior to this stage.

The degree of sample saturation was checked at the end of the isotropic consolidation stage in test NAIL2. An undrained isotropic stress increment of 54 kPa was applied which gave $B$ values of more than 0.94 within twenty minutes. The $B$ values calculated from the pore pressure measurements made at the base, mid-height and top of the sample are given in Figure 9.6. This value was deemed acceptable. $B$ checks were not otherwise performed as after the
air flushing, grouting and other operations during the nail installation stages, the degree of saturation would not be the same as that at the end of consolidation and could not be checked without risk to the sample fabric around the nail.

### 9.3.2 Undrained unloading

Prior to this stage the sample ends were flushed with air to remove surplus water from the gauze discs behind the polished sample end plates, as described in Section 5.8.2. The cell and ram pressures were then reduced as shown in Figure 9.7. The time plotted includes the period for installing the nail and grout setting. As the stage was undrained, pore pressure within the sample would be expected to reduce to maintain a constant effective stress of 250 kPa. The pore pressure response at the mid-height probe position are also shown in Figure 9.7, and can be seen to increase initially to values in excess of $-200$ kPa. There is a sharp initial decay when values reduce by about 50 kPa, thereafter the readings reduce much more slowly or remain essentially constant. The decay is probably related to a combination of the sample sucking in water from the radial side-drains of the sample and cavitation of the porous stone in the probe (the stone used had an air-entry value of 5 bars). Comprehensive details on this subject are given by Ridley (1993).

During unloading which was carried out in a period of about ten minutes, the axial height of the sample was generally kept to within ± 0.2 mm of the average initial value (which represents changes in axial strain of ~ ± 0.15 %). Usually, when the cell pressure had been reduced to zero, it was still necessary to apply a finite ram pressure to maintain the sample height within the defined limits. This resulted in an increase in deviatoric stress in the sample of about $+25$ kPa. The necessity to provide an axial stress when there is no all-round stress indicates that the sample was trying to expand axially. This provides further evidence that the sample was sucking in water from the radial drains and ends, as under undrained conditions no volume change would be expected. Any axial expansion would have to be countered by a radial contraction, which is unlikely under conditions of zero all-round pressure. The degree of control achieved in maintaining constant axial height can be assessed from Figure 9.8.

The positive ram pressure was maintained during the next stages although it was sometimes necessary to adjust it slightly in the initial period to maintain the constant height.

### 9.3.3 Installing the nail

The procedures for installing the nail are described in Section 5.8.3. Samples were taken during the drilling operation, providing a moisture content profile over the sample height, as shown in Figure 9.9. The overall shape of the distributions further confirms that surplus water was drawn in at the ends.

The nail was then locked at the base and grouted in place. Inspection and measurement
of the set grout after the sample was stripped away in test NAIL2 revealed that the grout was not uniformly distributed around the top of the nail. Wires to the gauges could be felt through the grout thickness which was thicker on one side. New guides for drilling were machined to a much higher tolerance. The resulting grout distribution in tests NAIL 3 and 4 was very uniform to within a few millimetres of the top of the sample.

Once the grout had set, several hours after unloading, the sample height had slightly increased and the effective stresses decreased (typically by 80 to 120 kPa) for the reasons discussed above.

The nail installation procedure and grouting technique was checked to assess fabric disturbance with pilot test TNAIL4. The sample was not reloaded after grouting was complete, and thin-section samples were prepared for microfabric analysis. The results from this analysis are discussed in Section 9.7.

9.3.4 Undrained reloading

The cell and ram pressures were increased together incrementally, as shown in Figure 9.10, keeping axial height constant. In maintaining this condition, the cell pressure was increased above the ram pressure resulting in the samples being subjected to greater radial stresses than those applied axially (i.e. negative values of $q$). Towards the end of the stage for test samples NAIL1 to 4 the final mean effective stress had not been reached and the deviator stresses were in the region of $-20$ kPa to $-50$ kPa.

This stress state again reflects that water was sucked into the sample and is evident from the excess pore pressures generated during reloading, as shown in Figure 9.10. The final expected pore pressure value being 200 kPa.

The axial strains that took place during undrained reloading are shown in Figure 9.11. In the case of NAIL0, the reference test with no nail, greater axial strains were allowed to develop. Under this condition negligible deviator stresses were induced, although lower mean effective stress values resulted, indicating that considerable additional water must have been drawn into this sample. Water may also have entered through the leak into the sample that was evident in the early stages of the test.

The axial strains that developed during reloading in tests NAIL1 to 4 were within a range of 0.1 %, in the case of NAIL0 the axial strain was at least three times this value. These strains were generally compressive, countering to some degree the negative strains that developed during the unloading stage (Figure 9.8). At the very end of the stage the ram pressure was increased slightly in tests NAIL1 and 3 to reduce the deviatoric stress level. The sample ends were then resaturated by flushing with water.
9.3.5 Reconsolidation

If the mid-height probe, at the end of the reloading stage, had returned to its reading prior to unloading, no sample consolidation would be expected. In practice, pore pressures typically rose by up to 10% in excess of their previous values. On opening the drainage with an applied back pressure of 200 kPa, consolidation commenced with pore pressure dissipation.

After 'reconsolidation' the mean effective stress had increased to within ±5 kPa of the target value of 250 kPa. Further adjustments to the ram pressure were made in tests NAIL2 and 3, after opening the drainage, to reduce the deviatoric stresses that had developed. In the other three tests the range of values of $q$ at the end of the stage were within $-5$ kPa to $-25$ kPa for all but test NAIL4 which increased to $-34$ kPa.

Axial, volumetric and radial strains that develop during reconsolidation are shown in Figure 9.12. Further compressive increases in axial strain occur in all cases (Figure 9.12a). In test NAIL0 the axial strain is in excess of 0.5%, giving a cumulative axial strain since the end of the unloading stage of about 0.9%. Although this counters the dilatant strain of $-0.6\%$ that occurred during unloading, such movements would undoubtedly have caused fabric disturbances if a nail had been installed in the sample. The maximum axial strains developed in the nail samples are about 0.15%, generally they are lower than this value. The possibility of fabric disturbance in the periods leading up to shearing are discussed in the context of the microfabric analyses in Section 9.7.3.

Volumetric strains are shown in Figure 9.12b. In all cases compressive strains occur in the initial stages. In the case of NAIL0, the volumetric strains are almost zero, the relatively large axial strains developed, are therefore countered by corresponding dilatant radial strains as shown in Figure 9.12c. The trend in volumetric strain exhibited by sample NAIL4 clearly indicates a leak into the sample, strains are occurring at a constant rate of almost $-0.3\%$/day. A correction is applied to the shearing stage results for this test to account for the leak.

The end of the reconsolidation stage was controlled by the axial strain rate which was to be less than 0.01% of the shearing rate. At this point the original intention was that the sample should be at the same stress level as at the end of the isotropic consolidation, with minimal fabric disturbance and a 'wished-in-place' nail installed. Assessing the degree to which this was achieved is somewhat subjective. Controlling the sample state to high tolerances was not straightforward for a variety of reasons. Even if accurate control of the boundary stresses and strains were possible, problems would still occur because of the changing sample state resulting from the water sucked in during the unloading stages. Manual control was used in all tests and compromises were made regarding minimising sample deformation and achieving the required final stress state.
9.4 SAMPLE BEHAVIOUR DURING SHEARING IN DRAINED EXTENSION

The shearing stage of the test is considered to be the principal stage in terms of modelling the behaviour of a soil-nail operating as part of a nailed-soil excavation. The sample responses observed between tests in the previous stages have been described comprehensively to facilitate an understanding of the behaviour during the shearing stage.

Modelling of the stress relief at the face of an excavation was achieved by shearing in axial extension, i.e. by unloading the ram pressure, thus extending the sample axially under constant radial stress, with the drainage open.

The five tests described in the following section should have similar histories up to this stage. Test NAIL0 was a reference test in which no nail was installed but which had been subjected to the same stages and stress paths as the nail tests. Tests NAIL1 to 3 should essentially be the same. A dummy uninstrumented nail was installed in test NAIL1 which was sheared to the limit of the available axial travel. Thin-sections were prepared from the sample at the end of shearing for microfabric analysis. Test NAIL2 was the same as NAIL1 except that an instrumented nail was installed, capable of measuring axial force at three locations along its length. The same instrumented nail was installed in test NAIL3, in this case shearing was stopped after a certain axial strain. The sample was then unloaded and again thin-sections prepared. The rate of shearing was constant for NAIL0 to 3 at about 0.625 mm/day. Test NAIL4 was sheared at four increasing speeds to assess rate effects.

9.4.1 Discussing differences in sample behaviour

The stress paths followed during shearing for all five tests are presented in Figure 9.13a in $p'-q$ space. The origin is shown in the upper part of the figure with lines of constant $\phi_v'$ equal to 15°, 20° and 25° marked. The final value of $\phi_v'$ reached in the reference test was 17.5°, while values for tests NAIL1 and 2 were 19.5° and 20.6° respectively, indicating that the nail installed has increased the resistance to shearing. The mobilised value of $\phi_v'$ for test NAIL3 is about 11° because shearing was curtailed at an early stage.

Under ideal conditions, the stress path in $p'-q$ space for shearing in drained axial extension would be a straight line inclined at 1:3 with the path direction such that the $p'$ and $q$ values were decreasing, as marked in Figure 9.13b. The paths shown for tests NAIL0 to 4 are seen to follow roughly this path, given the slightly different stress states at the start of shearing. Deviations most probably arise because the pore pressures registered at the mid-height probe were used to determine values of mean effective stress, $p'$. The plots are presented without any normalisation to correct for differences in initial stress.

At the very start of shearing, the mean effective stress in the samples increases slightly as a result of initial decreases in pore pressure (see Figure 9.13b). The pore pressure responses
at the base and mid-height position of the sample are shown plotted against axial strain in Figure 9.14. At the base where drainage is closed but directly connected to the top of the sample via the side drains, pore pressures are within 5 kPa of each other. There is an initial drop of 1 to 2 kPa which is recovered after about -1 % axial strain. At the mid-height where the probe is in direct contact with the soil and roughly mid-way between side-drains, the initial decrease is more severe than that measured at the base, but recovery occurs more quickly. These initial drops in pore pressure and consequent increases in mean effective stress point to an initial dilatant behaviour in the samples, even though shearing was under drained conditions. It is thought that these drops and consequent dilation are associated with mechanical behaviour rather than shearing rate. They reflect that the shearing rate was slightly too fast for this initial period.

The same behaviour is observed at the start of test NAIL4, but as soon as the shearing rate is increased, pore pressures rise with a consequent decrease in $p'$. The rates of shearing, in terms of plots of axial strain versus time are given in Figure 9.15. Shearing rates in tests NAIL0 to 3 are similar, the initial shearing rate of test NAIL2 appears greater than those of the other tests.

Evidence of the dilation that occurs at the start of shearing can be seen in Figure 9.16, in which volumetric and radial strains are plotted against axial strain. All tests undergo an initial dilation before contracting (Figure 9.16a). There is a wide spread of volumetric strains with axial strain, i.e. the degree of contraction is quite variable. The uncorrected curve for test NAIL4 is shown with a dotted line, these values have been adjusted by applying a rate correction to account for the leak that was observed during the reconsolidation stage (i.e. $-0.295 \%$/day). The different slopes within the corrected curve reflect the different shearing speeds used in this test because the correction is time-based.

Reasons for the variation in the volumetric strains and the dilations that occurred can be explained by considering the moisture contents and initial deviator stress values. The curves in Figure 9.16a have been annotated accordingly. The moisture contents are those obtained from the drilling operation during nail installation (an average was chosen from the profiles given in Figure 9.9). The degree of contraction increases with moisture content consistently for all tests, even though variations in moisture content are small. The effect cannot be checked with sample NAIL0 as no drilling was performed.

The initial dilation occurs because of the stress path followed. The samples have been normally consolidated under isotropic conditions to a set mean effective stress, with a current yield surface formed with its tip at that point. If the stress path moves back within the yield surface, the sample behaves elastically. Once the stress state extends beyond the yield surface plastic sample deformations commence. Because the stress path is in extension with $p'$ and $q$
decreasing it remains within the yield surface for the initial stage of shearing. At this time, elastic movement proceeds up the unload-reload line before the sample starts to contract, as shown schematically in Figure 9.17. This initial stage might be considered as an elastic softening. The closer the stress state is to the yield surface at the start of shearing, the shorter the elastic path and hence the smaller the degree of dilation that occurs. This trend is seen clearly in the enlarged section of the dilational volumetric strain plot shown in Figure 9.16a.

Radial strain plots deduced from axial and volumetric strains are shown in Figure 9.16b. The curves are almost linear and fall within a narrow range. A change in slope is evident at the very start of shearing in the same region corresponding to where the dilational volumetric strains occurred. The slope of these initial curves, if the sample is within its current yield surface, would be expected to represent the elastic Poisson's ratio (i.e. have a slope in a typical range of 0.2 to 0.5). The slopes thereafter are about twice as steep.

Deviator stress versus axial strain curves and the secant stiffnesses derived from them are plotted in Figure 9.18a and b for all tests. The curves are within a narrow band, with the exception of tests NAIL2 and 4. The reason for the increased stiffness of the sample in test NAIL2 is unknown, otherwise there is not much difference between the reference and nail test samples. In the case of test NAIL4, the stress-strain curve shown in Figure 9.18a can be seen to become flatter in steps as the shearing rate is increased. The stiffnesses are referred to again in Section 9.6.1 during interpretation of data from the nail instrumentation.

Shearing was continued up to the point when the ram system had reached the end of its travel in tests NAIL0 to 2, i.e. to values of $\varepsilon_x$ of $-5\%$ to $-10\%$. Test NAIL3 was stopped at a specific stage governed by the force response in the nail, so that the sample could be prepared for microfabric analysis.

9.4.2 Sample behaviour during shearing in test NAIL4

Shearing of the sample in test NAIL4 was carried out under four different shearing rates. The different shearing rates are evident in Figure 9.15, the rates of axial displacement were: (i) 0.625mm/day, the standard rate used for the other tests; (ii) 3.125mm/day; (iii) 6.25mm/day and; (iv) 13.5mm/day, i.e. twenty times faster than initially.

The deviator stress at the start of shearing was lower than the other tests, no adjustment was made to the ram pressure as it was for tests NAIL 2 and 3. Consequently the degree of dilation that takes place at the very start of shearing, representing the elastic response of the sample, is smaller than in the other tests (refer to Figure 9.16a). During shearing, the stress path for this test deviates furthest from the theoretical 1:3 line because of the excess pore pressures generated by the increased shearing rates as shown in Figures 9.13 and 9.14. The angle of internal resistance in extension, $\phi_i$, mobilised by the end of shearing was $16.3^\circ$ compared to the
value of $\phi_s' = 20.6^\circ$ from test NAIL2, where the sample was sheared to a similar degree in terms of axial strain. The secant stiffness of sample NAIL4 shown in Figure 9.18 is also significantly lower than those of the other test samples even up to axial strains of about $-1\%$.

The effect of increasing the shearing rate is further discussed in terms of the measurements made at the nail interface in Section 9.6.3.

**9.5 RESULTS FROM THE INSTRUMENTED NAIL PRIOR TO SHEARING**

Responses at the interface are presented and discussed in this section using the results from the instrumented model nail, prototype 3, capable of measuring axial force at three points along its length. A photograph of the type of nail used in these tests is shown in Figure 9.19. The nail was installed as described in Section 5.8.3. Calibrations were performed when possible on the grouted nail after testing to obtain regressions for the axial force in the nail. Average shear stresses are deduced over three lengths of the nail: between the lower and middle devices, the middle and the upper devices and between the upper device and the top of the nail. This final shear stress is calculated on the premise that the axial force at the top of the nail is zero. Its point of action is unknown, in the following figures it has been assumed to act mid-way between the upper device and the top of the sample.

The purpose of the discussions in this section is to establish the history of the interface stresses that develop leading up to the start of the shearing stage.

**9.5.1 Behaviour during reloading stage**

Axial force and derived interface shear stresses from the three tests are plotted versus deviator stress in Figure 9.20a and b. Interface parameters were interpreted in terms of deviator stress rather than axial strain as the latter was being held under control during reloading and therefore fluctuated while the deviator stress progressively decreased. Axial strain is included in Figure 9.20c for completeness.

The results presented are expressed in relation to the start of the reloading stage. The reason for this is to eliminate the effects of nail installation and setting of the grout which generally induced a small compression in the nail, mainly due to the grout contracting. These effects are not considered relevant to the study, additionally because one of the wires to the upper axial force device became dislodged during nail installation in test NAIL2 the readings from this case are incomplete from the beginning of installation. The circuit of the upper device in test NAIL2 was closed by forming a quarter-bridge enabling readings to be made but to a lesser resolution (see Section 7.4.1).

The responses of the axial force devices shown in Figure 9.20a indicate that the nail goes into tension as the sample goes into extension. The samples undergo different degrees of
extension before the nail forces develop in each of the tests. When the tension does start to pick up, there is little variation in force between the devices during the initial stages. This indicates that there is no shear stress over the instrumented length, but that because the force is developing there are shear stresses at the top of the nail. As reloading continues a divergence of the force outputs commences, indicating that shear stresses are starting to move down the nail from the top. In the cases of tests NAIL3 and 4 greater axial forces are developed towards the base of the nail, the converse occurs in test NAIL2.

The development of interface shear stress at the three positions along the nail are shown in Figure 9.20b. Development of shear stress at the upper position (square symbols) takes place first in all cases. Significantly lower stresses operate towards the base of the sample in tests NAIL3 and 4. In accordance with the ordered, diverging axial force plots in test NAIL2, negative and positive interface shear stresses are developed towards the top and base of the sample. The compressive strains occurring during this stage (Figure 9.20c) are contradictory to the fact that the sample is going into extension, but are explained in Section 9.3.4 in terms of the strains that occurred during the unloading stage.

Axial force and shear stress values at increasing stages of reloading are plotted along the length of the nail in Figure 9.21. Each line corresponds to a selected time during the reloading stage. The increasing tensile axial force is seen in all cases along with the development of shear stress towards the top of the nail.

The behaviour of the samples from tests NAIL3 and 4 follow a logical pattern that can be readily related to the idealised models developed in Section 8.4.2, in particular case 6, shown in Figure 8.34f. The different magnitudes of axial force and interface shear stress are thought to be related to the sample stress state at the start of the stage. In the case of test NAIL4 the sample was considerably more in compression than that of NAIL3 therefore requiring a greater degree of unloading to mobilise tensile shear stresses.

The responses taking place in test NAIL2 are not easy to explain but can be shown to be feasible by considering case 8 of the idealised models, shown in Figure 8.34h where tensile and compressive forces and shear stresses operate at the same time along the length of the nail. The non-uniform behaviour observed is possibly related to water absorbed in the sample at its ends and boundaries during the unloading and nail installation stages. Greater sample swelling may have occurred because of the additional time required for repairs to the instrumented nail at that time. Non-uniform swelling stresses would arise in the vicinity of the nail because of its restraint.

The continuous interface shear stress plots shown in Figure 9.20b indicate that by the end of the stage, the shear stresses towards the base of the nail were within a small range of ±
20 kPa. Towards the top of the nail the scatter was greater, being within $-50$ and $-125$ kPa.

### 9.5.2 Behaviour during reconsolidation stage

Sample reconsolidation was started as soon as the total stresses at the boundaries had been reapplied and the sample ends reflushed with water. On opening the drainage and back pressure, the sample went into compression and most of the strains developed within the first twelve hours (see Figure 9.12). Conditions thereafter were quite stable. The nail responses during this time are shown in Figure 9.22a and b where axial force and interface shear stresses are plotted against deviator stress. Axial strain is also plotted for reference in Figure 9.22c.

The nail in all cases went into compression in conjunction with the sample. Reasons for the variations in compressive nail forces that develop in the three tests cannot be ascribed to any one cause. In the case of test NAIL4 the change in $q$ was about 10 kPa and responses in the nail negligible, even though axial strains were comparable with the other tests. In tests NAIL2 and 3, greater deviatoric changes occurred and greater nail forces were induced. These forces continued to develop as consolidation took place, and stabilised as the creep diminished.

Differences in the force developed at the three positions along the nail are evident from the interface shear stress plots presented in Figure 9.22b. Larger shear stresses were mobilised towards the top of the nail where the potential for relative movement between soil and nail is greatest. Further down, interface stresses are considerably smaller. These trends can be seen in the results from all three tests, though in the case of NAIL4 changes are very small.

The pattern of behaviour is more clearly seen in Figure 9.23 where axial force and interface shear stress are plotted along the length of the nail. In all cases shear stress changes were greatest towards the top of the nail, although the degree of change is variable. In a similar manner to the reloading stage, the results from tests NAIL3 and 4 follow a logical pattern, which can be compared to case 7 of the idealised models, shown in Figure 8.34g. Axial forces increase towards the base of the nail, although only marginally, and shear stresses act in the same sense along its length.

In the case of test NAIL2 residual stresses, remaining from the reloading stage, result in force and stress distributions along the nail that are not easy to interpret. Axial force increases towards the free end of the nail and positive and negative shear stresses are registered along its length.

In conclusion, it is suggested that perhaps detailed explanation should not be sought for these stages prior to shearing, where the intention was to control the sample boundary stresses to cause minimum disturbance to the fabric whilst returning to the required, original stress state. The interface stresses developed are not insignificant, but nonetheless the axial strains that the sample underwent are very small, being 0.14 % maximum. This represents about 0.2 mm axial
displacement at the top of the sample, which reduces to zero at the base where the nail is rigidly connected to the lower platen. Relic stresses from the installation stage are even more uncertain but are thought to be responsible for the behaviour observed during NAIL2.

At the end of the reconsolidation stage the responses from nail instrumentation and the sample boundary stresses and strains were stable, the latter being within a set creep rate criterion.

9.6 RESULTS FROM THE INSTRUMENTED NAIL DURING SHEARING

In this section the nail responses occurring during the simulation of stress relief at the face of an excavation are presented and discussed. The forces in the nail are considered relative to two datums. First taking the force in the nail at the start of the reloading stage as zero, as with previous plots presented. Second taking the force at the start of shearing as a zero datum so that the mobilisation of stress from the shearing stage is isolated. The datum used is always marked on the figures.

The forces generated in test NAIL4 during the stages leading up to shearing were much lower than those from the other two tests. This is particularly so during the reconsolidation stage and is related to the degree of deviator stress applied to the sample. In the case of test NAIL4, no corrective adjustment was made to reduce $q'$ to bring the sample state nearer the isotropic axis at the start of reconsolidation. As a consequence at the start of the shearing stage the initial deviator stress was lower than for the other tests and the subsequent degree of shearing that takes place before the failure envelope is approached is also reduced. For this reason the nail responses were also markedly lower during shearing. Because of these differences, and the fact that in this test the shearing speed was increased at three points during the stage to assess shearing rate effects, the results are discussed separately from the other tests, in Section 9.6.3.

9.6.1 Data presented in relation to the start of the reloading stage

The development of axial force and interface shear stress in relation to the start of reloading are presented as continuous plots against deviator stress and axial strain in Figures 9.24 and 9.25 respectively. The nail starts going into tension as soon as shearing commences. The rate at which this occurs is similar at each of the three positions along the nail for tests NAIL2 and 3.

In both of these tests there are residual compressive stresses remaining from the end of the reconsolidation stage, which increase in magnitude towards the base of the nail. Maximum force is generally induced at the fixed base of the nail, demonstrated with the idealised models discussed in Section 8.4.2. A reversal of the force distribution occurs as shearing progresses, with the force at the base of the nail having the greatest changes as it switches from a maximum compressive force to a maximum tensile force. An example of this behaviour is shown in the
idealised model presented in Figure 8.34h. The switch-over of the force distribution occurs shortly after the start of shearing in the case of test NAIL3 but not until after \(-1\) to \(-2\%\) axial strain in test NAIL2.

In the case of test NAIL2, the rate of development of axial force along its length, shown in Figure 9.24a, is almost constant (i.e. the lines are parallel). This suggests that interface shear stresses are only mobilised above the instruments, which can be seen in the plots for the mid and lower positions in Figure 9.24b. An alternative and more probable explanation is that initially the nail is not completely free at the top, evidence of this can be seen in Figure 9.25a. A significant axial force develops within the nail with negligible change in axial strain for values less than \(-0.001\%\) (i.e. at the limit of the graph). However, the results from the reconsolidation stage do not indicate this problem. It is therefore assumed that a temporary sticking occurred at the top of the nail during the initial shearing in test NAIL2. The effect is not thought to have affected the overall subsequent trend in results. It may have contributed to the increased overall sample stiffness observed in Figure 9.18b.

The interface shear stresses shown in Figure 9.24b indicate more clearly the mechanism that takes place as shearing progresses. The maximum changes in interface shear stress, \(\tau_{\text{int}}\), occur at the top of the nail, which in the case of test NAIL3 is from compressive to tensile. A similar degree of shearing occurs with test NAIL2 but the shear stress is tensile from the start. Shear stresses in both tests decrease towards the fixed base.

The magnitude of the shear stresses at the top are based on the assumption that the force at the free end of the nail is zero, if forces are developed where the nail passes through the guide, e.g. at the sealing o-ring, the calculated shear stresses might be over-predicted.

The interface shear stresses shown for test NAIL2 have not been corrected for the initial sticking that is thought to have occurred. It is assumed that as friction at the top was overcome, the previously restrained movement took place, thus generating the shear stresses that would have otherwise developed earlier. This assumption is discussed further in the next section where the results are presented relative to the start of the shearing stage.

There is a sudden, small drop in the axial force along the nail at about the same deviator stress of \(q = -55\) kPa for both tests, as shown in Figure 9.24a. The nail data plotted against axial strain in Figure 9.25a, show that it takes place at small strains of between \(-0.05\) and \(-0.2\%\). The magnitude of the drop is only about 20 N but is quite marked. It is of significance because after it occurs, the build-up of force in the nail continues but at a slower rate than previously, and the development of shear stress, \(\tau_{\text{int}}\), over the upper length progressively reduces. In both tests, the shear stress at the upper position actually starts decreasing after about \(-1\%\) axial strain; the shearing stage in test NAIL3 was stopped at this stage. In test NAIL2 there is
a continued gradual reduction in the upper $\tau_{int}$ value up to an axial strain of about $-3\%$ after which it stabilises, possibly at a residual interface angle of friction.

Although the observed drop is only registered as a sudden change in shear stress at the upper position, indicating that it is induced in the vicinity of the free end of the nail, changes in the rate of development of $\tau_{int}$ lower down commence at the same time. In particular, the rate of mobilisation of $\tau_{int}$ at the mid-position starts to increase, suggesting that the component of overall sample deviator stress being carried by the nail starts to be transferred further down the nail at this time. There are also marginal increases in the development of shear stress at the base position. In the case of test NAIL2 the development of shear stress at the base almost stabilises after an axial strain of about $-3\%$. This probably reflects the limit of $\tau_{int}$ that can be developed towards the base because of the imposed condition of no relative movement between nail and soil at this point. This limit is clearly seen in the idealised models where the progress down the nail of the maximum $\tau_{int}$ value converges to a fixed point after many displacement increments (see for example Figures 8.34f and h).

It is felt that the observed drop is significant because it marks a point where the behaviour at the interface of the nail changes markedly. Only minimal additional shear stresses develop along the nail length after the drop occurs. Several reasons for the cause of the drop might be postulated. These are discussed in the final Section 9.9 after further interpretation of the results and consideration of the thin-section slides (i.e. the microfabric studies).

The results from the shearing stage, in relation to the start of the reloading stage, are plotted along the length of the nail in Figure 9.26. The magnitude and form of the distributions of axial force and interface shear stress for the three tests are somewhat varied. In tests NAIL3 and 4 the axial force increases towards the base of the nail from the start of the shearing stage. In the case of test NAIL2 this trend only starts to develop after significant strains have taken place, the larger axial force acting at the top of the sample might result from the sticking in the early part of the stage, however it could equally be caused by residual stresses from the reconsolidation stage.

The shear stress distributions have a similar form with the greatest mobilisation taking place towards the top of the nail for most of the stress relief.

**9.6.2 Data presented in relation to the start of the shearing stage**

In order to concentrate on the effects of shearing alone the results have also been plotted in relation to the start of the shearing stage, using this as a zero datum. Three corresponding plots to those just discussed are shown in Figures 9.27 to 9.29.

Some of the trends in behaviour are more clear with the data presented in this manner. In all cases the development of axial force, shown in Figures 9.27a and 9.28a, increases from
the top of the nail down to its fixed base, especially with test NAIL3 where there is an even
distribution of force along its length (see Figure 9.29).

In the case of NAIL2 the development does not occur immediately, because of the
sticking discussed in the previous section, consequently the lines are almost colinear initially.
The plots of axial force versus axial strain given in Figure 9.28a indicate that the force required
to overcome the friction at the top of the nail was about $-150$ N (the three devices register
values within the range of $-140$ to $-150$ N at the point where finite strains are shown, i.e. at $\varepsilon_s^n = -0.001\%$).

The similarity in results between tests NAIL2 and 3 when expressed versus deviator
stress (Figure 9.27) implies that the sticking observed at the beginning of test NAIL2 did not
significantly affect subsequent behaviour after it was overcome.

The greater development of force and shear stress along the nail in test NAIL3
(compared to that of NAIL2) might result from the distribution of grout covering the nail. After
stripping out the sample and examining the exposed nail in test NAIL2, the grout was found to
be non-uniform around the top of the nail, whilst a uniform grout thickness was observed in test
NAIL3 during subsequent thin-section analysis.

The results plotted along the length of the nail shown in Figure 9.29 also indicate good
compatibility between tests NAIL2 and 3 during the shearing stage. Axial forces developed
during the stage are almost the same up to the peak values at which test NAIL3 was terminated.
In the case of test NAIL2 axial force starts to decrease at the top of the nail in the later stages
of shearing. The interface shear stress profiles are also very similar with the development of
shear stress at the top of the nail well defined, with similar magnitudes for both tests. The
distribution of $\tau_{in}$ extends further down the sample in test NAIL2 as stresses at the top start to
decrease.

9.6.3 Results from test NAIL4

The sample in test NAIL4 was sheared at four rates of increasing speed, starting at that
used for the previous tests of about 0.625 mm/day to assess shearing rate effects. The rates of
axial strain with time for all tests are shown in Figure 9.15.

Test NAIL4 perhaps is not ideal for the purpose of discussing the effect of varying the
rate of shearing because its behaviour is not representative of that observed in tests NAIL2 and
3 during the initial period when conditions for all three tests were the same. The reasons for
this are associated with the value of deviator stress at the start of shearing, as discussed at the
beginning of Section 9.6.

The development of force and shear stress along the nail in test NAIL4 is thought to be
considerably reduced from the other tests because the sample had a much larger deviatoric stress.
acting on it at the start of the stage, compared with the samples of tests NAIL2 and 3. The initial stress state of the sample might also be responsible for the fact that the drop in shear stress towards the top of the nail, observed in the other two tests, was not evident in this test.

As a consequence, the effects of varying the shearing rate are only discussed relative to the behaviour observed in this particular test.

Considering the behaviour before the shearing rate was increased, the results in relation to the start of the reloading stage are shown in Figures 9.24 to 9.26. The sample exhibits the same trends observed with the other two samples and is consistent with behaviour predicted using the idealised models (e.g. as shown in Figure 8.34f). Tensile axial force increases towards the base of the nail and the shear stresses are greatest towards its free end. The relationship between the initial deviator stress in the sample and the shear stresses mobilised is quite clear. The small drop in axial force in the nail observed in tests NAIL2 and 3 is not evident.

As the shearing rate is increased to the second speed, five times that previously applied (i.e. 3.125 mm/day), there is a slight initial increase in axial force followed by a period when the deviator stress continues to increase by about 5 kPa without any further response from the axial force devices (see Figure 9.24a). This period corresponds to a temporary pore pressure drop that occurred immediately after changing speed, as shown in Figure 9.14, and reflects the overall sample response to the increase in shearing rate. Once the pore pressure starts to increase again, there is a small decrease in axial force, of similar magnitude to the previous increase, before the axial forces start developing again.

The rate of development of axial force with applied deviator stress, shown in Figure 9.24a, is reduced from that previously. There is virtually no change in axial force at the upper position (square symbols), consequently there is no further mobilisation of interface shear stress over the upper part of the nail. By the end of the period of shearing at the second speed, there are no further changes in axial force at the mid-position. However, similar responses are also observed at a corresponding level of strain and deviator stress in tests NAIL2 and 3.

When expressing the results against axial strain, as shown in Figure 9.25, the changes in response are not so detectable, probably because of the logarithmic scale used. There is little deviation from the original paths except at the very start of the period, the same applies to the \( \tau_{\text{int}} \) distributions.

The second shearing rate was applied over an axial strain range of about -2.5 % after which the pore pressures had risen steadily by about 15 kPa before stabilising. The increase in pore pressure indicates that at the faster shearing rate the sample is not being sheared in a truly drained condition. During the latter part of this period the development of shear stress starts to change with a reversal in the usual trends, i.e. with \( \tau_{\text{int}} \) increasing towards the base of the nail.
When the speed is increased on the next two occasions, the axial force in the nail, given in Figure 9.24a, starts to decrease at the mid and upper positions (i.e. in absolute terms rather than just rate). The force at the lower position only starts decreasing at the final speed of 13.5 mm/day. Interface shear stresses, shown in Figure 9.24b, also start decreasing at the increased speeds. However, these observations have to be compared with those from the other tests with the shearing speed held constant (NAIL2 and 3), where similar degrees of shearing also caused reductions in axial force.

Results expressed along the length of the nail, as shown in Figure 9.26, indicate the same trends as described above. By the end of the shearing stage the shear stresses increase towards the base of the nail.

The same plots but in relation to the start of the shearing stage are given in Figures 9.27 to 9.29.

The conclusion from test NAIL4 is that the speed of shearing does have an effect on the mobilisation of shear stresses along the nail interface. The greater the shearing rate the smaller the mobilisation at the top of the sample and the slower its rate of development. However, reducing shear stresses at the top has the effect of causing a greater transfer of stress down the nail towards the fixed end, although these stresses are of limited magnitude. At the very start of each phase of shearing at an increased rate, there is a decrease of pore pressure before the increase as observed at the start of shearing. During the initial decrease there is no mobilisation of interface stress along the nail.

In making comparisons with the other tests, the above observations have to be countered with the fact that the initial deviator stress was lower than in the case of tests NAIL2 and 3. This has the effect of decreasing the magnitude of the stress changes that can be developed, especially at the free end, because smaller displacements take place in reaching an equivalent stress state. The stress history of the sample and nail therefore has a significant influence on their combined behaviour.

9.7 RESULTS FROM MICROFABRIC ANALYSIS

The use of microfabric analysis for determining shearing mechanisms in clays has assisted several studies during the past thirty years at Imperial College. The technique relies on the birefringent properties of clay particles. When domains of particles are at a preferred orientation, they allow a different intensity of light to pass through them than the surrounding soil, when viewed under cross-polarised light. This subject is dealt with comprehensively by Tchalenko (1967). Tchalenko (1967) and Morgenstern and Tchalenko (1967a and b) initiated systematic methods for assessing degrees of preferred orientation and identifying and quantifying
the progressive development of fabric resulting from shearing within clay.

Martins (1983) described the development of fabric features alongside a model pile using these techniques. Various test conditions were investigated by varying the stress history of the sample and the installation method of the pile. One group of tests involved the same set-up as used here, where a model pile was installed in a 'wished-in-place' condition within a kaolin sample that had been normally consolidated at a stress ratio of $a'/a_0' = 1.5$. In this series of tests the pile was loaded up to a certain level and stopped, and samples prepared for microfabric analyses (in the same way as described in Section 5.10). Five tests were performed to investigate the disturbance caused by drilling and the shear-induced fabric from loading the pile to 70 % and 95 % of peak load, immediately post-peak and to residual conditions. There are several similarities between the test conditions in Martins' study and those imposed here (e.g. same clay, and method of installing the inclusion). For this reason and because the series of analyses is so comprehensive, Martins results are frequently referred to and comparisons drawn with his findings.

Samples were prepared for grinding to thin-sections, as described in Sections 5.9 and 5.10, from tests TNAIL4, NAIL3 and 1. These were to provide information on the degree of disturbance from drilling and grouting, and the shear-induced fabric from tests taken to just over 1 % and 9 % axial strain respectively. The author is indebted to Mr. Kevin Schrapel of Queen Mary College, London, for his expert preparation of the thin-section slides.

Preparing good quality slides was hampered for several reasons by the fact that the area of greatest interest was at the top of the sample in the vicinity of the free end of the nail. This area was susceptible to damage while removing the sample from the apparatus. In particular the polished plate was held in firm contact with the clay by suction, it was generally freed with the assistance of a wire saw. In the case where large strains were induced, further disturbance resulted from an ungrouted length of the nail being drawn into the sample as the shearing in axial extension took place. Finally, the ends of the sample are more prone to softening and cracking from stress relief, because of the lack of constraint from surrounding soil.

9.7.1 Method of thin-section observation and analysis

Morgenstern and Tchalenko (1967a) suggest several definitions, often based on those used by structural geologists, in the introduction to their work. The microstructure is said to have two main components within the context of the study of clay fabric features. Original fabric is derived from the composition of the particles and their deposition history, while shear-induced fabric results from post-depositional shear strains. Both types of fabric have a significant influence on the deformation and strength properties of clays.

Within the context of this nailing study the original fabric should be essentially the same
for all samples, as they were all prepared using a high water-content slurry, consolidated in the same way in two stages: one-dimensionally and then isotropically as described in Chapter 5.

Shear-induced fabric is divided into three types: kinematic, sequential and mechanistic. The *kinematic discontinuities* are divided into *displacement and strain* discontinuities, the former is where there is no strain effect in displaced domains either side of the discontinuity and the latter has a more strain dominant effect (*i.e.* a deformation). The order in which the fabric features appear is given by the *sequential* term, for example the bedding fabric is generally referred to by $S_j$. *Mechanistic* fabric is that related to a stress of known orientation and magnitude which acts on it. Examples of mechanistic fabric features are Riedel shears, thrust shears and principal displacement shears. These terms and others are described as they are used in the following discussions, often with reference to Martins' (1983) work where a greater variety of fabric features were observed that in the case of this study were inhibited by the imposed boundary conditions.

The *observation scale* should be specified when describing fabric features as, depending on the level of magnification, it is possible to reduce a displacement discontinuity down to a system of strain discontinuities and *vice versa*. Tchalenko (1967) refers to the microscopic scale as that viewed through an optical microscope at 10 to 1000 times, and the macroscopic scale as that visible with the naked eye.

A standard geological microscope (as described by Tchalenko (1967)) was found to give too great a magnification to assess the overall pattern of the shear-induced features, necessary for establishing the mechanisms operating within the samples in this study. Typically magnifications of five to ten times were required. The thin-sections were viewed and photographed under cross-polarised light in two ways. The slide was placed on a polarising sheet above a light table and a polarising filter, acting as the analyzer, fitted to the lens of the camera through which the slide could be viewed and photographed at a one-to-one scale. Magnification was achieved by enlargement during the printing process. Photographs produced in this way give reasonable resolution up to about five times magnification. Greater magnification, up to about ten times, was achieved using a photographic macro-enlarging system where the distance between the camera and the analyzer can be varied. The author is indebted to Mr. Richard Packer for his patience, skill and care in producing the photographs from the thin-sections.

### 9.7.2 Assessment of fabric disturbance from drilling and grouting

Several trial tests were performed on 100 mm diameter samples, to test nail installation techniques and grouting procedures. The samples were trimmed from clay cakes formed in the same way as those used in the main testing programme (Section 5.7). Test TNAIL4 was carried out to assess the degree of disturbance caused by drilling and reaming the hole for the nail and
grouting in a dummy nail.

Longitudinal and cross-section views of the soil adjacent to the nail are shown in Figure 9.30a and b at a magnification of five times. Corresponding sketches interpreting the observed fabric are given in Figures 9.30c and d, reference is made in the following text to the numbered annotated notes given in these figures.

The first fabric to consider is the bedding resulting from the method of sample preparation. Tchalenko (1967) quotes a linear shrinkage figure for Carbowax 6000 of about 8% and says that an additional internal shrinkage of 0.5 to 1.0% can occur with a high degree of bulk preferred orientation. The internal shrinkage causes cracking which forms parallel to the planes of greatest preferred orientation. Martins (1983) points out that the fact these cracks generally do not contain traces of Carbowax and are not observed near or at the outer faces of the samples, suggests that they form during cooling after removal from the bath.

The bedding planes observed in Martins’ slides are generally linear horizontally, parallel and regularly spaced. The $S_1$ bedding fabric seen in Figure 9.30 (denoted 2 and 2a) is not as uniform as that of Martins; it is generally horizontal or sub-horizontal and some curvature is evident. This results from the 'scoop-fulls' of slurry placed during filling of the oedometer. Even at the high water content used (130%), the placement method at the first stage has significantly affected the $S_1$ bedding structure. Martins prepared his samples in a 100 mm tube into which he could pour the slurry and then gently rotate the tube from side to produce an even surface at stages during filling. This is a useful observation for future microfabric studies. Although the bedding has little influence on the shear-induced fabric, linear horizontal planes within the sample are useful markers for investigating modes and magnitudes of movements. Martins was able to determine the degree of internal plastic straining at different locations away from the edge of the pile using the $S_1$ features, the reliability of such measurements decreases with curved planes.

Different scoop-fulls of slurry are evident from shrinkage cracks of between 0.15 and 0.25 mm width (denoted 2). Smaller shrinkage cracks can be seen within the 'intact' material, often these are not continuous (denoted 2a). These smaller $S_1$ cracks are comparable to those seen in Martins’ samples but to a lesser intensity, probably because most of the shrinkage is taken up by the larger crack system. The varying orientation of the scoop-fulls is evident from the different shading of areas within the slide. Small degrees of disturbance from the drilling operation are evident from the lighter areas adjacent to the nail boundary, extending no more than 0.5 mm into the sample (denoted 1). A faint 'wispy' structure can be detected within some parts of this area both in the longitudinal and cross-section views (denoted 3). The degree of disturbance is considered to be
negligible in relation to the shear-induced structures observed in the other thin-sections.

9.7.3 Assessment of shear-induced fabric after $-1\%$ axial strain

Shearing in test NAIL3 was stopped after an axial strain of about $-1\%$ to investigate the extent of fabric features that had developed shortly after a temporary drop in shear stress was observed towards the top of the nail, as discussed in Section 9.6. The degree of shearing is perhaps better considered in terms of the displacement of soil relative to the nail at its free end, which in this case is about 2 mm. This is the maximum displacement, which diminishes along the length of the nail to zero at its base.

One of Martins' tests was stopped after 1.4 mm displacement, representing 70% of peak load for the model pile, the microfabric observed from this sample provides a useful reference for comparison. The principal difference between Martins' tests and those discussed here is that uniform displacement was imposed along the whole length of the pile in his case.

Longitudinal and cross-section views from the slide from test NAIL3 are shown in Figure 9.31a and b, with again corresponding interpretative sketches in c and d. Both views are from the top of the sample where shear-induced fabric features are considered most likely to develop.

There is a light band directly adjacent to the nail over its upper length of less than 0.5 mm width (9) which stops abruptly at one of the $S_1$ bedding planes (denoted 4). The light band implies that some plastic straining has occurred in this vicinity. The fact that the band has only developed at the top, extending over one or two of the zones enclosed within the more major $S_1$ features implies that interface shearing has principally occurred in a localised area above one of the bedding planes, the zones of this area are labelled Z1 and Z2 in the interpretative sketch in Figure 9.31c. The lighter areas in the vicinity of the major $S_1$ features could also be an indication of shearing along them, pointing to a block type movement. Movement along the bedding plane at the base of zone Z1 is substantiated further because it is infilled with Carbowax, indicating perhaps that it was active during shearing and does not constitute a shrinkage crack.

The soil at the very top of the sample away from the nail has a uniform grey shading, indicating no shearing. It is not possible to assess the effectiveness of the lubricated ends because it was necessary to cut the soil immediately beneath the polished plates in order to remove them from the sample, as explained above. A very thin band of disturbance would be expected from movement alongside the 'lubricated' boundary which would have been removed with the above procedure. The sample was also additionally trimmed to expose the grout around the nail. However, evidence of how the soil slid over the polished surface is provided from the sample dimensions measured at the end of the tests which indicate that sliding took place. A
detail of the vicinity at the base of the sample after shearing in test NAIL0 is shown in Figure 5.5b which illustrates the effect.

Below zone Z2 there are no shear-induced fabric features visible except the occasional small area of disturbance caused by the drilling operation. There is a possibility that the band itself has resulted from drilling, but the degree of disturbance is greater than that observed in Figure 9.30a and the way in which it stops sharply at a point is not characteristic of a process of continued shearing resulting from inserting and retracting the drill or reamer several times.

In Martins' slide sheared to 1.4 mm displacement, a similar band exists along the whole length, because of the constant displacement imposed. He observes that within the band there are a number of 'nuclei' sites from where penetrative S₂ structures' might initiate. Within Martins' section there are at least four penetrative S₂ discontinuities at an early stage of development visible, these are usually referred to as Riedel shears.

A Riedel shear at an early stage of development is visible adjacent to the upper part of the nail in Figure 9.31a (denoted 6), there is also a faint trace of another above it (denoted 7), these are estimated to be within the top 25 mm of the sample. The more prominent Riedel is at an angle of about 15° to the axis of the nail and extends to about 1.5 mm from it. These values are similar to those measured by Martins. It might be considered that because of the axi-symmetric nature of the boundary conditions, the Riedels form cone surfaces. The cross-section view from a horizon near the top of the sample, shown in Figure 9.31b, indicates that they are only partly developed at this stage.

The light band, where plastic straining has occurred, is visible in the cross-section as an annulus of about 0.5 mm width immediately adjacent to the grout (which has remained intact in this slide). There appear to be two main circular surfaces within the light band (denoted 13), which extend significant distances around the nail but which are not continuous. The exact nature of these features is not known. The S₂ Riedels discussed above (denoted 6 and 7), propagate from this zone of plastic straining. The term penetrative discontinuity is used by Martins to describe the Riedels, in the nailing case this does not seem appropriate because the boundary conditions do not allow the formation of a group of shears, they are confined to the soil beside very top of the nail.

In addition to the Riedels there are two prominent discontinuities radiating out roughly perpendicularly from the nail by about 6 to 7 mm, which show signs of plastic straining along their edges, (denoted 11). The cause of these discontinuities is not certain but their orientation

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1Tchalenko (1967) defines *penetrative discontinuities* as a configuration of discontinuities repeated on a small scale throughout a portion of a body. Conversely, *nonpenetrative discontinuities* are unique or do not form a family, such as a displacement discontinuity alongside a pile.
indicates that a localised tensile type of failure has occurred. The reasons for this apparently
tensile failure are not obvious. Discontinuities at this orientation are not evident in the slide
from the sample sheared to large axial strains (9.4 %). This indicates that either they were
induced by a mechanism peculiar to the conditions of test NAIL3 or they are a shear-induced
feature that occurs very close to the top of the sample where the nail passes through the platen
and that this material was trimmed off in the sample prepared from test NAIL1.

An additional thin-section slide was prepared from the lower part of the sample from
test NAIL3 which confirms that there are no visible signs of shear-induced fabric below those
described above. Photographs from this slide are not presented.

9.7.4 Assessment of shear-induced fabric at large strains

Slides were prepared from the top of the test NAIL1 sample after shearing to about -9.4
% axial strain, representing in this case about 13 mm maximum displacement between the soil
and the nail. Longitudinal and cross-section views from the slide are shown in Figure 9.32a and
b, with corresponding interpretative sketches in c and d. Both views are from the upper part of
the sample where shear-induced fabric features are considered most likely to develop.

Two tests performed by Martins can be used for comparison: one sheared just post-peak
after about 2.5 mm and another, similar to the NAIL1 test, sheared to residual conditions after
10 mm displacement. Martins observes that there is a considerable growth of penetrative $S_2$
discontinuities associated with peak conditions. In his slides they are spaced at regular intervals
of 10 to 15 mm along the length of the pile. The $S_{2a}$ structures, referred to as Riedel shears
degenerate into $S_{2b}$ reverse kink bands (as defined in Morgenstern and Tchalenko 1967b, p324).
These reflect an unstable mode of deformation and appear as a region of soil undergoing simple
shear deformation separated from the surrounding soil by two almost parallel discontinuities.
The $S_1$ bedding structures, rotate within the kink band, but remain at roughly the same
orientation either side of it. Because of this rotation, only a certain degree of relative
displacement either side of the band can take place kinematically, Martins estimates this to be
about 1 mm.

A new structural $S_3$ set of discontinuities forms in response to the limited admissible
deformation that can take place with the $S_2$ features. Initially $S_{3a}$ structures, known as thrust
shears, are formed, but these also have limited freedom of movement. They are consequently,
followed by $S_{3b}$ en-echelon displacement discontinuities, lying sub-parallel to and very close to
the pile. These kinematically allow large displacements to occur, with resulting high local shear
strains and enhancement of the local degree of preferred particle orientation.

Most of the features described above are still evident in the slides from the sample
sheared to 10 mm displacement. Most of the pile movement is accommodated within the $S_{3b}$
structures which eventually coalesce to form a thin continuous principal displacement \( S_1 \) discontinuity close and parallel to the pile along which large displacements can take place.

Martins estimated the magnitude of plastic deformations occurring within the clay by measuring the deformation of the bedding fabric \( S_1 \) using the shrinkage cracks as markers. He suggests, in conclusion from his study of the microfabric of the wished-in-place pile tests, that pre-peak pile displacements are largely accommodated by plastic straining within the clay, whereas post-peak displacements involve shearing within a very narrow zone close to the pile. The Riedel shears do not seem to be disturbed by the large post-peak displacements.

The results from Martins' tests have been described to introduce the sequence of structures that develop with increasing shear displacement imposed along the length of the interface as the pile is loaded. These provide a useful reference against which to compare the results from the nail tests, where the magnitude of displacement varies from a maximum at the top to zero at the base from the nail being indirectly loaded from shearing the sample.

Considering the longitudinal section shown in Figure 9.32a and interpreted in Figure 9.32c, several features are visible adjacent to the nail towards the top of the sample. A Riedel shear can be seen extending to about 18 mm from the top of the slide (denoted 6). This marks the lowest point at which shear-induced interface fabric can be seen except some traces of localised disturbance from the drilling operation. The Riedel is at an angle of about 30° to the axis of the nail and extends to about 5 mm from the grout, which is visible in this slide (the grout intrusion into the three shear keys is clearly seen just below this point). Another similar shear plane is visible just above the lower one at the same angle but is not connected.

At the very top of the sample in the vicinity of the nail, there are two families of \( S_{1b} \) kink bands fanning outwards from common points, over a range of angles up to about 35° (denoted 14). Reverse kink bands are clearly visible in both fans where they are intersected by \( S_1 \) features. A further enlarged view of this part of the slide is given in Figure 9.33. No attempt has been made at quantifying the degrees of plastic straining in the way that Martins did because of the curved nature of the \( S_1 \) bedding planes. Also the range of angles at which the kink bands are orientated indicates that larger scale sample shearing occurred at greater strains thus distorting the fabric developed at the interface at an earlier stage.

Features similar to the Riedel shears seen in the fabric from test NAIL3 (Figure 9.31b) are much more evident in the cross-section view shown in Figure 9.32b and d. The circular surfaces extend up to 5 mm from the edge of the soil in this case and are almost continuous (denoted 6 and 14). It is not clear from this view whether the discontinuities visible are of \( S_{2a} \) or \( S_{2b} \) structures although they are slightly thicker than those from test NAIL3 which might indicate that they are of the latter. These almost continuous circular features indicate that at this
stage, fabric in the form of the outer surface of a conical frustum has developed, whose upper diameter is slightly larger than that of the outer edge of the grout. The disturbed area seen within the 0.5 mm annulus around the grout in Figure 9.31b is not at all visible in the large-strain cross-section. It is thought that the grout and soil immediately next to it must have been dislodged during the thin-section grinding.

An overall mode of sample deformation is evident from the longitudinal section, this is shown full size at a reduced magnification in Figure 9.34a with an interpretative sketch in 9.34b. On careful inspection a series of shears can be connected along and within a diagonal strip starting from the lower part of the sheared interface zone, heading down towards the outer part of the sample (see Figure 9.34b). This suggests that shearing is taking place on a cone-shaped surface whose apex is towards the upper part of the nail. Further evidence comes from the observation that the fabric below this strip is of uniform shading indicating no shear straining. The fabric at the very top of the sample away from the nail is in a similar state, as was noted for the case of test NAIL3.

Detailed analysis of the photographs taken at the beginning and end of test NAIL2, which had the the letraset grid marked on the membrane, see Figure 5.2, have not been made. It is unlikely that these features, only just visible from the thin-section, can be detected from the membrane boundary.

No sign is evident of the 0.5 mm wide band of interface shearing seen over the upper two zones of the sample from NAIL3 seen in Figure 9.31a and c. Possibly in this former case there was an initial preferred mode of shearing caused by the orientation of the zones between the S1 bedding planes, such that zone Z2 moved along the nail as a block.

The mode of movement indicated in Figure 9.34 is also corroborated by measurements made at the boundaries of the sample from test NAIL1. At the end of most of the tests the diameter and height of the sample was measured at several points using a vernier calliper. The diameter measurements are expressed in Figure 9.35a as radial strains over the height of the sample, these strains are from the shearing stage alone. The greatest radial displacements, which are all inward, generally occur between a quarter and a third of the way down the sample, they are about twice those at the base and significantly larger than those at the top.

In Figure 9.35b the height and the diameter measurements from test NAIL1 are combined and expressed as displacement vectors of points along the outer boundary of the sample. The greatest inward movements are sub-horizontal and could be associated with movements along a conical failure surface emanating down from the free end of the nail. Note that the vertical components of movement are governed by the movement of the lower platen, the upper platen being fixed to the top of the triaxial cell.
The microfabric analyses at large strains therefore indicates that the relative
displacements alongside the nail, even at the top, are insufficient to generate $S_3$ or $S_4$ structures.
The imposed boundary conditions, particularly fixing the nail at the base so that there is no
relative soil to nail movement there, have restricted development of interface shear fabric to the
top 25 to 30 mm of the nail. At large strains overall sample failure has occurred.

9.8 INTERPRETATION OF RESULTS

The results presented in the earlier sections are based on the measurements of axial force
made with the instrumented nail. Interface shear stresses have been calculated between the
points of measurement and also between the upper device and the top of the nail where axial
force has been assumed to be zero.

In the sand tests, measurements of radial stress as well as axial force were made. This
allowed the effects of three structural phenomena to be assessed in the context of the
experimental results: rigid body inclusion effects, arching, and restrained dilation. Clearly, it
would be beneficial to extend the discussion of these effects to the clay particle size.

The results from an interface shearbox test reported by Martins (1983) are used to
estimate ranges of values of interface radial stress, $\sigma_r$, and/or alternatively the angle of interface
friction, $\delta'$. The likelihood of any of the three phenomena occurring in the clay soil can then
be assessed.

9.8.1 Interface shearbox tests on kaolin

Martins (1983) performed interface shearbox tests to establish the interface friction angle
under known normal stresses. The results were then compared with values obtained from his
model pile study and used as part of the analysis. Rate effects at residual conditions were also
studied using the ring-shear apparatus.

In the direct shearbox tests, the interface was of polished epoxy-resin with grooves of
0.5 mm width at 1.5 mm centres. Kaolin slurry was consolidated against the interface and
shearing carried out under a normal stress of 250 kPa at a rate of 0.015 mm/min (21.6 mm/day)
to allow full dissipation of excess pore water pressures. The nature and conditions of the test
make it ideal for application to the results from the model nail tests performed in kaolin in this
study.

The results from a typical test are reproduced in Figure 9.36, expressed in terms
interface friction angle versus relative displacement. Martins observed that the peak angle of
interface shearing resistance was mobilised after about 1 mm of displacement, and for the series
of tests performed, was typically within a range of 17° to 19°. Residual strength was reached
after about 2.5 mm in the test result shown, but the required displacement typically varied
between 3.5 and 7 mm, tending to increase with surface roughness. Values of residual interface shearing resistance from the test series varied from 10.5° to 12°.

Ring shear tests carried out in conjunction with Lupini (1981) indicated that the residual angle of friction increases with the rate of shearing. The variation of $\delta'_r$ over the shearing rates used in this study (test NAIL4) ranges from 11.3° to 15.1° which corresponds to the rates used of 0.625 to 13.5 mm/day.

9.8.2 An approach for estimating interface radial stress

The different sections of the test curve from the interface shearbox test were modelled by curve fitting techniques or as straight lines as shown in Figure 9.36, where the equations of the curves used are given. Another model was considered, for reasons explained later, where the friction angles were arbitrarily increased by 5° as indicated in the figure.

The approach used, in applying the interface test results, is to estimate the relative soil to nail displacement at the positions along the nail where the interface shear stresses have been determined, and obtain a value of interface friction angle, $\delta'$ from the modelled curve given in Figure 9.36. Knowing $\delta'$ and the interface shear stress, $\tau_{int}$ enables radial stress at the interface to be calculated using the traditional Mohr-Coulomb equation (expressed in equation 8.12).

$$\sigma_{r\,int} = \frac{\tau_{int}}{\tan \delta'}$$  \hspace{1cm} (8.12 bis)

Equation 8.12 has been used to produce a series of lines representing the linear relationship between $\tau_{int}$ and $\sigma_{r\,int}$ for different values of $\delta'$ as shown in Figure 9.37. Even without knowing the displacements, the curves give an indication of the magnitudes of interface radial stress to expect for typical ranges of $\tau_{int}$ using values of $\delta'$ obtained from the shearbox test.

Relative soil to nail displacements are only known with certainty at two positions. At the top of the nail, where they are represented by the movements measured by the internal LVDT (explained in Chapter 5), and at the base where the nail is fixed to the platen and they are zero. For simplicity a linear distribution of displacement down the length of the nail has been assumed in estimating values at the three required positions.

One of the objectives during the stages prior to shearing was to reapply stresses whilst minimising axial strains. In order to assess mobilised $\delta'$ values during these stages with small displacements, the initial non-linear part of the curve leading to peak has been modelled closely. The approximation regarding the linear displacement distribution is clearly an assumption. During the stages leading to peak it provides an upper limit to the value of $\delta'$ that might be mobilised along the interface of the nail. Consequently, the worst case condition is assessed when considering the mobilisation of shearing resistance in terms of fabric damage.
Several assumptions are made in this process of estimating interface radial stresses and additional ones are discussed in the context of the results in the next section. The main objective of the interpretations being made is to obtain further information indirectly, albeit in a very approximate manner, about conditions at the interface. The intention is to supplement the results from direct measurement and to allow an extended discussion of the phenomena observed in the sand tests to be made in the context of a clay. The assumptions and necessary approximations are covered here to illustrate that they have been considered and to allow the accuracy of the estimations to be judged.

9.8.3 Interpretation of results from stages prior to shearing

The purpose of the model is primarily to provide estimates of radial stresses acting at the interface during the shearing stage. It is also used to estimate the degree of shearing resistance mobilised during the stages prior to shearing.

Estimating the degree of mobilisation of angle of interface friction prior to shearing, allows an assessment to be made of possible disturbance caused to the clay fabric during this time. As described in Section 9.7, Martins (1983) performed a series of tests, which were stopped at various stages leading up to peak and then carried out microfabric analyses. His results indicate that relative displacements leading to about 70% mobilisation of peak resistance, result in only very localised plastic shear straining within the clay, and that no penetrative discontinuities are present.

The magnitude and sense of displacements during the initial stages of the test should also be considered in relation to the sense of final shearing. During the reloading and reconsolidation stages the sample and nail go into tension followed by compression. If displacements are significant, changes in the shear stress sense will occur prior to shearing and the effectiveness of the model will reduce because of phenomena such as the Bauschinger effect influencing the results.

Displacements from both the reloading and reconsolidation stage have been combined to assess the effects discussed above. In all three tests the cumulative displacement was compressive and within a range of 0.23 to 0.38 mm. The mobilised friction angles were therefore in the opposite sense to those during shearing and from the model shown in Figure 9.36 within a range of 8.8° to 9.6° representing less than 60% of the peak value.

Given that the displacements considered are at the very top of the nail where they are maximum and that they are below levels likely to cause damage, it is assumed that the axial strains that occurred during the stages leading up to shearing had a negligible effect on the

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2The Bauschinger effect is that where a reduction in yield stress occurs as a result of loading in a direction opposite to the previous loading.
sample fabric. Moreover, it is recognised from microfabric studies performed in conjunction with shearbox and ring shear testing, that significant reorientation of clay particles only occurs after peak conditions (Lemos, 1997).

9.8.4 Interpretation of results from the shearing stage

In view of the discussions above, the displacements during the shearing stage are assumed to start at a zero datum for the purposes of the estimation of radial stresses.

The same specific points chosen to represent conditions during the shearing stage, (i.e. those corresponding to the lines given in Figure 9.29 showing the interface behaviour along the length of the nail), are used for applying the model. The relative soil to nail displacements assumed along the length of the nail at each of these times are given for the three tests in Figure 9.38, (the lines are compatible with those in Figure 9.29). The estimated values of mobilised $\delta'$, corresponding to these displacements using the modelled curve in Figure 9.36, are also shown on the right hand side of Figure 9.38.

In tests NAIL2 and 4 friction angles increase to peak and then decrease as sufficient strains were imposed for residual conditions to be reached by the end of the stage, according to the model and assumptions used. In test NAIL3, stopped after about $-1\%$ axial strain, friction angles increase to peak by the end of the test.

The estimated values of interface radial stress are given in Figure 9.39 in the diagrams on the left hand side. No account is taken of the radial stress acting on the nail from the applied boundary stresses, i.e. the cell pressure. The radial stresses shown are those that act on the interface, according to the Mohr-Coulomb equation, in order for the measured shear stresses to be generated by the assumed displacements.

The estimated radial stresses towards the top of the nail seem unrealistic, they are up to about six times those applied at the boundary (i.e. 250 kPa) in the cases of tests NAIL2 and 3. Further down the nail at the lower two positions, values are within twice those applied at the boundary. Because of the magnitude of these estimated radial stresses, the model was slightly modified by arbitrarily increasing the interface friction angle by 5° at all displacements, as shown in Figure 9.36. Radial stresses calculated using this model are given on the right-hand side of Figure 9.39: the values shown are still very high but are more credible.

Before discussing the results and the reliability of the method of modelling, an alternative approach to the problem is made. A value of interface radial stress is assumed and angles of interface friction angle calculated using the same Mohr-Coulomb relationship. In this way calculated values of $\sigma_{r, int}$ and $\delta'$ can be assessed by fixing the inter-related counterpart parameters (measured interface shear stresses and the assumed displacements are fixed in both cases). Another way of expressing the results would be to present the measured $\tau_{int}$ values.
against the assumed displacements along the nail boundary. These values constitute the product 
\[ \sigma_{r,\text{int}} \tan \delta' \] from which either component can be assessed holding its counterpart constant.

Two values of \( \sigma_{r,\text{int}} \) are considered, that of the applied boundary stress of 250 kPa and an elevated value of 300 kPa (calculations using elastic analyses, discussed in Section 7.5, indicate that this value could arise from rigid-body inclusion effects). Angles of interface friction calculated using these values are shown in Figure 9.40. The magnitude of the \( \delta' \) values, obtained using a value of \( \sigma_{r,\text{int}} = 250 \) kPa, are very high compared to those observed in the shearbox tests, in some cases being in excess of 40°.

The values obtained using both approaches are credible for the mid-points of the nail but seem exceptionally high towards its free end at the top in comparison with the results from the shearbox tests. The observations from the microfabric analysis indicate that the majority of the interface shearing takes place adjacent to the nail in the upper part of the sample. The results from the interpretation made here indicate that either very high values of interface radial stress or angles of friction are generated in this vicinity.

An alternative explanation is that one of the assumptions made is incorrect, e.g. if the force in the nail at its free end is not zero, interface shear stresses at the top will be overestimated. It is thought, justified by several considerations, that the nail moves freely through the upper platen. At the end of the tests when the samples were removed from the apparatus, no sticking between the guide and the nail was observed. In the discussion of the results from test NAIL2, it is concluded that the nail was sticking for the initial stage of shearing. This was quite evident from the axial forces generated in the nail which were of the same magnitude and were developed at negligible strains; when the nail became free, the shear stresses were rapidly mobilised. It is therefore thought that if sticking were taking place it would be quite evident from the instrument outputs. Additionally, results from numerous sand tests indicated that the main interface shear stresses were occurring towards the top of the nail.

**Consideration of elevated values radial stress acting at the interface**

It is possible that high values of radial stress acting at the interface could occur from one or more of the three phenomena discussed in Chapter 8.

Rigid-body effects could partly explain the elevated values of \( \sigma_{r,\text{int}} \), although the elastic analyses made when considering cell-action effects in Section 7.5 indicated that these effects would not induce values of \( \sigma_{r,\text{int}} \) greater than about 330 kPa, also Martins’ pile tests in kaolin with measurements of radial stress showed no rigid-body inclusion effects. Arching during the the sand tests caused decreases in radial stress and is more likely to occur with the granular interlocking associated with larger particle sizes.

The remaining phenomenon is that of restrained dilation. Dilation only generally occurs
in over-consolidated clays during shearing. The samples tested in this study were initially normally-consolidated, but because of the path followed during shearing, where $p'$ is continuously decreasing, the samples become over-consolidated. However, the samples are only likely to reach a lightly over-consolidated state. It is therefore difficult to ascribe the cause of the increased radial stress to restrained dilation unless shearing against the interface induces dilation very locally or unless it can be shown that again locally, the clay has become over-consolidated to a greater degree.

Research into the behaviour of displacement piles in clay provides insight into what could have caused a localised consolidation of the clay around the nail. Morrison (1984), who performed model pile tests in Boston Blue Clay, suggested that a clay layer or cake formed around the pile shaft during pauses between jacking strokes, when excess pore water pressures, generated during interface shearing, dissipated. Chow (1997) observed similar behaviour during installation of the Imperial College instrumented pile into lightly over-consolidated medium to low plasticity silty clay at Pentre. Pore water pressures increased or decreased depending on whether interface shearing was taking place in fresh or pre-sheared soil. On pile extraction, a 10-mm thick clay layer was found strongly adhering to the pile shaft, which was thought to have formed during the pauses in jacking when pore pressure dissipation and the radial migration of water created a zone of stronger clay next to the pile.

This reported behaviour might be related to conditions in the model nail test where a limited degree of shearing has occurred during the reloading and reconsolidation stages prior to shearing. The nail instrumentation and microfabric studies indicate that the majority of this shearing took place at the top of the nail. This might suggest, by the hypothesis given above, that increased consolidation takes place in this vicinity, corresponding to where the greatest increase in radial stresses during shearing is observed.

Although local pore pressures in the vicinity of the nail were not measured, the samples from tests NAIL2 and 4 were dissected after shearing. Moisture contents taken at varying distances from the edge of the samples (of radius 76 mm) and at different levels within them provide evidence that different amounts of pore water have drained from the sample in localised regions. These measurements are summarised in Figure 9.41. As the samples were taken at the end of the tests, the distributions shown also include the effects from the shearing stage. In the case of the sample from test NAIL2 there is a clear trend of lower moisture contents adjacent to the nail. Values of $w$ throughout the sample increase towards the base, the lowest values of $w$ are therefore in the vicinity of the free end of the nail.

The deductions above indicate that the high values of interface radial stress interpreted could possibly have been induced by the phenomenon of restrained dilation. However, in the
absence of radial stress measurements along the nail, this behaviour cannot be proved conclusively and must be considered to be tentative.

**Consideration of elevated angles of interface friction**

Studies made by other researchers into the behaviour of soil adjacent to excavations have shown that very high angles of internal shearing resistance are generated under conditions of passive stress relief (e.g. Burland and Fourie, 1985; Ninis, 1997). In some cases apparent $\phi'_v$ values of up to 70° have been measured in clays. It is postulated that a similar effect could occur with $\delta'$, the angle of interface friction, acting alongside the nail at the top of the sample where the greatest degree of stress relief is taking place in the vicinity of the free end of the nail. Similar behaviour was discussed in the context of the sand tests.

As the observations regarding elevated interface friction angles can be related more readily to observations made by other researchers, it would seem reasonable that of the two components given in equation 8.13, it is most likely that the $\delta'$ value is causing the very high interface shear stresses.

The results from the interpretations made in this section, undertaken to provide a more overall picture of the stresses acting along the interface of the nail, are discussed in the context of the measured values and the findings from the microfabric studies, in the conclusions in the next section. The possible occurrence of phenomena identified in the sand tests is also investigated.

### 9.9 DISCUSSION AND CONCLUSIONS FROM THE CLAY TESTS

The results from five tests performed using normally consolidated kaolin samples have been presented and discussed, one of these was a reference test with no nail. Three of the remaining four were performed with an instrumented nail capable of measuring axial force at three points along its length. Two of the samples were prepared for microfabric analysis.

**Sample preparation**

The method used to prepare the sample from a slurry by filling a large oedometer with a scoop, resulted in a curved, non-parallel bedding structure which hindered detailed microfabric analysis. It is recommended that the slurry be poured into the oedometer, possible at a higher water content than twice the liquid limit as far as is practicable. The mechanical behaviour of the soil is thought to have been unaffected the bedding structure.

**Stages leading up to shearing**

Microfabric analysis indicates that minimal fabric disturbance occurred in the stages leading up to shearing. The same conclusion can be reached from consideration of an interface shearbox test result combined with microfabric analysis performed by Martins (1983).
Maximum displacements leading up to the shearing stage are less than 0.2 mm in all nail tests, at which less than 60 % of the peak angle of interface friction, \( \delta'_{\text{peak}} \) is mobilised. The thin-section analysis indicates that no significant fabric development occurs before 70 % of \( \delta'_{\text{peak}} \).

During the reloading and reconsolidation stages after nail installation, results from the instrumented nails indicate that the nail went into tension and compression respectively. Most of the interface shear stresses, \( \tau_{\text{int}} \) mobilised are towards the top of the nail. The detailed behaviour of the nail is complex and not easy to interpret because of residual stresses from the installation stage. Results from the shearing stage are therefore considered in relation to the start of the shearing stage as well as the reloading stage.

**Behaviour during shearing up to displacements of 2 mm**

During shearing the sample underwent a small initial dilation followed by steadily increasing contraction, the degrees of which depended on its initial stress state and moisture content. The presence of the nail marginally increased the peak angle of shearing resistance by about 2°, although this result is based on only one reference test.

Tensile forces within the nail continuously develop during shearing, with a maximum value at the fixed base, decreasing to zero at the top of the nail. Interface shear stresses rapidly develop at the top of the nail where they are almost twice those further down. At axial strain levels of \(-0.05\) to \(-0.2\) % (corresponding to displacements of about 0.1 to 0.3 mm) there is a sudden drop in the shear stress at the upper part of the nail. The drop is not large but is significant because thereafter the build-up of force continues at a slower rate; after about \(-1\) % strain, \( \tau_{\text{int}} \) at the top starts decreasing and there is a greater transfer of stress further down the nail, although not of significant magnitude.

Several reasons for the drop might be postulated. In terms of the Mohr-Coulomb relationship, a reduction in \( \tau_{\text{int}} \) could be explained by a decrease in \( \delta' \) or \( \sigma'_{\text{int}} \). Neither of these seem likely. The interface shearbox results indicate that a post-peak reduction in \( \delta' \) will only occur after much further shearing and displacements. Similarly there is no apparent reason for the radial stresses to decrease. Results from the numerical analyses described in Chapter 4 indicate in fact that radial stresses increase, and at the same time circumferential and axial stresses in the localised area around the top of the nail decrease.

The time at which the drop takes place corresponds to when the overall sample volume strain is changing from a dilatant to a contractant sense, *i.e.* the magnitude of the radial components of volume change become greater than those of the axial components. A reason for the drop in relation to this observation is not obvious either.

The microfabric analysis of soil from a test stopped after about \(-1\) % strain show two fabric features that might be related to the drop in shear stress. Riedel shears at the top of the
sample are just starting to develop from the interface. However, they are at an early stage of development. Fabric features that appear to be more significant are vertical shears seen radiating perpendicular to the nail boundary. Their formation could be associated with a localised tensile failure, but how this could be induced under conditions of increasing radial stresses in combination with decreasing circumferential and axial stresses (indicated by the numerical analyses described in Chapter 4) is not clear.

The high magnitude of the interface shear stresses acting at the top of the nail become apparent when they are interpreted in combination with results from interface shearbox tests performed by Martins (1983), making very approximate estimates of the soil displacements taking place along the nail. Assuming that the interface shear stress, $\tau_{int}$, is governed by the product of $\sigma'_{int}$ and $\tan\delta'$ (as given in equation 8.13), it can be observed that either very high values of interface radial stress or interface friction angle (or a combination of both) are acting at the free end of the nail.

The simple analysis, based on the shearbox results, indicates that radial stresses developed at the upper part of the interface are much greater than those applied at the boundary of the sample, after displacements of less than 0.3 mm. The magnitude of the radial stresses interpreted at the top of the nail is two to four times that at the sample boundary, depending on the interface model used. These might be explained by a localised over-consolidation in the vicinity of the top of the nail, allowing the phenomenon of restrained dilation to take place. However, this explanation must be considered very tentative at the best, given the limited present evidence of this phenomenon and the lack of actual measurements of radial stress in the experiments.

Further down the nail, the maximum interpreted radial stresses are similar to those applied at the sample boundary. Martins (1983) performed three tests with a radial stress measuring devices incorporated in the pile. Values measured did not exceed those at the sample boundary and he suggests that radial stresses at the interface, after reapplication of cell pressure following installation of the pile, are about the same as those applied.

The alternative explanation for the high $\tau_{int}$ values could be very high $\delta'$ values. Other researchers have recorded very high values of angle of internal shearing resistance during passive stress relief tests at very low stress levels. As the soil in the vicinity of the free end of the nail is undergoing very large degrees of stress relief it seems reasonable that similar effects are occurring with the angle of interface friction $\delta'$. This explanation might also be reinforced by considering the high degree of principal stress rotation taking place in this area as was discussed in the context of the sand test results in Chapter 8 (i.e. a contractant behaviour is observed which causes a localised arching, consequently applied radial stresses are not transmitted to the nail.
In conclusion, the behaviour at the time when a drop in shear stress at the top of the nail occurs, cannot be readily explained by the measurements made or consideration of conditions within the apparatus. It seems likely that it could be associated with the observed microfabric features at the top of the sample but the evidence is not conclusive. However, it is thought that the very high interface shear stresses generated during the initial stages of shearing (i.e. to displacements of about 2 mm) can be explained by high angles of interface friction, possibly in combination with slightly elevated values of radial stress from rigid-body inclusion effects or restrained dilation.

**Behaviour during shearing to large displacements**

As shearing continues the progressive mobilisation of interface shear stresses diminishes. At axial strains of about —3 % further changes in $\tau_{int}$ become negligible, decreasing values at the upper position are countered by increases at the mid-position but changes are small. At the base there is a limitation to the development of shear stress imposed by the boundary condition of the fixed nail at the base. Similar behaviour is seen in the idealised models where after initial applied displacement steps, the number required to produce an increase in $\tau_{int}$ increases dramatically. Similarly the microfabric analyses show that only pre-peak displacements by plastic straining can develop at the upper part of the interface because of this imposed boundary condition. Post-peak shear-induced fabric observed in Martins’ pile tests does not develop.

It is concluded that only minimal interface shear stresses are transmitted along the nail after the initial drop discussed above. This is confirmed from the deduced $\tau_{int}$ values and the microfabric analysis from a sample sheared to an axial strain of about —9.4 % which revealed that shear-induced fabric only developed over about the upper 25 mm of the sample adjacent to the nail.

Careful inspection of this slide revealed that overall sample failure occurred with the formation of an irregular conical failure surface with its apex towards the upper end of the nail. The surface appears to have initiated from the lower part of the zone of shear-induced fabric, with its subsequent path influenced by the bedding fabric. No surface was visible from an outer inspection of the sample.

**Rate effects**

Rate effects were assessed during shearing in one of the tests, although the results are limited to the effects after axial strains of about 0.6 %. These indicate that mobilisation of $\tau_{int}$ at the top of the nail significantly decreases with increasing rate; as a result more shear stress is transferred further down the nail. Research into the rate effects of displacement pile installation (e.g. Martins, 1983; Chow 1997) indicate that the principal controlling factor is the
variation of residual strength of clay with shearing rate and its consequent effect on the mode of shearing, (see also Lupini et al., 1981; Tika et al., 1996). It is thought that residual conditions are not developed in the nail test apparatus, except perhaps at the very top of the sample, because of the imposed condition of the nail being fixed to the base platen.

The subject of rate effects is discussed further in the next chapter, where the results from the sand and the clay tests are discussed in relation to their application to field conditions and design.
CHAPTER 10

Application of results to practice

10.1 INTRODUCTION

So far the results from a series of element tests performed on sands and clay have been presented and discussed in detail within the context of the boundary conditions of the test apparatus used. Full-scale boundary value problems are notoriously difficult to interpret from model tests because of effects such as scale and the simulation of overall conditions found in the field. Nonetheless much useful information on aspects of the overall behaviour can be derived from such tests where specific variables are controlled, to allow others to be investigated. Thus at least part of the overall picture is obtained which can be added to research and field observations made by others.

In the case of this study, element tests have been performed with the specific intention of modelling conditions at the interface of a single nail. Several assumptions have been made regarding boundary conditions and modes of behaviour in the field. These have been discussed at various points within the preceding chapters, for instance when the philosophy of testing is explained or during consideration of how the boundary conditions of the test apparatus influence the experimental results.

In this chapter the results are discussed in terms of field conditions and design.

10.1.1 Applied boundary conditions

The boundary conditions to be modelled and those applied in the tests were introduced and discussed in Chapter 3. They are summarised again here prior to interpreting the results, to emphasise the field conditions being modelled.

The case of soil nailing used for the construction of excavations is being considered, in particular where drilled and grouted nails are used, although the results are probably equally applicable to other nail installation methods as well. Staged top-down construction is the method used in practice. A step is cut and a lightweight mesh and drainage material placed against the soil before a layer of shotcrete is applied. The nail is then constructed by drilling a hole, installing a reinforcing element and grouting it in place. A small facing plate is usually fixed to the reinforcing element after the grout has set and a small nominal tension applied to the securing nut. Once a row of nails have been installed, another step is excavated and the process is repeated until the required depth is reached. Thus, the upper rows of nails are progressively stressed as the magnitude of stress relief at the face increases.

Displacements at the face of the excavation occur as the excavation deepens, the largest
movements are generally at the top of the wall, both horizontally outwards and vertically downwards. As these displacements take place, shear stresses are mobilised along the lengths of the nails. The main theme of the research described herein has been to investigate the development of these stresses as they control the internal stability of the excavated structure. Each of the field boundary conditions and the method of modelling them are now discussed.

After excavating each step, the wall is constructed. The wall is lightly reinforced shotcrete; in relation to most anchored, or other structural retaining walls, it can be considered to be quite flexible. In many cases nothing further is done, sometimes a more aesthetic front facing is constructed, often in a panel form. The method of modelling the wall differs in the apparatus for clay and sand testing. In the former it is a rigid boundary and in the latter it is quite flexible. The rigid boundary imposes conditions of uniform displacement and non-uniform stress. The converse occurs for a flexible boundary, stresses are uniform but strains within the sample are not. The flexible boundary is thought to be more representative of field conditions. In designing the apparatus to perform the tests, the boundaries were chosen for practical reasons. Because of the greater propensity of granular materials to form internal structures such as arches, it is fortuitous that the sand was tested under the condition of a flexible boundary.

The next stage is the installation of the nail. Again, different procedures were adopted for the two types of material for practical reasons. The nail was installed into the clay sample using a drill and grout procedure very similar to that carried out in practice. The main difference was that boundary stresses were removed during this operation. This is not thought to have affected the model in any way but it allowed the boundaries of the sample to be controlled to facilitate microfabric analysis. In the case of the sand tests, the soil was placed around a nail that was already fixed in position within the apparatus. Although the results indicate that this caused a degree of arching, especially when it was necessary to densify the sand around the nail, similar effects might occur from radial inward movement of the soil during drilling operations in practice.

The nail in both cases is comprised of an inner steel section surrounded by an epoxy-resin grout. Both are less stiff than a cementitious grouted nail and more ductile. Throughout the processing of the data from the tests, the nail has been assumed to be rigid. This seems reasonable given that the elongation of a necked section under a load of 500 N is estimated to be less than 0.015 mm, representing an equivalent strain over the sample height of 0.01 \%, this is deemed negligible at the levels of strain in the sample necessary to induce such a load. Although this approach is satisfactory for the experimental conditions, in practice stretching of the nail could have a significant effect on its behaviour, particularly in the resistant zone. This subject has been extensively studied in relation to the behaviour of nails during pull-out tests.
(Recommandations Clouterre, 1991) and anchors (Barley 1991, 1995). The topic is discussed in the next section in the context of the behaviour of a soil nail loaded by stress relief of the surrounding soil.

The interface of an epoxy-resin nail is less rough than one formed from a cement grout. In the case of the granular material, because the sand was placed around the nail with the grout already cast on it, there is no possibility of grout bleed into the surrounding soil which is certain to happen in practice. However, having grout of a known dimension is necessary for the accurate determination of shear stress. It is also easier to assess shearing conditions on a surface of known characteristics (rather than being part grout and sand-grout conglomerate).

The compliance of the nail radially at the instrumented sections is thought to have an effect on the performance of the radial stress measuring devices (i.e. cell-action effects), but overall the stiffness of the nail is such that compliance effects are negligible. Care was taken to orientate the devices in different directions to reduce any adverse effects.

In practice the nail is always connected to the facing, even if not rigidly. The philosophy of soil nailing is that movements within the soil mass are restrained by the development of the interface shear stresses rather than being held back by the facing. The primary role of the facing is to prevent localised failure between the nails. For this reason, conditions within the apparatus were set so that the soil could move freely past the upper part of the nail. Forces at the top of the nail were essentially zero where it passed through the top platen, thus modelling the case where there is no connection between the facing and the nail. This represents a limiting boundary condition, where mobilised interface shear stresses resulting purely from the indirect loading of the nail by the soil can be assessed, rather than the case where the nail is pulled by the facing.

As excavation proceeds beneath a given row of nails, stress relief at the front face progressively increases. This has been modelled by shearing the sample, with the nail installed axially within it, in drained extension. This involves reducing the axial stress in the sample while holding radial stresses constant. The application of an all-round pressure to a circular boundary implies that horizontal and vertical stresses in the field are equal. This would rarely be the case but the test set-up has the advantage that the intrinsic anisotropic characteristics of the material in this sense can be ignored. The radial boundaries were fully flexible for both test conditions.

At the front of the excavation in the vicinity of the facing, the outward movement of the soil is greater than that of the nail because nail movement is restrained by frictional forces acting on it further back in the excavation. The magnitude of outward soil displacement diminishes with distance from the face. Towards the far end of the nail, soil movements are less than those
of the nail which is being pulled forward from the friction generated along its interface near the face. These two regions are divided by the point where the relative movement between the soil and the nail is zero. This point or plane has been used as a boundary condition in the tests, by fixing the nail to the base platen the condition of no relative movement between soil and nail is achieved. This boundary condition has a significant effect on the behaviour of the soil-nail system.

In the case of the clay tests highly polished stainless steel discs were placed against the sample ends to minimise friction. In field conditions some friction might be generated at the facing by the shotcrete, but within the soil mass at the plane of no relative movement between nail and soil none would be expected. These discs are thought to have worked well.

10.1.2 Influence of model size

Prior to discussing the results within the context of field conditions and design, factors relating to the size of the nail, the sample and the soil grains should be considered.

In view of the fact that numerical analyses were performed to assess conditions within the apparatus for kaolin, it is assumed that size effects are minimal for the clay. However, the behaviour of sand at small scale is not necessarily that of a continuum and needs to be considered further because of its particulate nature.

The grouted nail diameter was 9 mm while \(D_{50}\) for the three sands tested varied from 0.10 to 1.40 mm. In the field, a typical drilled and grouted nail diameter is about 100 mm. This means that there is a linear scaling factor of about ten between model and prototype. Scaling up the grain sizes linearly would result in equivalent field grain sizes of 1 to 14 mm, implying that conditions of nailing in gravels have been modelled in the coarse sand tests!

The effects of grain size were considered when discussing under-registration of the radial stress measuring devices in Chapter 8, by estimating the number of grain contacts on a circumference of the nail. This varies from about 280 to 100 to 20 for the fine, medium and coarse grain sizes respectively.

It is assumed that a sufficient range of contacts is made for the fine and the medium grain sizes to give a uniform distribution of stress around the nail circumference. In the case of the coarse sand this is questionable. Similar conclusions were drawn from consideration of these effects in relation to the test results in Chapter 8, where the effect of the increased curvature resulting from the reduced nail diameter was assessed in relation to the degree of arching that took place during shearing.

In conclusion it is assumed the results from the fine and medium grained sands are not adversely affected by scale effects, those from the coarse sand tests should be viewed with caution.
10.2 IMPLICATIONS OF RESULTS WITH REGARD TO THE USE OF PULL-OUT TESTS

The results from the model nail tests are used in this section to assess the differences in nail behaviour under conditions induced by pull-out tests and stress relief. This has relevance to practice as pull-out tests are used routinely for designing and checking the performance of constructed nails.

10.2.1 General observations on the interface shear stress distribution along the nail

The distribution of interface shear stress along the length of the nail in the active zone has been focused on closely in this work. Given the imposed boundary conditions, the form of the distribution depends primarily on the soil and its state.

In the case of the normally-consolidated clay, there is a clear trend for the highest interface shear stresses to be mobilised over about the top 25% of the nail length in the active zone (i.e. in the close proximity of the facing), even after large displacements. Measurements indicate that shear stresses at the other positions along the nail were about half of those at the top. The results from the microfabric analyses confirm the length estimated above, as they indicate that shear-induced fabric is also only developed over this region (about 20% in this case).

The results from the numerical analysis presented in Chapter 4 show a more uniform distribution of shear stress almost to the end of the nail with a sharp increase at the very top (previously thought to be an error caused by a numerical instability in the vicinity of the gap between the nail and the sample). If such a distribution does occur in practice, the implication is that high shear stresses operate at the very end of the nail. This is because the magnitudes of shear stress presented in all previous plots, are average values based on the force measured by the upper device divided by the area of the shaft from that position to the top of the nail. If a reduced length of the nail were used to calculate the interface shear stresses, much higher values would be obtained. The distributions developed after a displacement of 2 mm obtained from the numerical analysis and those estimated from the experimental work are plotted together in Figure 10.1. Good agreement between the numerical analyses and the experimental results can be seen for most of the nail length. The disparity in magnitudes between the two test results arises from the initial stress state at the start of shearing; this is discussed in Section 10.3.3.

Similar distributions were observed with the dense sands tested, with the effect becoming more marked with increasing grain size. In the case of the sands neither numerical analyses nor a sand equivalent of microfabric analysis are available with which to make comparisons. The positions of the axial force measuring devices in the sand tests were such that the region in which the high shear stresses were measured is only known to be within the upper third of the
nail. However, judging by the negligible stresses measured at the next position down, the indication is that the high shear stresses are generated at the very top of the nail in the vicinity of its free end as with the clay tests.

A more uniform distribution is observed with the loose and medium dense sands, especially with the fine grain size. Higher shear stresses are mobilised during the shearing stage with the loose sands than the dense. As the grain size decreases, the measured distributions indicate that a greater length of the nail (towards the base) is being utilised, e.g. for the loose sands at least two-thirds after displacements of about 4 mm. The degree of shear stress that can be mobilised is always limited by the condition of no relative movement between soil and nail at the base of the sample. If this boundary condition assumption is correct, conditions of a uniform constant shear stress distribution cannot be achieved along the whole length of the nail in the active zone. The same conclusion was drawn from the numerical parametric studies performed at the start of the study, where uniform constant stresses along the nail boundary were only achieved when its diameter was small enough that it barely contributed to the strength or stability of the system.

10.2.2 Pull-out tests general

The implication of this finding could be important when considering the mode of behaviour induced in pull-out tests, which are often adopted to establish or check design strengths. The behaviour of a nail during a pull-out test, whether a constant load or constant displacement test, is dependent largely on its length. If the nail being tested is short, say less than 4 m, a condition of uniform displacement along the length of the nail being tested might be assumed (i.e. elastic stretching of the nail is negligible). As a consequence a uniform distribution of shear stress would be induced, the magnitude of which increases with displacement. Conditions would be similar to those shown in idealised model case 1, Figure 8.34a.

In the case of a long nail, a progressive mobilisation of stress along its length takes place as the displacements at the far end are much less than those at the face because of stretching of the nail. Consequently, different degrees of shear stress mobilisation take place, decreasing from the front to the far end. The difference in shear stress mobilisation along the length of short and long nails is shown in Figure 10.2, reproduced from Recommandations Cloutierre (1991). If the material has a brittle behaviour, post-peak drops in strength can occur at the front of the nail while shear stress is being mobilised further back, an example of this effect is shown in the strain-softening case of the idealised models, Figure 8.34d, although the boundary conditions are different from those of a pull-out test.

The maximum pull-out force at failure \( T_L \) is used to determine an average value of
interface skin friction, which is then generally applied uniformly over the relevant length of the nail being considered for analysis or design purposes.

10.2.3 Extrapolating results from model nail test to field scale

The interface shear stress distributions observed in the model nail tests indicate that, for the clay and dense sand, most of the shear stress is developed over about one quarter of the nail length at the front of the active zone. In these tests elastic extension of the model nail is assumed to be negligible and thus conditions are relevant to those of a short nail. If the magnitudes of shear stresses observed at the front in the model test are comparable to those obtained from pull-out test conditions (i.e. the average value), applying an average shear stress over the whole length could over-estimate mobilised capacity by four times. The effect might not be as severe in the case of the loose fine sand where a more uniform mobilisation of shear stress occurs along the nail length. It also has to be considered that the results might indicate that the active zone is very narrow. In practice if the nail were free to move, the resistant zone might start to develop beyond the point where interface shears stresses diminish.

Normally-consolidated kaolin

To investigate the possibility that interpreted results from pull-out tests might give over-estimates of the interface shear stresses that can be mobilised, recourse is made once again to the results from Martins’ (1983) test programme. Although his tests were performed to investigate pile shaft capacity, the mode of loading the model pile was directly analogous to a pull-out test. The pile was installed in a wished-in-place condition in such a way that it passed through the sample without restraint at the ends. Because of the method of installation, direct comparisons can be made with the results from the model nail study. The stress paths followed during the consolidation of Martins’ samples were varied but found to make little difference to values of average peak shear stress which were generally between 80 and 100 kPa and were mobilised after 1 to 2 mm of pile displacement. An average value of 90 kPa has been included in Figure 10.1 for comparison.

It can be inferred from Figure 10.1 that the average value of shear stress for the nail distribution, taking the lack of stress at the base (plane of no relative soil to nail movement) and the peak near the free end (facing) into account, would in this case be about the same as that from the pull-out condition. However, it does not necessarily follow that net average values are always similar, either for reduced scale test conditions or those in the field; modes of mobilisation are quite different.

A numerical analysis was performed to model the conditions of a pull-out test for the sample and nail dimensions adopted for the experimental nail study. The results from this analysis are presented in Figure 10.3 in the form of a series of sections through an axi-symmetric
slice showing contours of various quantities, namely $\sigma_a$, $\sigma_n$, $\sigma_b$ and $\tau$. The initial stresses imposed in the analysis were isotropic of 250 kPa. The contours represent the accumulated stress after 2 mm downward displacement of the nail boundary, which corresponds to the left hand border of each plot. Confidence in the results is gained by comparing the average value of pull-out shear stress calculated from Martins' tests, shown in Figure 10.1, with that from the analysis, there is very close agreement with values being within about 10 kPa.

A comparable series of plots are shown for the conditions of the nail test after the same displacement in Figure 10.4. In this case the nail and the lower platen are moved together to model axial stress relief, as was discussed in Chapter 4. The very different stress distributions resulting from the two loading conditions are marked.

The pull-out test and FE interface shear stress distributions, shown in Figure 10.1, are based on results from the same analyses as the contour plots shown in Figures 10.3 and 10.4 respectively. Despite the differences in the distributions, it is concluded from all the curves shown in Figure 10.1 that, for the normally-consolidated clay, in terms of average interface shear stresses, the values obtained from pull-out tests are representative of those that would be mobilised from the stress relief experienced under conditions of a nailed-soil excavation. Shear stresses over the central length of the nail are similar to those from the pull-out condition and the lack of mobilisation at the base is more than countered by the increased values near the face.

Dense sands

Results from the model tests are now assessed to see whether a similar conclusion can be applied to the case of the dense sands. The interface friction angles calculated indicate that only small soil displacements take place along most of the nail length. This fact was corroborated by measurements of soil movements in the vicinity of the core of sand around the top of the nail which are typically less than 20 % of those of the overall soil mass surrounding it. Consequently, shear stresses were only partially mobilised. In Chapter 8, this behaviour is largely attributed to the effects of circumferential arching, the degree of which increases with density and grain size (see Section 8.4). Shear stresses are only mobilised at the very front of the face where the movements from the stress relief break up the arched structure.

In comparing the behaviour at the interface resulting from the stress relief condition with that from pull-out, the likelihood of arching occurring at field scale and its influence have to be assessed.

At model nail scale, it was observed that the degree of circumferential arching reduced with decreasing grain size. Conversely, it might therefore be postulated that enlarging the nail diameter has the same effect. In which case arching would not be expected to influence field nail behaviour to the same degree that it does at small scale because the grain size of the sands
tested cover the range of fine to coarse sands typically encountered in the field. Another factor known to increase the degree of arching in the tests is the method of raining and densifying the sand around the nail in the apparatus, consolidation of the sand further increases the effect. In the field, arching around the nail is most likely to occur as a result of relaxation of the borehole sides during drilling and installation operations. It is not certain how the degrees of arching under field conditions compares to those induced in the experiments.

The influence of arching is still to be assessed even though the degree induced under field conditions cannot be readily quantified. In the model tests, arching reduced transfer of radial stress to the nail boundary, and by its nature it also tended to divert the deviatoric 'driving stress' into circumferential stress. Similar behaviour might be expected in the field.

Comparisons are initially made between conditions of stress relief and pull-out for the case where arching does take place. They are made in terms of four aspects of behaviour observed in the model stress relief tests:

(i) reduced normal stress acting on the nail;
(ii) reduced soil movement in the immediate vicinity of the nail;
(iii) localised development of $\tau_{nm}$ at the free end (i.e. facing) and;
(iv) no restrained dilation.

In the case of a pull-out test, item (i) would be the same as for that of stress relief. Because the nail is being pulled, relative soil to nail movements (item ii) would be induced along its length proportionally with applied displacements (assuming short nail conditions). As a result, $\tau_{nm}$ would have a uniform distribution along the length (item iii), with magnitude affected by (i). Its net value is likely to be greater than that of the stress relief case where stress mobilisation is very localised. Restrained dilation would be induced along the length, although its degree would also be reduced by (i).

The same aspects can be considered now for the case where there is no arching effect. The normal stress would be the same for both stress relief and pull-out conditions, it is probably a different value from that acting in situ because of drilling or installation disturbance. The soil movement and mobilisation of $\tau_{nm}$ can be considered together. In the loose sand tests a progressive development of $\tau_{nm}$ occurred with increasing displacements of the soil at the face. If it is assumed that the dense sand would behave in the same way if arching were not present, values of $\tau_{nm}$ generated would be expected to be the same for both stress relief and pull-out conditions over most of the length in the active zone. The higher stresses mobilised at the free end of the nail (in both loose and dense sands) probably occur for similar reasons as discussed for the clay results, therefore the same net $\tau_{nm}$ will be obtained for both conditions. Finally, a greater degree of restrained dilation is likely to be induced in a pull-out test because of the
uniform displacement imposed along its length.

The conclusion for the dense sand is that results from pull-out tests are likely to overestimate net interface shear stresses whether arching occurs or not, but to different degrees. In the case where arching does not occur, differences only result from the effects of restrained dilation (although these can be significant as reported by Recommandations Clouterre 1991, and Plumelle et al. 1987). Where arching does occur, pull-out test results are likely to give larger over-estimates because of the additional contributing factors, given by items (ii) and (iii) above. Confidence in results can be gained from performing interface shearbox tests under conditions of free dilation for normal stresses close to those estimated in situ.

**Loose and medium dense sands**

In the case of loose sands, where the test results indicate a uniform distribution of stress, pull-out tests are probably more compatible with conditions of loading in a nailed-soil excavation. The model nail test results indicate that values of interface friction angles away from the free end are slightly lower than those obtained from interface shearbox tests, particularly with the fine sand. Pull-out tests might therefore over-predict the shear stresses that can be developed under conditions of stress relief. The test results indicate that the behaviour of medium dense sands is generally similar to that of the loose sand.

**10.2.4 Progressive debonding**

The condition of 'progressive debonding' is known to occur during the loading of an anchor (Barley, 1988) and is included in this section on pull-out conditions as it results from direct loading of the reinforcing element (anchor, nail). It is a condition that tends to occur in anchors of length greater than about 4 m and results from the different stiffnesses of the soil, grout and tendon components of the system. If the stiffnesses were the same, debonding would not occur. Debonding can take place either between the grout-soil or the grout-tendon interface, depending on the bond strength at each interface and also on the relative stiffness of the materials. The effect of the process is shown schematically in Figure 10.5, along with a system of installation that is implemented to maximise loads from a single borehole; further details are given by Barley (1991 and 1995).

The question of whether this process occurs under conditions of a nailed-soil excavation needs to be addressed. In the active zone, the way in which the nail is loaded is very different from that of an anchor where load is applied directly, and hence progressive debonding from extension of the nail is thought unlikely to occur. However, during pull-out tests, conditions are quite similar to those of an anchor. Behaviour in the resistant zone is also quite similar, although there is also a small degree of stress relief taking place.

One of the main factors governing the degree of severity of progressive debonding is
the anchor length. Barley (1995) quantifies anchor performance in clays in terms of an efficiency factor which relates undrained strength and anchor dimensions to the ultimate pull-out resistance. He observes that full clay shear strength is mobilised for short lengths up to 3.5 m, the efficiency decreases progressively with lengths greater than about 4 m.

Nail lengths encountered in practice vary from about 5 to 10 m, but because the nail loading in the active and resistant zones is quite distinct, the length under consideration is less than that of the total. The distance from the front face to the locus of maximum force, or the plane at which no relative movement between soil and nail takes place is estimated to be about one third the excavation height, at the upper ground surface (Recommandations Clouterre, 1991). This implies that for a 12-m high excavation, the maximum length of nail within the active zone is about 4 m. As nail forces in the active zone have to be balanced by those in the resistant zone the nail lengths in this zone are usually similar.

A 12-m high excavation is towards the upper limit of typical depths in which soil nailing is used. It is therefore thought that progressive debonding is also unlikely to occur in the passive zone except with nails in excess of 10 m length. The effect should be considered when carrying out pull-out tests on nails greater than 4 m long.

10.2.5 Conclusions regarding pull-out tests

The purpose of this section has been to assess how representative is the behaviour induced from pull-out tests, where the nails are loaded directly, compared to that for the case of stress relief of the face where they are loaded indirectly.

The results obtained from a pull-out test are significantly influenced by the length of the nail being tested. The interpretation of results from short nails (less than 5 m long) is more straightforward than from long nails where the effects of nail extension, progressive mobilisation of interface shear stress and the possibility of progressive debonding should be taken into account. Moreover, testing long nails is not representative of conditions within a nailed-soil excavation because in practice, shears stresses on the nail act in opposite senses either side of the plane of no relative soil to nail movement. Operational lengths in the two regions are unlikely to exceed 5 m. The previous discussions and the following conclusions are based on a consideration of the more representative conditions of pull-out tests on short nails.

In normally-consolidated clays the average interface shear stress deduced from the results of a pull-out test is thought to be similar to that for the stress relief condition both in the active and resistant zones.

Slight over-predictions of the average mobilised shear stresses that can be developed for loose to medium dense sands are likely to be inferred from pull-out tests. The very high stresses observed at the free end of the nail in the clay tests were not witnessed with the sands.
Although full mobilisation of shear stress took place in this region, it does not counter the reduction in stress towards the plane of no relative soil to nail movement. The test results indicate that only about 75% of the friction angle was mobilised over the central region of the active zone.

Comparisons for dense sands are hindered by the effects of arching and restrained dilation. The differences between the degree of arching occurring in the field and that observed under the experimental conditions are not certain. Arching effects are thought to be reduced by the larger diameter of field nails but could still be induced from radial ground relaxation during borehole drilling and nail installation. Regardless of whether arching has been set up or not, the interpreted average shear stresses from a pull-out test are thought to over-estimate those operating under stress relief in the active zone. The difference increases with severity of arching around the nail. Results from pull-out tests should provide similar net values of interface shear stress to those generated in the passive zone in the stress relief condition.

10.3 PRACTICAL APPLICATIONS OF THE RESULTS

In this section some of the specific observations from the model tests are considered with respect to aspects of construction. The application of the findings to other construction activities, such as tunnelling and heave-reducing piles is also briefly discussed.

10.3.1 Details of the facing connection and stiffness

The differences in stress states resulting from loading the nail indirectly from stress relief of the surrounding soil, or from loading it directly by pull-out have been discussed in the previous sections. The mode of loading has a significant influence on the way in which the nail mobilises shear stress and its consequent distribution.

One of the assumptions made at the outset of the experimental study was that there is no connection between nail and facing. It is generally considered that the latter only provides support and prevents localised failure of the newly excavated soil between the nails. However, as mentioned in Section 10.1.1, in practice the nail is in most cases fixed in some way to the facing. This being so, the strength of the fixing and the stiffness of the wall itself could have a significant effect on the nail behaviour.

If the connection between the wall and the nails is strong (whether rigid or pinned) and the wall is stiff, the outward movement of the soil at the front of the face will tend to move the wall (without excessive local deformations), which in turn will tend to load the nails in a similar manner to a pull-out test. The consequence of this is that the load mechanism usually associated with a soil-nailing system, is distorted. The reinforcing inclusions will act partly as nails and partly as anchors in the active zone. In terms of interface shear stresses developed, those
induced from the stress relief will act in the opposite sense to those from the pull-out mode of loading (see Figures 8.34a and b of the idealised models for the sense of shearing). The net result would be to convert the nailed-soil wall to an anchored system with the front length of the nails, where interface shear stresses are acting against each other (the 'active zone'), thus representing the free length of the anchors, and the length in the 'resistant zone' the fixed length.

In some of the cases discussed in Chapter 3 there is evidence of rigid connections between the nails and the facing from the distribution of axial force measured in the nails. In such cases the maximum axial force in the nail is generated just behind the facing. Examples of distributions showing these effects can be seen in Figures 3.3b and 3.6d.

Another disadvantage of a strong rigid connection between nail and wall is that bending moments are likely to be induced at the nail head as deformations take place. This could have a detrimental effect on the nails if the soil in the immediate vicinity of the face is disturbed.

In conclusion, providing substantial walls and connections to the nails, might be detrimental to the primary mode of shear stress mobilisation and consequent restraint of soil movements usually associated with the philosophy of a nailed-soil excavation.

It should be noted that Tatsuoka (1993) concludes that nailed-walls become less deformable and more stable as wall stiffness increases. Nishida and Nishigata (1996) and Kodaka et al. (1995) also say that the restrainment effect can be improved, with fewer reinforcing elements when a rigid facing is adopted. In these cases the observed improvement in restraining deformations perhaps reflects that the conditions imposed by an anchored type of structure reduce overall movements. Care should be taken in such cases if the walls are designed as nailed structures. Particularly when assigning values of bond (shaft) resistance to lengths of the nail in the active zone where, by the mechanism imposed, interface shear stresses cancel each other. In which case provision should be made for increased lengths in the passive zone.

10.3.2 Nail installation method

All of the discussions so far have concerned drilled and grouted nails. Modifications to this method of construction and other types of installation techniques are now discussed in the context of the results.

Drilled and grouted nails are usually of constant diameter over their length and grout is in most cases introduced by gravity feed. Results from the tests on dense sands indicate that the effects of arching are detrimental to nail capacity; they inhibit the transfer of normal stress to the nail and the consequent mobilisation of shear stress and also the possible benefit of restrained dilation. Injection of grout under pressure could reduce these effects by offsetting the arching induced by relaxation of the borehole walls. The eventuality of loss of grout into the
surrounding soil needs to be taken into consideration. Jet-grouting constitutes such a system and would therefore offset the effects of arching and also provide an increased effective nail diameter.

Driven or fired nails (see Myles and Bridle, 1991) would also tend to reduce arching because of the outward displacement of the soil as they enter the ground. These types of nail have the disadvantage of a limited sectional area on which to mobilise interface shear stress. This deficiency is usually countered by installing them at closer centres than the drilled and grouted nails. They are also generally of shorter length because of the limitation of installing without buckling, maximum lengths are typically about 6 m.

Inclusions which can be expanded after installation, e.g. the wedge pile (see French, 1990) would also tend to reduce the effects of arching as well as increase normal stresses.

The results from the model tests in both clay and sands (Chapter 8 and 9) indicate that the largest shear stresses are mobilised in the close vicinity of the facing. Although probably not practicable, an enlarged section towards the front of the excavation might allow a greater degree of force to be mobilised in the nail and transferred back into the soil mass, thus providing a greater restraint to face deformations.

10.3.3 Minimising stress changes in the soil during the works

It was observed during the clay tests that the stress state at the start of shearing had a significant effect on the mobilisation of shear stresses along the nail length (see for example Figure 9.27). During the unloading of the sample an almost bi-linear response was observed. In the initial stages there was a rapid mobilisation of interface shear stress. The end of this phase was marked by a small temporary decrease in $\tau_{in}$, after which its development or rate of increase was much slower, i.e. the stiffness of the nailed-soil response decreased.

In order to maximise the benefit of the nail, it should therefore be installed in the ground as early as possible prior to the onset of significant stress changes from both the excavation in its vicinity (i.e. those taking place from the formation of the step at which it is to be installed) and those during further excavation steps.

During the usual construction procedures that are adopted, installing the nail prior to significant stress changes occurring could be achieved by reducing the time between excavating the step and installing the nail, and by reducing the depth of the step itself. Alternatively the excavation step could be carried out using a system of bays with short berms left in position between them while nails are installed. The idea is shown schematically in Figure 10.6a, which is a plan view of a rectangular excavation where some berms have been left in position during the excavation of one of the steps. Two nails are shown installed from some of the bays between the berms. Further nails could be installed after removal of the berms. It is possible
that difficulties with the shotcreting operations might be created using this approach, clearly practicalities such as this have to be considered.

Tatsuoka (1993) mentions the use of 'pre-propping' techniques where vertical minipiles, improved-soil columns or H-piles are installed at locations just behind the proposed position of the facing prior to excavation. These measures are intended to reduce deformations and are recommended for cases where only very small movements can be tolerated, e.g. when there are buildings in the close vicinity of the excavation. If levels of deformation are reduced, it is likely that stress changes will be as well, they would help offset the effects of stress changes occurring prior to the nails being installed.

Inhibiting stress changes prior to nail installation might also be achieved in a similar manner by installing the nails before excavating the soil in the vicinity of the facing using a two-phase excavation process. Steps could be excavated within an area smaller than that bounded by the proposed facing position and the nails therefore installed from further within the excavation. This idea is shown in the schematic plan of Figure 10.6b. The final removal of soil to the required facing position could then take place, with the front part of the nails broken off (clearly this length of the nail need not be reinforced). The time and economic benefit from such an approach would depend on the site conditions, the nail lengths required and the plant available.

It is also worth noting that the effects resulting from stress changes prior to nail installation might be more accentuated in the field because of the orientation of the bedding layers and the stress state in the ground. The tests performed in the experimental programme were on samples that had been first one-dimensionally and then isotropically consolidated. The nail was then installed in the samples perpendicular to the bedding planes. In the field it is far more likely that the nails are installed roughly parallel to the bedding. If the soil is over-consolidated, greater stress changes will occur in the horizontal sense. These factors provide further reasons for attempting to install the nails before significant stress changes occur.

10.3.4 Improving conditions in the vicinity of the facing

In all cases, the analysis of the test results indicated that interface shear stresses were initiated and developed the greatest magnitudes at the free end of the nail. In Figure 10.7, planes defining the 'active' regions of the samples are shown along with the lengths of the nail where most of the interface shear stresses are developed. Under field conditions this relates to the region in the vicinity of the facing. In the case of both the sands and the normally-consolidated clay, the instrumentation on the model nail showed that after some degree of mobilisation of interface shear stress, $\tau_{mn}$ there was a marked change in the rate of development of further stress. It decreased significantly and a greater degree of interface shear stress was
transferred further along the nail, although the magnitudes of stress were less than those that developed at the top initially.

The reason for this change in behaviour has not been positively identified. It seems likely that it is related to a form of soil yielding. In the case of the clay, this might also be associated with the onset of fabric features such as Riedel shears.

After large displacements failure in samples of clay was in the form of conical surfaces, as shown in Figure 10.7b. Although it was not possible to observe failure modes with the sand samples, potential zones in which conditions within the sample are inert are postulated in Figure 10.7a. They are shown to be bounded by conical surfaces, in the same manner as the clay.

In practice it would be beneficial if the stress level could be controlled so as to avoid reaching that at which the rate of mobilisation of $\tau_{\text{mat}}$ decreases. This might be achieved by initially installing much shorter 'secondary' nails around the proposed positions of the 'primary' nails, as shown in Figure 10.8. The primary nails could for example be drilled and grouted, while the secondary could be fired or driven nails. The operation of installing the fired nails is very quick and could be carried out immediately after excavation. The primary drilled and grouted nails could then be installed after shotcreting the face.

The secondary nails would have a twofold purpose, they would inhibit the degree of stress changes resulting from the excavation of the current step, prior to installing the primary nails, as was discussed in the previous section. Also, as the depth of the excavation increases and further interface shear stresses are mobilised on the nails, a proportion of the stress that would usually be carried by the primary nails alone would be transferred to these secondary elements.

The exact mechanisms that would operate with such a system of reinforcement have not been investigated and are outside the scope of the research described here. It is mentioned as an example of how possibly to optimise conditions at the face. The analysis of the arrangement shown should include the three-dimensional nature of the problem, which is not straightforward. Also, the position of the plane of $T_{\text{mat}}$ (or of no relative movement between soil and nail) shifts as the excavation proceeds, it might therefore be necessary to consider conditions at each stage of the works. The spacing of the nails is therefore an important aspect of design from both the lateral perspective as well as the vertical sense that is usually considered.

It is also worth noting that the presence of the facing, which is generally reinforced with mesh against the surface of the exposed soil, will alter the stress distribution from that acting under the experimental conditions. In the tests there was little restraint of inward radial movements of soil towards the free end of the nail because the platens were either smooth or flexible. The facing could help prevent the very localised distribution of interface shear stresses
in its vicinity, and if secondary nails were present the diverted stresses would be partly
transferred to them. It could also help prevent the formation of any tensile cracks radiating from
the nails, as was observed in the thin-section fabric of one of the clay tests, if features such as
this do occur in practice.

There are other reasons for minimising high stress concentrations in the immediate
vicinity of the facing. The soil in this region is in a disturbed state from the excavation
operation and therefore probably has a reduced stiffness. This effect was not modelled in the
experimental tests, it might result in a greater degree of stress being mobilised further down the
nail in practice, although with consequent increased displacements. The soil at this position is
most prone to the action of frost, which was discussed in Section 3.2.3. Also, it is probably
susceptible to softening from seepage as well as stress relief because of run-off at the back of
the facing (often a geogrid drainage membrane is placed against the soil prior to shotcreting to
prevent any build-up of pore pressures behind the face).

10.3.5 The effects of the rate of nail loading

In one of the tests in the experimental programme with clay samples, the shearing rate
was varied to assess the effects of the rate of loading. Generally the samples were sheared at
a sufficiently slow rate to avoid build up of excess pore water pressures. As the shearing rate
was increased less interface shear stress was developed at the top of the nail, stresses were
transferred further along its length. The consequent distribution was more uniform but the
average magnitude of stress was lower. The results from this particular test indicate that the rate
of shearing does affect the way in which stresses are mobilised and their magnitudes, however
detailed interpretation of the results from it should not be made for the reasons discussed in
Section 9.4.2.

As with most construction works, generally those for a nailed-soil excavation are carried
out at a safe and expeditious rate. The effects of rate in nailed-soil excavations only influence
the behaviour of clay soils. Any benefit that might be gained from reducing the speed of
excavation would be countered by losses in terms of time. Additionally it is good practice to
complete the excavation and seal the face with minimum delay to avoid localised softening at
the base of individual steps.

The limited information from one test is insufficient to draw firm conclusions on the
effects of rate of excavation. If rate effects are significant, their influence on the magnitude of
interface shear stresses developed along the length of the nail might be reflected in the results
from pull-out tests. These are unlikely to be performed at a sufficiently slow rate to allow full
dissipation of excess pore pressures, although the rate might be comparable to that of the
mobilisation of shear stress along the nail as the works progress.
10.3.6 Application of soil nailing in tunnelling

The use of soil nailing in tunnelling was mentioned in Chapter 1 with reference to the construction of the Thames tunnel (Skempton and Chrimes, 1994). In that instance the nails were installed to prevent face collapse while a damaged shield was replaced.

In modern applications nails are usually installed in the face and sometimes radially around the periphery of the tunnel (see Edmunds, 1997). Their purpose is still primarily to provide stability but their application for limiting ground movements is becoming more of a consideration, especially in heavily built-up urban environments. Nails used in tunnelling are usually temporary and because they have to be cut away as excavation of the tunnel proceeds, they are often made from fibre-glass (e.g. Barley and Graham, 1997).

The mode of operation of nails installed in a tunnel face is directly analogous to that of a nailed-soil excavation. Interface stresses are mobilised along the length of the nail as stress relief at the face takes place.

As the tunnel face is excavated, interface shear stresses are progressively mobilised. The plane of $T_{max}$, as defined by the condition of no relative movement between the soil and the nails, is unlikely to be far back from the face, say within half of a diameter. The plane has the form of a curved or domed surface because of the circular nature of the tunnel and the restraint provided by the surrounding soil mass. This plane moves back into the soil mass as the tunnel advances. The test results from both the clay and the sands indicate that interface shear stresses are mobilised predominantly in the immediate vicinity of where the stress relief takes place. Therefore the nail length in a tunnel face application probably does not need to be more than about a diameter long to be working to its full benefit. Clearly for practical considerations (e.g. to avoid disruptions to the progress of the works) and because its length is being continuously shortened, nails are usually installed as far forward of the face as possible. This observation might have application regarding the point at which to install the next set of nails, as for economic reasons the 'lap-length' between nails should be minimised.

The results from the experimental study confirm in a qualitative sense, the benefit that can be gained from the nails, in terms of both stability and limiting movements. Assessing the magnitude of restraint that can be achieved with inclusions in the face is the subject of current research (e.g. Jassionnesse et al., 1996). As yet there are no guidelines or methods of analysis available, although the technique is being used as there is little doubt concerning its benefit.

10.3.7 Consideration of behaviour of heave-reducing piles

Heave-reducing piles installed prior to removing soil within a proposed excavation also

\^{1}Nails installed in the face are sometimes termed spiles.
operate in the same manner as soil nails, except in the vertical sense (see O'Reilly and Al-Tabbaa, 1990, who consider the case of piles subjected to heave from vertical stress relief). The piles should be constructed to a depth sufficiently below the base of the excavation to provide an adequate length within the resistant zone.

The conclusions drawn concerning the connection and rigidity of the facing in the soil nailing case, discussed in Section 10.3.1, apply equally to the base slab in the heave-reducing pile case. If the slab is stiff and in contact with the excavated ground surface and the piles are strongly connected to it, the same mechanism described before will take place, i.e. interface shear stresses developed from the swelling of the soil will be countered by the opposing stresses generated from the piles being pulled by the slab. In practice, this occurrence is unlikely as a separating layer is usually provided beneath the slab to prevent cracking from uplift of the soil.

10.4 APPLICATION OF RESULTS TO ANALYSIS AND DESIGN

In the following sections, the results are discussed in terms of aspects of analysis or design. Most of the comments given, relate to qualitative observations. Quantitative recommendations are not given because of the limitations of the test apparatus, for example in terms of the boundary conditions and scale.

Conditions within a nailed-soil excavation are considered first in terms of the position of the plane of $T_{	ext{mm}}$, and the lengths of the nail either side of it. The behaviour at the nail-soil interface is then discussed and the feasibility of incorporating it into estimates of bond strength. Phenomena such as rigid-body inclusion effects, arching and restrained dilation can have a significant influence on the behaviour at the interface, these are covered in this same context. Finally, some comments are given on analysis using numerical methods (FEM) as this is becoming increasingly popular and is the only method available at the moment for providing information on the magnitude of deformations to expect at serviceability state.

10.4.1 Position of the plane of $T_{	ext{max}}$ and the length of the nails

The plane of $T_{	ext{max}}$ represents the position where there is no relative movement between the soil and the nail and where interface shear stresses are zero. This plane moves further back into the soil mass as the excavation increases in depth. Observations from models and full-scale tests indicate that for most of the upper part of the wall the plane is roughly vertical and located at a distance of about one third of the excavation height from the face (see Figure 1.5). The plane curves in towards the base of the excavation, roughly in the form of a log spiral.

The results from the experimental element tests on sand indicate that in all cases interface shear stresses were mobilised predominantly at the free end of the nail, as would be expected. Values of $\tau_{\text{mm}}$ decreased along the length of the nail towards its fixed base. In some
cases it diminished to zero at a point some distance from the fixed base. Beyond this point, there was either (i) little change in magnitude of $\tau_{int}$ or, (ii) $\tau_{int}$ started to develop again but in the opposite sense. This point, where $\tau_{int} = 0$, moved progressively along the nail with continued axial unloading, therefore simulating the movement of the $T_{max}$-plane observed in the field as excavation progresses.

Condition (i) occurs when the mobilisation of shear stresses along the nail is still progressing. This is generally the case for the loose and medium dense sands. Condition (ii) occurs when there is no further mobilisation of the interface shear stress front from the top of the sample, in which case the position of $\tau_{int} = 0$ remains roughly constant. Small degrees of $T_{int}$ were mobilised in the opposite sense from the extension and movement of the nail. This latter effect could not develop significantly because the nail was fixed to the base. The distance at which the point where $\tau_{int} = 0$ occurred depended on the degree of unloading that had taken place, and the relative density and grain size of the sample.

As mentioned above, in some cases of condition (ii), the distribution of $\tau_{int}$ diminished to zero at a distance well away from the fixed end of the nail and there did not seem to be any further mobilisation of $\tau_{int}$ beyond this point. This indicates that there was no relative movement at the point or beyond it. Thus, the 'plane' in the experimental apparatus, intended to be at the base of the sample, was for some cases, a 'zone'.

If the nail had not been connected to the base platen, it seems likely that a plane would have formed at the same point where the zone starts, beyond which a reversal of interface shear stress would have taken place. Thus, the lower part of the sample would have behaved as a resistant zone. This behaviour is evident in some of the results from the tests on dense samples (see Figures 8.40c, 8.46c and 8.52c). There is a reversal of interface shear stress with progressive unloading, but the magnitude of stresses further down the nail is not significant because of the fixed nail connection at the base.

The implication of the above observations is that the position of the plane of $T_{max}$ in the field, might be controlled to some extent by the relative density and grain size of the soil as well as the level of the excavation below the nail (i.e. the degree of unloading). The experimental test results indicate that the position of the $T_{max}$-plane is primarily controlled by relative density rather than grain size. It also seems unlikely that the grain size under full-scale conditions would have the same influence as in the experimental model arrangement, because its effect is lessened by the much larger relative nail diameter to grain size ratio.

The position of the plane of $T_{max}$ is important in design as it is usually taken to represent the plane where failure will ultimately occur and components of activating and resistant forces are summed either side of it to provide an overall factor of safety. Nail lengths beyond the
plane are used to calculate the nail forces that contribute to the overall stability.

In the case of a nailed-soil excavation, the facing is generally not intended to provide major structural support, as was discussed in Section 10.3.1. If it does, the mode of operation of the reinforced soil system changes and it essentially becomes an anchored wall. The shotcrete facing is generally considered to provide only local support to the newly exposed soil of the excavation. The nails are not rigidly connected to the facing and besides the facing is generally taken to be flexible. If this is the case, it is important that forces attributed to nail lengths extending beyond the potential failure surface do not exceed those that can be sustained in the active zone. Otherwise pull-out could occur within the active zone under conditions of failure without the nail length in the passive zone being fully utilised. Ideally the interface shear stresses generated either side of the plane of $T_{\text{max}}$ should balance each other.

In most cases non-uniform distributions of $\tau_{\text{int}}$ are observed from the results from the tests which simulate conditions in the active zone. The high values measured in the vicinity of the free end of the nail (representing the vicinity of the facing) are thought to be associated with the dramatic stress changes and rotations that take place there and the low stress levels. Although conditions within the passive zone were not investigated experimentally, the results from numerical analyses and field measurements indicate that the $\tau_{\text{int}}$ distributions would be uniform for most of the operational length. If this is so the nail length in the resistant zone would have to be longer than that in the active zone to achieve equal degrees of shear stress mobilisation. This idea is shown in the sketch in Figure 1.5 of the $\tau_{\text{int}}$ distribution, the areas under the curves either side of the $T_{\text{max}}$-plane being roughly equal.

10.4.2 Interface modelling

In Chapter 2 the approaches used for estimating the interface bond resistance (or shaft friction) for purposes of design were described. Only one of the methods used an approach where the angle of interface friction and the stresses acting around the nail were taken into consideration. In the other methods, either the results from pull-out tests are used or design values are chosen from charts based on previous experience covering different soil and nail types.

The results from the experimental tests illustrate the problems associated with modelling conditions at the interface realistically. The conditions of a nailed-soil excavation are such that during excavation interface shear stresses are progressively mobilised along the nail length in both the active and resistant zones simultaneously. Stresses are mobilised first at the front of the nail in the vicinity of the facing and then progressively work their way back along the nail length. At the same time these stresses are balanced by a corresponding development of stress in the resistant zone. The dividing line between these zones, the plane of $T_{\text{max}}$ moves back
along the nail as excavation proceeds.

In addition to the varying length of nail that has to be considered, the distribution of stress in the active zone is non-uniform, as described in the previous section. Modelling the development of interface shear stress in this zone is not straightforward. Values of $\tau_{\text{m}}$ in the vicinity of the facing are much higher than elsewhere along the nail length because of the degree of stress rotation that is taking place there and the very low stress levels. They diminish to zero at the $T_{\text{max}}$-plane. Within the resistant zone the distribution of stress would be expected to be much more uniform, more akin to the conditions along an anchor or a pile.

Taking account of the variation in $\tau_{\text{m}}$ is not straightforward because although the reasons for its distribution are understood in a qualitative way, there is insufficient information from the test results to quantify its magnitude and extent along the length of the nail. It seems most feasible that these conditions could be modelled using numerical methods with the FEM by applying the appropriate constitutive soil and interface models. At present no simplistic approach emerges, except using pull-out tests and interpreting the results with an understanding of the difference in loading conditions for pull-out and stress relief, as was discussed in Section 10.2.

Matters are further complicated by the influences of rigid-body inclusion effects, arching and restrained dilation. The test results indicate that these phenomena can have a significant effect on the mobilisation of interface shear stress along the nail.

Rigid-body inclusion effects can occur in both clays and sands, although as grain size increases the effects of arching might offset its influence. Interface radial (or normal) stresses acting on the nail increase from this effect, which could occur at any point along the nail length.

Arching has been seen to become more prevalent as grain size and density increase. It therefore mainly affects the granular soils and leads to a decrease in radial stress acting on the nail interface, which could occur in both the active and resistant zones. The results from the experimental tests probably over-emphasise the phenomenon because of scale effects associated with the model nail.

Restrained dilation has greatest significance in dense granular soils, although it may also manifest itself in clays and other fine grained materials. The effects of restrained dilation have not been witnessed during the shearing stages of the model nail tests. This is thought to be because of the arching that was taking place in the dense samples which tended to inhibit the effect. It seems likely that restrained dilation could considerably enhance bond strengths in the resistant zone, its effect has been observed from pull-out test results.

At present the only way in which to model and interrelate the various factors that influence conditions at the nail-soil interface is by numerical methods, e.g. the FEM, as
10.4.3 Modelling using numerical analysis

The finite element method is a powerful method of analysis often used for the solution of geotechnical problems. The methods in which it was used to model the experimental conditions for this study were described in Chapter 4. The results from such an analysis provide much insight into the conditions within a soil mass.

The results from the experimental test programme have provided useful information on the way in which interface shear stresses are mobilised along the nails of an excavation and some of the influencing factors that control the behaviour of the system. These conditions could be modelled using numerical analyses providing realistic constitutive models are adopted.

In terms of soil and interface behaviour, the study has shown the importance of modelling the following:

- (i) effects of principal stress rotation resulting from the relief of horizontal stresses;
- (ii) behaviour at low stress levels, as encountered under conditions of passive stress relief;
- (iii) rigid-body inclusion effects resulting from stresses applied across adjoining materials of different stiffness;
- (iv) the effects of arching resulting from localised granular assemblies within the soil mass, and;
- (v) restrained dilation.

Some of these factors are already incorporated into current constitutive models. For example, rigid-body inclusion effects arise because of the differences in stiffness between the soil and the nail. These effects were encountered when considering cell-action effects in Chapter 7, and are evident in the results from the numerical analyses in Chapter 4. Similarly, the influence of restrained dilation might to some degree be reproduced with some of the existing models.

In other such as cases (i) and (ii), experimental data is available and new more comprehensive constitutive models are currently being developed.

Effects of phenomenon such as arching are more related to the grain structure of a particulate material and cannot be simulated readily with constitutive models based on continuum mechanics. Research has been undertaken to model granular assemblies using discrete elements but these are still at an initial stage of development (e.g. Cundall and Strack, 1979).

A numerical analysis carried out with constitutive models that incorporate these factors could be validated and perhaps calibrated using the results from the test programme. The analysis could then be extrapolated to full-scale conditions. Ideally this would involve a fully
three-dimensional analysis. This is another current research topic being undertaken at Imperial College. At present analyses are usually carried out by reducing the boundary conditions to those of plane-strain, plane-stress or axi-symmetry.

Performing numerical analyses for the design of each proposed nailed-soil structure using complex constitutive soil and interface models and under three-dimensional conditions is probably unrealistic at present. However, once such an analysis is possible, parametric studies could be undertaken which might provide sufficient information to formulate more simplistic but more realistic design approaches than those currently available.

10.5 CONCLUSIONS ON THE APPLICATION OF THE TEST RESULTS

The results have illustrated the complexity of soil and interface behaviour in the vicinity of a soil nail being loaded by stress relief, although in some respects it has not been possible to quantify aspects of behaviour. The application of the test results have been considered under three headings, in relation to pull-out tests, construction practice and analysis and design. Some of the conclusions drawn in this chapter have been previously discussed by other researchers and practitioners. The results from this study provide further evidence of the significance of these matters and in some cases allow issues that have not been previously dealt with to be addressed.

10.5.1 Comparisons with reference to pull-out tests

Comparisons were made with pull-out tests as these are commonly used for obtaining and checking design parameters. The distribution of interface shear stress in the active zone depends on whether the nail is loaded directly by pull-out or indirectly by stress relief. It was concluded that for the normally-consolidated kaolin clay tested the average value of interface shear stress deduced from a pull-out test would be similar to that resulting from stress relief. Pull-out tests are thought to slightly over-predict for the case of loose to medium dense sands and would over-predict for dense sands especially if there is arching around the nail. The results from pull-out tests are thought to be representative of conditions in the passive or resistant zone.

Results from pull-out tests on elements longer than about 5 m might be misleading because of the effects of progressive mobilisation of interface shear stress along its length. Moreover, it is unlikely that nail lengths within the active or resistant zones will be greater than 5 m for typical excavation depths.

10.5.2 Practical considerations

The facing connection to the nails and its stiffness have been discussed. It has been observed that if the facing is stiff and the nails rigidly connected to it, the reinforced soil system will effectively become one of an anchored wall. Interface shear stresses generated along the nail in the active zone from stress relief of the facing will tend to be nullified by the opposing
shear stresses developed as a consequence of the wall's outward movement pulling the nail from within the soil mass.

The method of installation of the nail affects their subsequent behaviour. In the case of drilled and grouted nails, any relaxation of the walls of the borehole might lead to arching effects and also a softening of the ground in the immediate vicinity of the nail. Jet-grouted nails will minimise arching effects and are likely to cause increased radial (normal) stresses and provide enlarged nail sections.

Results from the clay testing indicate that greater restraint of deformations should be achieved if the nails are installed as early as possible. Methods of installing the nails prior to completing the full scope of each step of excavation are suggested.

Interface shear stress development is greatest in the vicinity of the newly exposed soil. It is thought that incorporating additional short 'secondary' inclusions between the main 'primary' nails towards the top of the excavation could result in a greater restraint of deformations.

Observations have been made on the similarity of modes of operation of nails in a nailed-soil excavation and those in tunnelling applications and heave-reducing piles.

10.5.3 Improving methods of analysis and design

The position of the plane of $T_{\text{max}}$ has been discussed in relation to the test results. It has been confirmed that the position of this plane progressively moves further back into the soil mass, up to a certain point, as excavation proceeds. The position of the plane has also been found to be dependent on the relative density of the soil (in the case of the sands) and on the grain size. It is thought that the influence of grain size might not be as great as is indicated by the model tests because of scale effects.

Realistic modelling of conditions in the soil and at the nail interface for conditions in the active zone of a nailed-soil excavation is not straightforward. Factors that should be taken into account include rotation of principal stresses, effects low stresses associated with passive stress relief, and phenomena such as rigid-body inclusion effects, arching and restrained dilation.

At present the most likely way of achieving a close simulation of overall conditions is with the sophisticated techniques of numerical analysis. It is suggested that analyses made using appropriate constitutive models for the soil and interface could be validated and perhaps calibrated using the results from the tests. The models could then be extrapolated to full-scale field conditions.
Among the innumerable mortifications which waylay human arrogance on every side may well be reckoned our ignorance of the most common objects and effects, a defect of which we become more sensible by every attempt to supply it. Vulgar and inactive minds confound familiarity with knowledge and conceive themselves informed of the whole nature of things when they are shown their form and told their use; but the speculatist, who is not content with superficial views, harasses himself with fruitless curiosity, and still, as he inquires more, perceives only that he knows less.

Samuel Johnson, 1758
CHAPTER 11

Summary, conclusions and proposals for future research

The research discussed in this thesis concerns a study of the interface resistance of soil nails for conditions of a nailed-soil excavation in the active zone. In particular, the development and distribution of shear stress along the nail length and the displacements required to mobilise them have been investigated. The method of research has mainly been experimental, element tests in sands and a clay were performed using an instrumented model nail.

The conclusions from this study are given under three headings: the development of experimental apparatus and the instrumented model nail, the test results and the application of the results to practice. Many lessons were learnt from the different aspects of the research carried out for this study, only the main conclusions from each are presented in this final chapter. Following from this, proposals for future research are described.

11.1 DEVELOPMENT OF APPARATUS AND INSTRUMENTATION

11.1.1 Conditions to be modelled

The initial stage was to identify the boundary conditions relevant to a nailed-soil excavation. Two zones exist, an active and a resistant (or passive) zone. Within each of these, interface shear stresses are developed in opposite senses, the plane between them marks the point along the nails where a maximum axial force acts and where there are no soil movements relative to the nail.

Most previous experimental research has focused on directly loading the nail, either longitudinally or transversely. However, under field conditions there is stress relief from the front of the excavation, and as the soil moves past the nails in the active zone, interface shear stresses are generated along the nail lengths. Thus the nails are loaded indirectly in a longitudinal sense.

It was decided that the active zone was the critical area to investigate. Loading conditions in this area are complex because of the indirect mode of loading and because the stress state of the soil mass in this region is progressively changing with time and as the excavation proceeds. In cases where the nails are not rigidly connected to the facing and the facing does not have significant rigidity, the stability of a nailed-soil excavation is governed by the behaviour within the active zone. It is important to note that the length of the nails in the resistant zone can be varied while the length in the active depends on the behaviour of both the soil and the nail-soil interface.
11.1.2 Testing apparatus

The approach taken in modelling field conditions experimentally was to perform element
tests under triaxial conditions with the model nail installed along the axis of the sample. Two
apparatus were used. The first was a modified Bishop and Wesley triaxial apparatus which was
used for the clay tests. The second was a stress chamber apparatus specifically designed for the
sand tests which were performed with dry sand.

Conditions within the apparatus for clay testing were analysed using the finite element
method. Parametric studies were carried out and the dimensions of the clay sample and model
nail optimised so that stress distributions were reasonably uniform and so that failure would
occur along the nail rather than across the sample.

Several ancillary devices and systems were designed so that the boundary conditions
imposed within each apparatus would simulate those in the field. In both cases the nail was
fixed at one end of the sample to model the condition of no relative nail to soil movement, thus
constituting the maximum limit of the active zone. At the other end it passed freely through the
platen to model conditions at the face. The samples were isotropically compressed under an all-
round stress, which was then maintained constant and the sample sheared by unloading in axial
extension under drained conditions.

Specific observations relating to the apparatus and testing methodology

- In general both apparatus worked very well and conditions similar to those existing
  around a nail in a nailed-soil excavation are thought to have been achieved. It was
  possible to observe the progressive development of shear stress along the nail as has
  been observed in the field and with numerical analyses.
- It would have been beneficial to have been able to test larger samples. This would
  have allowed a larger diameter model nail to have been used which would have
  facilitated the design of the nail instrumentation. In the case of the granular soils it
  would also have reduced the incidence of circumferential arching around the nail.
- In the case of the sand tests, the sand was rained around the nail which is thought to
  have also contributed to the observed effects of arching. A system for installing the nail
  in a wished-in-place condition would have avoided this, perhaps by using wet or damp
  sand and applying a small suction to temporarily maintain stability.
- The end boundary that was unloaded in the tests was rigid in the clay apparatus and
  flexible for the sand testing. The flexible boundary simulates field conditions more
  closely than the rigid one because a constant stress boundary is maintained allowing the
  soil to deform as it would in practice. Average displacements were determined in this
  case which were found to differ significantly from those measured locally in the vicinity
of the nail.
- Although the nail was fixed at the base to impose the condition of the plane of $T_{\text{max}}$ tests performed without such a connection would have provided information on the simultaneous development of interface shear stresses in the resistant zone during the early stages of shearing.

11.1.3 Instrumented model nail

In order to study in detail the behaviour along the nail interface, it was necessary to measure axial forces and radial stresses acting on it. Interface shear stress can be calculated if axial force is measured along the nail length. Interface radial stress measurements are also required if a comprehensive analytical interpretation of the conditions is to be made.

The 8-mm diameter of the model nail was chosen following the numerical parametric studies; consideration was also given to the practicalities of designing miniature instrumentation. Axial force and radial stress could be measured at three points along the 150 mm length of the final version of the model nail. This version was only used in the sand tests. At the time the clay tests were performed it was only possible to measure axial force accurately.

**Main conclusions from the development and use of the instrumented model nail**

- The axial force transducers worked very well. They were of a simple design which comprised components with a necked-section where strain gauges were bonded. Their output was essentially linear with an adequate sensitivity to measure force to less than a Newton. Some corrections were required for hysteresis during unloading and cross-effects from changing radial stress, these were straightforward to implement.

- The radial stress measuring devices also worked very well. Their design was more complex and each had the form of a segmental beam with strain gauges bonded to its underside. This beam rested within a precision machined slot within individual sections of the nail body. Their response to applied stress was non-linear and hysteretic but could be modelled with a fourth order regression and was always repeatable. Corrections were also required for cross-effects from applied axial force and reversals in loading.

- The magnitude of cell-action effects was assessed using elastic solutions and superposition. It was concluded that the radial stress measuring devices might under-register by up to about 30 % and 5 % for soil stiffnesses of 30 MPa and 3 MPa respectively. No corrections have been made for these effects.

- It was established during the sand tests that the radial stress measuring devices under-registered when used in coarse sand samples.

- Alternative means of measuring radial stress were attempted with limited success at
this small scale. Thin-walled radial shells with strain gauges bonded to their inner surfaces were found to be very temperature sensitive. Enclosed-reservoir type devices were very difficult to deair and hence had a soft response and were also very temperature sensitive.

- Each transducer was made as a separate component, these were then connected together using interference-fit push-joints. This system had the advantage that the components could be made individually. The disadvantage was that once made, the nail components could not be disassembled. Screw-thread connections were found to be unsuitable because of problems with wire twists and the threads becoming loose.

- The arrangement of the devices should be chosen to optimise the information that can be obtained overall, e.g. it should be possible to determine interface shear stress at each point where radial stress is measured. Axial force measuring devices positioned at the base and the top of the model nail would have provided useful additional information concerning the minimum and maximum forces in the nail and the interface shear stress distribution, without a significant amount of extra effort.

- An epoxy-resin was used to model the grout around the nail. This worked well, being stiff enough to transfer stresses from the soil to the nail and yet not so stiff as to reduce the sensitivity of the instrumentation.

11.2 RESULTS FROM THE TESTING PROGRAMME

Two test programmes were carried out, the first one on clay and the second on sand. In the main body of the thesis the results from sand tests are presented first, as a much greater scope of testing was performed and the final version of the instrumented model nail was used during these tests. Three sand gradings were tested at different values of relative density, to provide data for a range of conditions to cover the behaviour of fine, medium and coarse sand at loose, medium dense and dense states.

A corresponding reference test with no nail was made for each nailed sample test for comparison. Also, a series of shearbox tests were performed to provide additional information on the sand-sand and sand-interface behaviour of the fine and medium grained sands. The interface was made in the same way as the coverings for the radial stress measuring devices on the instrumented model nail.

The results from five tests on normally consolidated kaolin have been presented. Essentially the same stress paths were followed in each of the tests: one of the tests was performed without a nail; in others, shearing was continued to different displacements and specimens prepared for thin-section microfabric analysis; in the final test, shearing was carried
out at different rates.

A series of case examples with different applied loading conditions was formulated using idealised models to aid with the interpretation of the results. The models relate applied displacements to resulting interface shear stresses, allowing the behaviour along the length of the nail to be assessed. These cases and models, although simplistic, were found to be very useful in understanding the form of some of the observed stress distributions.

An individual summary of the main points and conclusions from each of the test programmes is given in the next two sections.

11.2.1 Summary and conclusions from the sand tests

The sand in all tests was dry. The fine and coarse sand were graded from the same deposit of Cretaceous Greensand, the medium grained sand was the new Ham River Sand (HRS).

Conclusions from the shearbox tests

- Peak values of $\phi_m'$, $\delta'$ and $\psi$ all decreased with increasing applied normal stress and were higher for the HRS compared to the fine sand. In general the displacements required to reach these peak values decreased with increasing density, implying that sample stiffnesses increased with relative density.
- Peak values of $\delta'$ were only marginally lower than those of $\phi_m'$ (i.e. within 3°) and were more ductile. The interface values of $\psi$ were also lower than those generated in the sand-sand tests and dropped off in a brittle manner. The displacements required to mobilise peak values of $\phi_m'$, $\delta'$ and $\psi$ were sometimes greater for the sand-sand tests although generally they were similar for both the sand-sand and the interface tests.
- The degree of contraction of the loose sands increased with increasing normal load and was greater for the fine sand than HRS.

Presentation of results

- The results from the triaxial tests have been expressed in two ways. In the first, changes in the measured stresses and displacements at the sample boundaries were considered, this provided information on the overall sample behaviour. With this type of presentation the results from the nailed sample tests were always compared to those of the reference tests. In the second, measurements from the instrumented model nail were analysed to provide information on internal conditions at the nail-soil interface.
- Measurements made at the sample boundaries were expressed as strains, stiffnesses, differences in strains and differences in work expended during shearing of the samples. These quantities were plotted against stress ratio, $\eta$ which was chosen as a common parameter between nail and reference tests. Work was adopted because it incorporates components of both volumetric and distortional stress and strain.
- Measurements made at the nail-soil interface were expressed as distributions of axial force and radial stress and calculated interface shear stress and interface friction angle, all along the length of the nail. The individual results were also expressed continuously by plotting them against stress ratio and as stress paths.

**Sample preparation and testing procedures**

- The sand was rained in the apparatus in layers to produce a loose state. Further densification was carried out, when required, by tapping the cell to produce a medium dense sample and then tamped for a dense sample. It is thought that the tapping might have contributed to a circumferential structural arrangement of particles being induced in the sample.

- The samples were isotropically compressed to 250 kPa, left overnight and then sheared in axial extension to simulate conditions of stress relief at the facing of a nailed-soil excavation.

**Phenomena influencing sample behaviour**

- The results throughout both the isotropic compression and shearing stages indicate that the sample and the interface behaviours were strongly influenced by phenomena whose existence and effect are dependent on the structural arrangement of the grains.

- During deposition and compression of the sands, annular rings of differing relative density are thought to have been formed around the nail. As stress level increases the particles in the rings tend to reorientate themselves creating a denser packing. Eventually the inter-locking between grains in the most dense rings is such that there is little freedom for further particle reorientation. At this stage circumferential arching starts to occur. Radial stresses applied at the boundaries of the sample are transmitted to these dense rings, where they are transferred into circumferential stresses. This effect is referred to as arching in the context of this study. When arching occurs around the nail there is a reduction of radial stress transferred to its interface and consequently only limited shear stresses are generated.

- The degree of arching was found to increase with relative density and grain size.

- Rigid-body inclusion effects are usually associated with materials that behave as continua with differing values of stiffness. e.g. if the sand and the nail are assumed to be elastic media with the nail of higher stiffness than the sand, elastic solutions indicate that if a radial stress of 250 kPa were applied at the boundary of the sample, the radial stress at the nail interface would be about 333 kPa. The magnitude of the stress increase is governed by the relative dimensions and stiffnesses of the sand sample and nail.

- Rigid-body inclusion effects were found to increase as relative density and grain size
decreased. Arching might be considered to be a case of negative rigid-body inclusion effects.

- Restrained dilation is another phenomenon that has been observed in field and laboratory studies on soil nailing carried out by others. It occurs predominantly in dense sands. As shearing along the interface takes place, the sand tries to dilate, but in instances where there are high stresses or a dense packing around the shearing zone this dilation is inhibited. As a result, very high normal stresses can be developed at the nail interface, consequently resulting in high interface shear stresses.

- Evidence of the effects of restrained dilation was not observed in the tests results from the shearing stage.

**Behaviour during isotropic compression**

- Results from the boundary measurements indicate that axial and radial strains increased with decreasing density, hence so also did volumetric strains. In many cases strains in the nailed samples were greater than those of the reference samples.

- In general the samples were stiffer axially than radially indicating that an anisotropic structure was set up during sample preparation.

- Results from the measurements with the instrumented model nail indicate that in all cases the nail went into compression. The distribution of force was uniform in most cases, indicating that interface shear stresses were mostly generated towards the top of the nail, the magnitudes of which were similar regardless of grain size or relative density.

- Observations of the radial stresses acting at the different positions along the nail indicate that the interface shearing that took place at the free end of the nail often resulted in a change in the grain structure in this vicinity. Loose and dense samples becoming more dense and loose respectively, as inferred from the different degrees of arching taking place around the top of the nail. In the case of the loose samples, measured incremental radial stresses towards the base of the nail often exceeded those applied at the sample boundary indicating rigid-body inclusion effects, these stresses reduced towards the top of the sample due to a degree of arching induced there. Conversely for the dense samples arching along the whole nail length is inferred from the much lower radial stresses that were measured compared to those applied externally, but these were greater towards the top of the sample where the arching structures had been disrupted by shearing. The medium dense samples showed no signs of rigid-body inclusion effects but the degree of arching was less than with the dense samples and distributions of radial stress were generally uniform. The effects of arching increased
significantly as the grain size increased.
- In conclusion, the radial stresses measured during isotropic compression were strongly
  influenced by grain size and relative density. Signs of rigid-body inclusion effects and
  arching were also evident.
- Values of interface friction angle calculated from the interface shear stresses and the
  radial stresses were unrealistically high for the coarse sands indicating that the radial
  stresses were under-registered by the measuring devices as well as reduced from the
  effects of arching.

**Behaviour during shearing**

- Measurements at the boundaries indicate that axial stiffnesses of the nailed samples
  were generally lower than those of the reference samples up to strains of about $-0.1\%$;
  by about $-1\%$ they were stiffer or more or less equal. In general, axial stiffness
  decreased with increasing grain size, there were no clear relationships with relative
  density.
- Volumetric strains during shearing differed markedly with grain size and state. The
  loose samples contracted, the degree of which increased as grain size reduced, while the
  dense samples were dilatant, more so with coarser grain sizes. The presence of the nail
  inhibited both the onset of volume change and its magnitude.
- Overall comparisons between nailed and reference samples were made by considering
  differences in work expended during shearing of corresponding tests. This provided a
  measure of the overall benefit of the nail. The results indicate that generally the nail
  became beneficial after axial strains of $-1.0\%$ to $-2.0\%$, the degree of benefit
  increased steadily with shearing.
- Another observation, relating to the external measurements, is that very high values
  of stress ratio, $\eta$, were reached in both the reference and nailed samples. According to
  critical state theory this implies that very high values of the angle of internal friction in
  extension, $\phi'_e$, were achieved, *i.e.* within a range of $55^\circ$ to $65^\circ$. Similarly high values
  of $\phi'_e$ have been recorded when testing at low stress levels during passive stress relief
  tests by other researchers.
- Measurements made at the model nail interface show that, because of the forces
  induced during the isotropic compression stage, the nail did not go into tension until
  after a certain degree of shearing. The results indicate that initially interface shear
  stresses were only mobilised at the top of the nail. A shear stress 'front' moved down
  the nail with progressive shearing, it did not reach the first of the instruments until axial
  strains of about $-1.0\%$. 
- In the case of the *loose* samples the mobilisation of shear stresses progressed steadily towards the base of the nail, with magnitudes over the instrumented length increasing in magnitude during this time. Radial stresses measured at the interface also increased steadily with shearing, values measured in the fine sand test exceeding those applied at the boundary because of rigid-body inclusion effects ($\sigma_{r, im} \sim 325$ kPa compared to $\sigma_{app} = 250$ kPa). The magnitudes of both interface shear stresses and radial stresses that developed decreased markedly with coarseness of grain size.

- Calculated values of interface friction angle over the instrumented length of the nail were generally lower than those of the peak values measured in the interface shearbox tests for the fine sand and HRS, indicating that displacements of the soil relative to the nail at this point were about one quarter of the average values measured at the top. Local measurements of axial displacement of the soil core immediately adjacent to the nail at the top of the sample confirm this interpretation.

- In the case of the *dense* samples, interface shear stresses were mobilised predominantly at the top of the nail. Measurements from the upper instruments indicate that a very small degree of interface shear stress development did take place at their level but that there was negligible mobilisation further down. There were only very small increases in radial stresses. As with the case of the loose samples, the magnitudes of both the interface shear stresses and radial stresses generated during this stage decreased markedly with grain size. E.g. in the case of the dense coarse sample, there was essentially no increase in radial stress and the magnitudes of interface shear stresses were only marginally greater than those generated during the isotropic compression stage.

- Interface friction angles calculated for the dense samples are generally much lower than the peak values obtained from the shearbox tests. However, in the case of the coarse sands unrealistically high values were obtained which are thought to result from under-registration of the radial stress measuring devices with this grain size.

- In conclusion, the observations from the test results indicate that there is a progressive mobilisation of interface shear stress along the nail as shearing takes place. This implies that the point of no relative movement between the soil and the nail, representing the plane of $T_{min}$ in field conditions, moves back with shearing which represents the stress relief from the excavation. The distance the 'front' moves along the nail is dependent on the sand grain size and relative density. In all cases these observations imply that during the early stages of shearing in the test apparatus, the lower part of the sample is essentially inert. In the case of the dense samples, the progress of the front along the nail was limited, it did not advance beyond the level of the instruments implying that
the lower part of the dense samples remained inert throughout the shearing stage.
- During shearing the degree of arching continued to be strongly dependent on grain size and relative density, increasing with both.
- In the case of the loose samples the benefit of the nail is clear from consideration of conditions both within the sample and those measured at the external boundaries.
- Measurements at the boundaries in the dense sample tests also indicate that the nail was beneficial, however this was only partly reflected by the results from the instrumented model nail. These suggest that all the nail benefit is derived from the mobilisation of interface shear stresses in the vicinity of the free end of the nail. Under conditions of shearing by axial extension, the constant radial stress acts as the ‘driving stress’ during shearing. It is thought that the arching induced around the nail caused this radial driving stress to be translated into a more stable circumferential stress. Thus the benefit observed from the nailed samples with dense sand might be partly derived by indirect means. The degree of arching set up in the field is uncertain, however, if arching does not occur, the effects of restrained dilation are likely to compensate.
- Clearly care is needed in extrapolating the results from the experimental tests to field conditions.

11.2.2 Summary and conclusions from the clay tests

The testing programme for the clay tests was limited to five tests on normally consolidated kaolin in which the same stress path was followed. One of the tests was a reference test, three others were with an instrumented nail, two of the samples were prepared for thin-section microfabric analysis after different degrees of shearing.

At the time the clay tests were performed, development of the instrumented nail was such that radial stress could not be reliably measured. The nail used enabled measurements of axial force at three points along its length to be made.

The results from an interface shearbox test performed by Martins (1983) have been used to supplement the results from the instrumented nail. The tests were performed with kaolin prepared from a slurry consolidated against an epoxy-resin interface.

Sample preparation and stages leading up to shearing

- The samples were prepared from a slurry at twice the liquid limit, which was carefully spooned into a large oedometer. This resulted in a curved, non-parallel bedding structure that hindered detailed microfabric analysis. It is recommended that the slurry be poured into the oedometer at a higher water content if practicable.
- The sample was consolidated one-dimensionally to $\sigma^* = 200$ kPa, and then set up in the triaxial apparatus and consolidated isotropically to 250 kPa.
- The nail was installed in a wished-in-place condition by unloading, drilling a hole down the axis, grouting in the nail, and reloading, maintaining a constant axial height and with the sample undrained throughout.

- Microfabric analysis and the results from the interface shearbox test indicate that minimal fabric disturbance occurred in these stages leading up to shearing where axial displacements were kept to within 0.2 mm.

**Behaviour during drained shearing up to displacements of 2 mm**

- During shearing the sample underwent a small initial dilation followed by steadily increasing contraction, the magnitude of which depended on the initial stress state and moisture content.

- Tensile axial forces in the nail continuously developed as interface shear stresses were mobilised at its free end during shearing, which simulated excavation. The magnitude of interface stresses at the top of the nail were almost twice those further down.

- A marked drop in interface shear stress took place at axial strains of $-0.05$ to $-0.2\%$ (i.e. 0.1 to 0.3 mm displacement). The magnitude of the drop was not large but is significant because the rate of further development of interface shear stress with shearing reduced. After axial strains of about $-1\%$ interface shear stresses at the top of the sample started to decrease and there was a greater transfer of stress further down the nail, although not of significant magnitude.

- The cause of the drop in interface shears stress is not certain. In terms of a Mohr-Coulomb relationship, it could be explained by a decrease in either the angle of interface friction or the radial stress acting on the nail, or both. However, none of these options seems likely.

- Microfabric analysis of soil from adjacent to the nail at the top of a sample sheared to about $-1\%$ axial strain reveals two fabric features that might be related to the drop in interface shear stress. There are two Riedel shears visible at the top of the sample that are just starting to develop from the interface, although they are at an early stage. There are also vertical shears visible, radiating out perpendicularly from the nail boundary. Their formation could be associated with a localised tensile failure but reasons for this are also far from clear.

- The magnitude of interface shear stress values at this stage of shearing at the top of the nail are in fact quite significant. This implies that there are either high radial stresses acting or high angles of interface friction angles. It is interesting to note that the shear stresses are two to three times greater than those predicted from the numerical analyses.
- If the data from the interface shearbox tests are interpreted using conservative estimates of displacement, very high values of radial stress are obtained, e.g. two to four times those applied at the boundary. Such magnitudes cannot be justified by rigid-body inclusion effects. An alternative reason might be formulated by consideration of the distribution of moisture contents within the sample after shearing. These imply that the clay in the vicinity of the upper part of the nail is in an over-consolidated state, similar observations have been made with soil around exhumed piles. This might then allow the occurrence of high radial stresses to be ascribed to the effects of restrained dilation. However, because of the lack of actual measurements of radial stress and a lack of well-documented evidence of this phenomenon, this suggestion must be considered as tentative.

- An alternative explanation can be provided by the consideration of unusually high values of the angle of internal shearing resistance observed in passive stress relief tests on clays as well as sands, observed by other researchers. It seems reasonable that high values of interface shearing resistance might be generated for the same reasons. At the present stage of understanding of the complex conditions acting around a soil nail subjected to passive stress relief, this is probably a more reasonable explanation.

**Behaviour during shearing to large displacements**

- The progressive mobilisation of interface shear stress diminished with continued shearing, and after axial strains of about −3% further changes became negligible. At the base there was a restriction to the development of interface shear stress because of the boundary condition of the fixed nail to the platen.

- Microfabric analysis of a thin-section from adjacent to the top of the nail in a sample sheared to almost −10% axial strain indicates that shear-induced fabric only developed within about the upper 30 mm of the sample. Careful inspection of the slide revealed that overall sample failure had occurred with the formation of an irregular conical failure surface with its apex towards the upper end of the nail. The surface appears to have initiated from the lower part of the zone of shear-induced fabric, with its subsequent path influenced by the bedding fabric.

- The influence of shearing rate was investigated to a limited extent with one sample after shearing to axial displacements of about 1 mm. The results from the instrumented model nail indicated that mobilisation of interface shear stresses at the top of the nail decreased with increasing rate. As a result more stress was mobilised further down the nail.
11.3 APPLICATIONS TO CURRENT PRACTICE

The application of the results to current practice are considered under the following three headings: pull-out tests, construction practice and analysis and design. Some of the conclusions drawn reinforce those previously discussed by other researchers and practitioners, but a number of new issues are also addressed.

**Pull-out tests**

- The results from the experimental tests and numerical analyses indicate that the distribution of interface shear stress in the active zone depends significantly on whether the nail is loaded directly by pull-out or indirectly by stress relief.
- Results from the normally-consolidated kaolin clay indicate that average values of interface shear stress deduced from a pull-out test would be similar to those from conditions of stress relief.
- In the case of sands, pull-out tests are thought to slightly over-predict in the case of loose to medium dense sands and to over-predict for the case of dense sands, especially if there is arching around the nail.
- The results from pull-out tests are thought to be representative of conditions in the passive or resistant zone.
- It is recommended that the length of nails used for pull-out tests be limited to 5 m to avoid misleading results from the effects of progressive mobilisation of interface shear stress along its length. It is also unlikely that nail lengths within either active or resistant zones would be greater than 5 m for typical excavation depths.

**Practical considerations**

- The facing connection and its stiffness have been considered in relation to the interface shear stress distribution observed in the model tests. It is concluded that if the wall were stiff and the nails strongly connected to it, the reinforced soil system would effectively become one of an anchored wall. The opposing senses of the interface shear stresses generated by stress relief and pull-out would tend to nullify each other.
- The method of nail installation affects their subsequent behaviour. In the case of drilled and grouted nails, any relaxation of the borehole walls might lead to arching effects and possibly a softening of the soil in the immediate vicinity of the nail. Jet-grouted nails would tend to minimise arching effects, might lead to increased radial stresses and provide larger nail sections.
- Results from the clay testing indicate that greater restraint of deformations should be achieved if nails are installed as early as possible. It might be possible to carry out excavation steps in stages to offset full stress relief until after the nails have been
installed.

- As interface shear stresses are greatest in the vicinity of the newly exposed soil, it is thought that incorporating short ‘secondary’ inclusions between the main ‘primary’ nails towards the top of the excavation could result in a greater restraint of deformations.
- It is concluded that the modes of operation of spiles in tunnelling applications and heave-reducing piles are very similar to those of nails in a nailed-soil excavation. The results indicate that the lap lengths of groups of spiles installed to reduce movements of the tunnel face can be quite short because most of the interface shear stress development takes place at the free end of the nail.

**Analysis and design**

- The very high interface shear stresses mobilised in the vicinity of the free end of the nail (even in the clay tests) which simulates conditions at the facing are reassuring given the uncertainty of the position of the plane of $T_{\text{max}}$.
- The fact that high interface shear stresses were mobilised in clay as well as sand is significant. There are only a limited number of cases where soil nailing has been used in clay. This is thought to be due to the uncertainty regarding the magnitude of shear stress that can be mobilised at the nail interface. The results indicate that mobilised values can be very high.
- The test results indicate that the position of the plane dividing the active and resistant zones (plane of $T_{\text{max}}$) progressively moves back into the soil mass away from the face, up to a certain point, as excavation and stress relief take place. Under test conditions, the position of the plane was found to be dependent on the relative density of the soil (in the case of sands) and the grain size. The influence of the grain size in the model tests might be exaggerated because of scale effects.
- It is concluded that realistic modelling of conditions in the soil and at the nail interface within the active zone of a nailed-soil excavation is not straightforward. Factors that ought to be considered include the rotation of principal stresses, the effects of low stress levels associated with conditions of passive stress relief and phenomena such as rigid-body inclusion effects, arching and restrained dilation.

**11.4 RECOMMENDATIONS FOR FURTHER RESEARCH**

Recommendations for further research are divided into experimental and numerical studies and field monitoring.

**Experimental studies**

- Under experimental conditions, the soil state, boundary pressures and displacements
can be accurately controlled. It would be beneficial to perform further tests with an instrumented model nail for measuring axial force and radial stress at several points along its length.

- In particular there is a need to accurately measure radial stress as close to the free end of the nail as possible to assess whether it is increases in radial stress or angles of interface friction that are causing the high values of interface shear stress observed in the programme of stress relief tests described in this thesis (particularly for the tests in clay). An axial force measuring unit at this position would be useful for confirming that the nail is free at this point.

- In order to quantify more accurately the differences between loading the nail directly and indirectly, pull-out tests should be performed in parallel with the stress relief tests, monitoring interface stresses in the same way.

- It is also important to understand how stresses are mobilised in the resistant zone and how the plane of $T_{max}$ migrates back into the soil mass and to compare them to the interface shear stress distribution induced from pull-out conditions. It is thought that this could be achieved with the apparatus described in this study, by leaving the nail free at the base rather than fixing it rigidly. Although ideally the model nail probably needs to be longer than that used for this study.

- Further microfabric studies are required to confirm the nature of shear-induced fabric features formed after various stages of shearing in clay. In order to make a quantitative analysis it is important the bedding induced during sample preparation from slurry be as uniform and parallel as possible.

- It would be beneficial as far as is practicable to use a model nail of diameter closer to full size scale to assess or eliminate scale effects associated with grain size. This would also facilitate the design of its instrumentation.

- More realistic conditions would be achieved if the nail could be installed in the sand after compression, rather than placing the sand around the nail from the start which is thought to contribute to grain structure effects.

- The effects of arching and restrained dilation need to be further investigated and quantified. Although increased instrumentation on the model nail would lead to this for the nailed-soil excavation case, it is desirable that these effects be studied using simpler boundary conditions initially, as there is little reliable information available on the mechanisms associated with these effects. An approach that might provide information on the stress conditions in the soil around the nail would be to use photoelastic analysis.
Numerical modelling

- The numerical analyses carried out for this study to model the soil and nail within the experimental apparatus were done so under the simplified conditions of axial symmetry. It would be useful to perform several further analyses in the same manner to assess the influence of changing certain of the boundary conditions.

- No numerical analyses were made to model conditions for the sand tests. As the results from the sand tests are far more comprehensive than those for the clay, a greater overall picture of conditions within the apparatus is available for comparison, particularly conditions at the nail interface modelled with the instrumented nail.

- Varying specific boundary conditions within the analysis of the experimental apparatus could provide further information on the influence of flexible sample platens (as was used for the sand tests), making the platens rough to model the facing and assessing its influence on conditions at the free end of the nail.

- A greater understanding of mechanisms such as arching and restrained dilation associated with particulate granular materials could be gained from numerical analyses of the experimental apparatus where artificial boundaries within the sample are imposed, e.g. by incorporating a ring of unbonded blocks to simulate arching, or imposing an annular displacement boundary in the close vicinity of the nail where outward radial movements are restricted, to induce the effects of restrained dilation.

- The flexibility and sophistication of numerical analysis using the finite element method makes it an ideal tool for studying complex boundary value problems such as the case of a nailed-soil excavation. At present conditions are broken down into those of plane-strain, plane-stress or axial symmetry. It is therefore not always possible to model all aspects of full-scale behaviour. However, advances are being made in the analysis of three-dimensional conditions.

- Three-dimensional analysis of full-scale conditions would allow some of the suggestions made in Chapter 10 to be investigated, e.g. installing the nail at an early stage prior to excavating using a system of bays, or from within a preliminary excavation, and incorporating shorter ‘secondary’ nails at the top of the excavation.

- Three-dimensional analysis would also allow the group effects and the interaction between nails to be investigated. This is important for optimising both the horizontal and the vertical spacing of the nails.

Full-scale field monitoring

- Although experimental and numerical studies provide much useful information on modes of behaviour under controlled conditions, neither can substitute for full-scale
monitoring.

- Before more quantitative use can be made of the results from this study, the mechanisms observed should be confirmed from full-scale monitoring.

- Full-scale monitoring with instrumentation for measuring axial forces and radial stresses in the nails and surface and subsurface ground displacements are invaluable for substantiating and calibrating the results from modelling. Such monitoring could verify and quantify factors such as the degree of high interface shear stresses observed in the vicinity of the free end of the nail, the effects of the connection to the facing, the occurrence and influence of phenomena such as rigid-body inclusion effects, arching and restrained dilation.

- Full-scale observations are required to give confidence in new approaches to analysis and design which take such factors into account.
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