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Aspects of the behaviour of reinforced earth walls

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REINFORCED EARTH WALLS

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YASMIN ASHAARI, B.E.(HONS)-CIVIL

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STATEMENT

I hereby certify that the work presented in this thesis has not been submitted for a degree to any other university or similar institution.

YASMIN ASHAARI
"From our hardest struggles come our fondest memories..."

S.A. Lowe (1990)
ABSTRACT

This thesis is concerned with the behaviour of reinforced earth model walls under different conditions. The following investigations have been carried out as part of this thesis:

(a) Experimental investigation of the effect of backfill thrust on the vertical normal stress within the reinforced zone. The reinforced zone was 300 mm high (H = 300); the length determined by the length of the reinforcement strips was 370 mm (L = 370); and the width was 800 mm (B = 800).

(b) Experimental investigation of the effect of the width of reinforcement strips on the vertical normal stress within the reinforced zone and the backfill.

(c) Experimental investigation of the influence of reinforcement on the shear strength of sand in direct shear using a shear box specially built for the purpose. The effect of submergence on the shear strength of reinforced sand was included as part of this investigation.

(d) Numerical analyses of reinforced earth model walls, using a finite element method and considering the reinforced zone as a composite material. The influence of the stiffness ratio between the reinforced zone and
backfill was included as a part of the investigation.

In the experimental investigations the overall effect of backfill thrust was an increase in the vertical normal stress along a horizontal plane close to the base of the reinforced zone. This behaviour is a reflection of the contribution of two components of lateral pressure, namely,

(a) The horizontal lateral stress component and,
(b) The tangential or frictional component acting vertically downwards along the vertical interface between the reinforced zone and the backfill.

If the effect of the frictional force is separated from the overall effect, the influence of the horizontal component of lateral stress can be identified. This is found to be a decrease in vertical normal stress at the back of the wall (near the interface) and an increase in pressure in the middle region. This behaviour is partly but only qualitatively similar to that predicted by the trapezoidal theory since the observed increase at the front of the wall (near the facing) is not maximum but smaller than that in the middle region. The wall obviously behaves as a flexible structure and there is clear evidence of an arching phenomenon as well (arching in the length direction - L direction).

A series of model reinforced earth walls were built where the width of the reinforcement strips was varied. The
length and number of strips were kept constant. Measurement of the vertical pressure close to the base, showed that the width of reinforcement affects the shape of the vertical pressure distribution along the reinforced zone. It also affects the vertical pressure across the wall and arching in the width direction (B direction) appears to influence the normal stress distribution.

Direct shear tests were performed using a purpose-built, 300mm square box. Several parameters were tested to investigate their effect on the shear strength of reinforced sand. The results show that the relationship between the strip width and the strength of the soil is non-linear. Moreover, there is an optimum width which results in a composite material (reinforced soil) with the highest strength. The influence of the type of reinforcement was tested and the efficiency of ribbed strip over smooth strip was quantified. The behaviour of reinforced sand under submerged condition was also studied. Under submergence, the efficiency of reinforcement is the same as in the dry condition.

Numerical analyses, using the finite element method, were performed to simulate the reinforced earth model wall. The parameters were related considering the reinforced zone to be a composite material. The effect of the 'stiffness' of the reinforced zone on the vertical normal stress,
horizontal normal stress, shear stress and deformations were studied. Comparisons were made between the experimental results and the results from finite element analyses. In some respects the results were similar but in other respects, the results differed significantly. The importance of boundary conditions and of the relative stiffness between reinforced zone and backfill was found to be significant.

Considering reinforced earth as an anisotropic material, the 'elastic' (or stiffness) parameters were evaluated on the basis of an available theory. These parameters were used in additional finite element analyses. Some of the analyses were performed considering full-scale model walls.

In the final chapter of this thesis the important results of all the investigations are summarised and the possible implications of these findings for design are discussed.
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NOTATION

All notations are defined when they first appear in the text. For convenience, the more frequently used notations and their meanings are listed below.

\[ A_r \] total cross-sectional area of reinforcement in a direct shear test

\[ A_{sr} \] cross-sectional area acted on by each reinforcement in a direct shear test

\[ A_s \] cross-sectional area of the shearing plane (central plane) in a direct shear test

\[ B \] width of a reinforced earth model wall

\[ c \] cohesion

\[ d \] height of fill above the base of a model wall

\[ E \] elastic modulus

\[ E_b \] elastic modulus of the backfill (unreinforced zone)

\[ E_r \] elastic modulus of the reinforced zone

\[ E_f \] elastic modulus for elements near the facing

\[ E_1 \] elastic modulus of 'soft' material

\[ E_2 \] elastic modulus of 'stiff' material

\[ e \] void ratio

\[ e_r \] efficiency factor

\[ e_m \] eccentricity

\[ f \] friction coefficient between soil and reinforcement

\[ H \] height of a wall

\[ h \] fill height above the level considered
K  general coefficient of earth pressure  
K_a  coefficient of active earth pressure  
K_c  a constant  
K_o  coefficient of 'at rest' earth pressure  
K_p  coefficient of passive earth pressure  
L  length of a reinforcement strip; also the length of the reinforced zone (for a wall of uniform length of strips)  
L_a  effective length of a reinforcement strip  
LVDT  Linear Voltage Displacement Transducer  
NA  Not Applicable  
P  resultant backfill thrust  
P_a  backfill thrust (lateral thrust)  
P_R  force in a reinforcement  
q  surcharge (from strip load)  
S  height of backfill above base of a model wall  
S_H  horizontal spacing of reinforcement strips in a reinforced earth wall  
S_v  vertical spacing of reinforcement strips in a reinforced earth wall  
T_{max}  maximum tensile force  
t_s  thickness of reinforcement strips  
W  weight of reinforced zone  
W  width of reinforcement strips  
x  x - coordinate  
X_1  horizontal shear displacement  
y  y - coordinate
\( \alpha \)  
angle

\( \phi \)  
internal friction angle of soil

\( \phi_m \)  
mobilized friction angle at the back of the wall

\( \phi_r \)  
internal friction angle of reinforced soil or internal friction angle of composite

\( \gamma \)  
unit weight of soil

\( \psi \)  
fraction angle between soil and reinforcement

\( \mu \)  
\( \tan \phi_m \) or friction coefficient at the back of the wall

\( \nu \)  
Poisson's ratio

\( \nu_1 \)  
Poisson's ratio of 'soft' material

\( \nu_2 \)  
Poisson's ratio of 'stiff' material

\( \theta \)  
angle

\( \sigma \)  
normal stress

\( \sigma_v \)  
vertical normal stress

\( \sigma_{yy} \)  
normal stress at the shearing plane in a direct shear test

\( \sigma_1 \)  
major principal stress

\( \sigma_3 \)  
minor principal stress

\( \tau \)  
shear stress

\( \tau_{yx} \)  
shear stress at the shearing plane in a direct shear test
CHAPTER 1
INTRODUCTION, AIMS AND SCOPE

1.1 Introduction

Earth reinforcement is an effective and reliable technique for increasing the strength and the stability of soils. It is by no means a new concept, as man has utilised its basic principles for centuries. For example, bricks made from clay reinforced with reed or straw have existed since biblical times. Nature demonstrates its basic principle through the stabilisation of slopes by tree roots.

The modern form of earth reinforcement was introduced by Henri Vidal, a French architect and engineer, in the 1960s. He came upon the idea on a beach where he observed that combining pine needle with sand resulted in a steeper stable mound of sand. His fascination led to research and experiments which he eventually turned into a practical concept, with the launching of a new engineering material known as 'Reinforced Earth'. In reinforced earth the soil is reinforced by tensile resistant metallic strips placed at regular intervals in the horizontal and vertical directions. The concept of reinforced earth is to build an homogenous block of composite structure. This composite action is derived from the frictional action between the
reinforcement and the granular soil. The inclusion of reinforcement appears to give an anisotropic cohesion to the soil mass.

Reinforced earth is now an established concept in civil engineering practice. Reinforced earth walls and other constructions have performed well. Applications include retaining walls, bridge abutments, dams and marine structures. Its popularity has been largely due to its economic viability, ease of construction and simplicity. Research studies on reinforced earth have also been conducted. These include analytical, numerical and experimental, studies along with field instrumentation of actual reinforced earth structures. However, much greater research effort is needed to understand the behaviour of reinforced earth at a fundamental level. One area of research and development concerns the interaction between the soil and reinforcement of various types. One of the outcomes of study in this area is that geosynthetic reinforcement and steel grids are now being used as an alternative to steel strips.

A major part of this thesis is concerned with the interaction between sand and reinforcement strips in direct shear tests, and in laboratory model walls. Direct shear tests in a specially designed shear box were conducted on sand with and without strip reinforcement. Several series
of reinforced earth model wall tests were performed, primarily to study the distribution of vertical stress on a horizontal surface within the fill. The purpose of this thesis is to provide further knowledge on the behaviour of reinforced earth wall and to discuss the implications of these findings on the design of reinforced earth walls. The behaviour of submerged reinforced soil, which has had little previous investigation, is also studied in this thesis. Another part of this thesis is concerned with numerical studies using a finite element method to investigate the stresses and deformations within a reinforced earth wall.

1.2 Aims of this Thesis

This thesis is concerned with the behaviour of reinforced earth under static loading conditions. The primary objective of the studies incorporated in this work was to contribute to a better understanding of the interaction between soil and reinforcement, and the implications for the design of reinforced earth walls. In order to achieve this aim, a comprehensive program of laboratory work was carried out. This consisted of two parts, one concerned with direct shear tests and the other with tests on model reinforced earth walls. Moreover, numerical studies were done using a finite element method. The main aims of the thesis are as follows:
1) Review of literature concerning the concept of reinforced earth and the mechanics of reinforced earth behaviour under static loading conditions.

2) Review of recent research concerning soil-reinforcement interaction, including theoretical models and experimental studies.

3) Review of current design methods, with emphasis on the vertical stress distribution.

4) Study of vertical stresses within the reinforced and unreinforced zone of a model reinforced earth wall.

5) Study of the influence of reinforcement on the strength of sand in direct shear with and without submergence.

6) To study stresses and deformations within a reinforced earth wall using a finite element method.

7) To consider the implication of the research findings on the design of reinforced earth walls.

A major part of the study was concerned with the experimental work relating to aims (4) and (5) above. The direct shear tests were conducted on a specially designed
shear box. A series of model wall tests were devised and appropriate instrumentation planned to achieve aim number (4) above.

1.3 Scope of the Thesis

This thesis is not concerned with the development of new analytical or numerical models. The behaviour of reinforced earth walls under dynamic loading is also outside the scope of this thesis. The numerical studies in this thesis are concerned only with linear elastic behaviour of reinforced earth walls. While some aspects of the thesis are concerned with the effect of submergence, the study is not concerned with the effect of seepage forces within a reinforced earth wall.

1.4 Structure of the Thesis

A comprehensive review of literature with emphasis on the mechanics of soil reinforcement is presented in Chapter 2. This includes a look at theoretical models and experimental studies.

Chapter 3 is concerned with recent research on the behaviour of reinforced earth walls. Basic design methods are also included where the importance of the vertical pressure distribution is emphasised.
Chapter 4 is concerned with the investigation of the effect of the backfill thrust on the vertical pressure distribution in reinforced earth model walls. The construction, instrumentation and test procedures of the reinforced earth model walls is described, and the results presented. Analysis of the results is also performed.

Chapter 5 is concerned with the investigation of the effect of the reinforcement strip dimensions (in particular the width) on the vertical pressure distribution in reinforced earth model walls. The construction, instrumentation and test procedures are described and the results presented. An analysis of the results is performed.

Chapter 6 is concerned with a series of investigations on reinforced soil using direct shear tests. This type of test is an investigation of reinforced soil at the unit cell level, with the added merit that the test set-up allows the specimen to have a configuration similar to that of a reinforced earth wall. The series of investigation are:

- to find the effect of reinforcement strip width on the behaviour of reinforced soil,
- to observe the behaviour of soil reinforced with wide strips,
- to find the effect of reinforcement strip type on reinforced soil and
- to observe the behaviour of submerged reinforced sand.

The results from these series of investigations are presented. Analyses of these results are performed. The findings from these investigations are presented.

Chapter 7 contains a literature review on a finite element models of reinforced earth walls. As well as looking at research that has been conducted in this area, this chapter will form a basis for understanding the work presented in Chapter 8.

Chapter 8 contains numerical studies using a finite element method to simulate the physical models described in Chapters 5 and 6. A composite approach is used and the development of this technique for each particular model is discussed. Results of the numerical simulations are presented. Analysis is performed on the results and comparison is made with results from the physical models.

Chapter 9 consists of two parts. In the first part, the findings of this thesis are summarised. In the second part the implications of the findings from the experimental work and numerical analyses, on the design of reinforced earth walls is discussed.
CHAPTER 2
MECHANISM, CONCEPTS AND APPLICATIONS

2.1 Scope of this Chapter

The main focus of this chapter is on the mechanism and concept of soil reinforcement. However before this subject is dealt with, an introduction to reinforced earth is presented where the components of reinforced earth are discussed and the usage of reinforced earth is described.

2.2 Reinforced Earth Wall

2.2.1 Introduction

The most common application of reinforced earth is in the construction of embankments and retaining structures. Reinforced earth walls are used as highway embankments, bridge abutments, earth dams and walls for coal storage slot, to name just a few of the possible applications. Since its introduction, reinforced earth has proved to be a state of the art construction material due to its very good performance in terms of stability, low cost, ease of construction and good settlement behaviour. An example of the flexibility of a reinforced earth structure is the reinforced earth wall built on poor foundation soil at Sète in France. The concrete facing of the wall deflected less
than 25mm even though the wall had a differential settlement of 380mm (Price, 1975).

2.2.2 Components of reinforced earth wall

The essential components of a reinforced earth wall are:

- reinforcement strips
- selected backfill and
- facing units.

The facing units do not serve a mechanical purpose i.e. they do not contribute in any significant way to shear strength or overall stiffness of the reinforced earth wall. However, facing is required to prevent erosion of the soil backfill as well as for aesthetic purposes. The amount of reinforcement required for the reinforced earth block (Figure 2.1) is very small. It ranges from 0.04% to 0.08% by volume (Boyd, 1980). The reinforcement strips are commonly either plain or ribbed metal strip. It should be noted that reinforced earth structure may be built using other non-metallic types of reinforcement such as plastic grid or made from geotextiles, in which case the structure is better known as a reinforced soil wall (Hanna and Rzzouki, 1984; John, 1986; Bathurst et al., 1988; Juran and Christopher, 1989). These polymeric reinforcements have
Figure 2.1 Reinforced earth wall
higher extensibility than metal strips and thus the deformation of a wall with extensible reinforcement is higher than that for a wall with metallic reinforcement. Walls reinforced with metallic reinforcement and walls reinforced with extensible material may be considered to be in two different categories since the extensibility affects the pull-out performance of reinforcement as well as deformation of the wall which in turn influences the state of stress in the soil. Since this thesis is concerned with walls reinforced with metal reinforcement, emphasis is on these types of walls.

In order to prevent corrosion of the reinforcement, and to provide good soil-reinforcement frictional interaction and sound structural behaviour of the reinforced earth wall, the backfill material should be granular and free-draining. The specifications for its physical, chemical and electrochemical characteristics are summarised as follows; (based on Boyd, 1980)

- It should be free from organic matter or other deleterious matter.
- Grading limits:
  i) nothing over 350mm
  ii) not more than 25% (by weight) particles larger than 150μm
  iii) not more than 15% particles smaller than 75μm
Where ribbed reinforcement is used, and provided the internal friction angle is not less than $25^\circ$, the particle size of the lower grading limit may be extended to not more than 15% smaller than 13.5μm. Investigation by Elias and Swanson (1983) has shown that a high percentage of fines can result in a significant reduction in strip pull-out capacity and that the reduction is more pronounced in saturated soils.

- Chemical limits:
  i) pH between 5 and 10
  ii) not more than 200ppm, chlorides
  iii) not more than 1000ppm, sulphates

- Electrochemical limits:
  resistivity under saturated conditions must be not less than 3000 ohm.cm

Jewell and Jones (1981 p.701) stated that:

The main reasons why fine grained and cohesive soils are generally considered unsuitable for reinforced earth construction are:

i) short term stability: the bond between cohesive soil and strip reinforcement is poor, time dependent and subject to reduction if positive pore water pressure
develops. It is unlikely that the current, largely empirical design methods for reinforced cohesionless soils may be satisfactorily applied to cohesive soil.

ii) corrosion: fine grained cohesive soils are significantly more aggressive than cohesionless soils. It is known that clay minerals, such as illite accelerate metal corrosion.

iii) post-construction movements: it is thought that long term deformations might occur when plastic soils are reinforced.

The importance of using the specified backfill cannot be overemphasised. This can be demonstrated by the case in South Africa (Blight and Dane, 1989) where corrosion of the reinforcement strips was accelerated by the fines in the fill. However, meeting these requirements can sometimes be very costly where the material is not available locally. Some research and development have been initiated concerning the use of lower quality backfill material such as clay (Ingold, 1981), mine waste (Jewell and Jones, 1980) and cinder (Sargunan and Kalyanasundaram, 1980). One of the aspects of such research is concerned with the type of reinforcement which is compatible with this material and which can offset the lower frictional resistance of the finer backfill. The use of geogrids as reinforcing material
is to be appreciated in this context. Jewell and Jones (1981) suggest that both the short and long term shear strengths of cohesive soil may be increased by grid reinforcement. The introduction of polymer reinforcements which are now categorised as geosynthetics, has been due to concern about corrosion.

2.3 Other Applications of Reinforced Earth

Although most applications of reinforced earth are in the form of walls, it can also be used for reinforcing foundations (Brown and Poulos, 1978; Akinmusuru and Akinbolade, 1981); reinforcing subgrade (Fragaszy and Lawton, 1984; Verma and Char, 1986; Hausmann, 1987). For these applications, generally the effect of reinforcement is to increase bearing capacity of the soil and to improve settlement characteristics.

Reinforced embankments on soft cohesive soils where geotextile were used, have proved to be feasible and efficient (Rowe, 1987). Reinforced soil wall has also been used for correction of landslides (Delmas et al., 1987).

2.4 Introduction to the Basic Mechanism of Reinforced Earth

The basic mechanism of reinforced soil involves
frictional interaction at the soil-reinforcement interface. This friction holds the soil and restrains its deformation and, therefore, increases the overall stiffness of the soil. The restrained deformation together with the induced tensile stress in the reinforcement can be interpreted as an enhanced confining pressure within the reinforced soil mass. The enhanced strength of the reinforced soil mass can be attributed to such an increase in confining pressure.

Consider a strip of reinforcement embedded in an unconfined mass of soil (Figure 2.2). When a stress, $\sigma_v$, normal to the plane of reinforcement is applied, there will be frictional force generated at the soil-reinforcement interface which consequently will induce tensile force in the strip. The difference in tensile forces, $T_1$ and $T_2$, generated at the ends of the element, $dl$, is given by:

$$dT = 2\sigma_v w f dL \quad (2.1)$$

where $w$ is the width of the reinforcement strip

$f$ is the coefficient of friction between reinforcement and soil

To ensure that there is no slippage between soil and reinforcement,

$$dT/2\sigma_v w dL < f \quad (2.2)$$
Figure 2.2 Consideration of a reinforcement strip under normal stress (after Vidal, 1969)
The tensile force generated is also limited by the tensile strength of the reinforcement. The effectiveness of reinforcement in increasing the strength of the soil mass depends on the amount of tensile force generated and the restrained deformation of the reinforced soil mass. In particular, the strength of reinforced soil depends on the stiffness and roughness of reinforcement and on the dilatancy of the soil.

Three concepts have been proposed to explain the strength enhancement of reinforced soil. Of these the simplest concepts are: (a) the induced anisotropic cohesion concept; and (b) the enhanced internal friction angle concept. The strength enhancement in both these concepts is associated with the concept of: (c) enhanced confining pressure, which can itself be used to explain the enhanced strength without invoking increase in either cohesion or the internal friction angle.

2.5 Enhanced Confining Pressure as a Result of Reinforcement

Ingold (1982) included enhanced confining pressure as one of the factors contributing to strength increase as a result of soil reinforcement; however it should be emphasised here that the enhanced confining pressure is the very basis of reinforced soil. This strength enhancement is
illustrated by the increased diameter of the Mohr circle of stress in Figure 2.3. The enhancement of confining pressure is a result of radial compressive stress which is induced by the frictional shear stress at the soil-reinforcement interface. The increase in confining pressure, $\Delta \sigma_3$, increases linearly with the applied confining pressure, $\sigma_3$, as shown experimentally by Yang (1972) (Figure 2.4). Yang performed his investigation using triaxial tests on sand where the heavy steel platen acts as a reinforcing disc. He concluded that, since the major principal stress, $\sigma_1$, increases linearly with applied confining pressure, the frictional shear stress, $\sigma_{1f}$, which induces the radial compressive stress will increase as well. In another series of tests using fibreglass netting as reinforcement, Yang (1972) found that the equivalent confining pressure increases linearly with the applied confining pressure; however, there is a critical value of the applied confining pressure where the reinforcement failed in tension. Beyond this limit the increase in confining pressure was constant.

It is known that the deformation or stiffness modulus of soil depends on the confining pressure (Duncan and Chang, 1970; Lambe and Whitman, 1979). This effect of enhanced confining pressure in reinforced soil can be seen in the increase of elastic modulus of reinforced soil mass compared to that of soil alone (Verma and Char, 1978; Gray and Ohashi, 1983; Hoshiya and Mandal, 1984). Results from
Figure 2.3 The concept of enhanced confining pressure represented by Mohr's circle of stress (A - Unreinforced soil; B - Reinforced soil)

Figure 2.4 Relationship between increase in confining pressure with applied confining pressure (after Yang, 1972)
Verma and Char (1978) on the initial tangent modulus of the reinforced and unreinforced samples are given in Table 2.1.

From triaxial compression tests on clay reinforced with plastic discs, Ingold and Miller (1983) concluded that the basic mechanism of the increase in strength and stiffness of reinforced soil is strain controlled. Comparatively inextensible reinforcement inhibits radial strain and consequently enhances radial stress or confining stress. They hypothesised that this radial stress increase is associated with the shear stress developed at the soil-reinforcement interface. The radial stress increase is highest near the interface and decreases in magnitude towards the mid-plane - the plane midway between two reinforcing discs. Beyond this plane the shear stress changes its sign and increases in magnitude, reaching a maximum at the next disc. Inherently, the rotation of the principal strain axes due to reinforcement follows the same pattern (i.e maximum at the interface and minimum at the mid-plane) and this was confirmed from results of radiographic tests. The radiographic results also show that the rotation of principal strain axes is zero at the centre of the sample and decreases with radius. This is in agreement with the finding of zero mobilized friction at the centre by Long et al. 1983.

These findings are vital to a study of the state of
Table 2.1 Initial Tangent Modulus for Unreinforced and Reinforced Sand (after Verma and Char, 1978)

<table>
<thead>
<tr>
<th>Sample, type of reinforcement</th>
<th>Applied confining pressure (kPa)</th>
<th>Initial Tangent Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>69.0</td>
<td>137.9</td>
</tr>
<tr>
<td>Unreinforced sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5% by volume mild steel</td>
<td>13.08</td>
<td>35.32</td>
</tr>
<tr>
<td>circular plates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5% by volume aluminium solid</td>
<td>43.60</td>
<td>54.50</td>
</tr>
<tr>
<td>circular plates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5% by volume aluminium solid</td>
<td>65.40</td>
<td>79.71</td>
</tr>
<tr>
<td>circular plates</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
stress and strain in a soil mass between the reinforcing layers in plane strain deformation which is generally relevant to real reinforced earth walls. For example, it would be of interest to find out the range and magnitude of the radial stress for a given pattern or configuration of reinforcement strips. Assuming that this would give the size of the influence zone for any particular reinforcement strip, a correlation could then be found between this size and the tributary area generally assumed in design. On the other hand, the findings referred to above were based on the results of triaxial compression tests; thus their application to a plain strain situation is not entirely satisfactory. In this connection it is important to state here that all experimental work reported in this thesis is for reinforced soil in plane strain deformation which is the type of deformation relevant to walls, embankments and footings which are straight and long in comparison to other dimensions.

2.6 A Theoretical Model Based on Shear Box Test

Many researchers (Jewell and Jones, 1981; Gray and Ohashi, 1983; Jewell and Wroth, 1987; Dyer, 1988) have conducted direct shear tests to study the mechanics of reinforced soil. In the direct shear test, the theoretical model for the strength enhancing mechanism of reinforcement was formulated by applying the principle of limit
Considering a reinforcement strip in a soil sample inclined at \((90^\circ - \theta)\) to the shear plane, the interaction of soil and reinforcement results in a force, \(P_R\), being induced in the reinforcement (Figure 2.5). This force has a normal component:

\[
P_R \cos \theta
\]  

(2.3)

and a tangential component:

\[
P_R \sin \theta
\]  

(2.4)

The tangential component reduces the shear stress which the soil must support from the level, \(\tau_{yx}\), (in unreinforced state) to:

\[
\tau = \tau_{yx} - \frac{P_R}{A_{sr}} \sin \theta
\]  

(2.5)

and the normal component increases the normal effective stress along the shear plane in the sand from \(\sigma_{yy}\) (in unreinforced state) to:

\[
\sigma = \sigma_{yy} + \frac{P_R}{A_{sr}} \cos \theta
\]  

(2.6)

where \(A_{sr}\) is the area of soil acted on by each reinforcement.
Figure 2.5  Definitions of stress and force for a reinforced soil soil under direct shear test (after Jewell and Jones, 1981)
Thus additional frictional shearing resistance can be mobilized because of the increase in normal stress. Taking into consideration the decrease in shear stress which the soil must support along the shear plane, the total effective increase in the shearing resistance due to the reinforcement is given by:

$$\tau_{\text{ext}} = \frac{P_r}{A_{er}} (\cos \theta \tan \phi + \sin \theta) \quad (2.7)$$

where $\phi$ is the internal friction angle.

2.6.1 The effect of reinforcement stiffness on reinforced soil

From the above theoretical model, it can be seen that the effectiveness of reinforced soil depends on the magnitude of the induced force in the reinforcement. Jewell and Wroth (1987) presented the same theoretical model wherein they investigated the influence of the axial stiffness of the reinforcement on reinforced soil, the axial stiffness being defined as the product of the Young's modulus and the cross-sectional area of the reinforcement. Their investigation was performed using direct shear tests. A typical stress ratio-shear displacement plot from the tests is shown in Figure 2.6. By comparing the curves for unreinforced sand with those for reinforced sand, it is found that, although reinforcement is only effective after the sample is strained a certain amount in order to
Figure 2.6 Typical stress ratio - shear displacement plot in direct shear stress (after Jewell and Wroth, 1987)
mobilize the shear stress between the soil and reinforcement, it does not take much shearing displacement for reinforcement to be effective. A sample with very stiff reinforcement reaches its maximum strength at a lower displacement than a sample with extensible reinforcement and the maximum is greater in magnitude for the stiff reinforcement than for the relatively flexible one. This is because a more extensible reinforcement would tend to deform with the soil, thus requiring greater deformation of the soil to generate the shear stress between the soil and reinforcement (Ingold and Miller, 1983). It can be seen that the inclusion of extensible reinforcement made the composite material more ductile. McGown et al. (1978) investigated the behaviour of sand reinforced with inextensible inclusion made from aluminium and extensible inclusion made from polymer fabric. The plane strain unit cell tests which utilised a triaxial compression machine showed that besides strengthening the sand, inextensible inclusion increases the deformation modulus (stiffness). It also causes the sample to be more brittle whereas extensible inclusion strengthens the sand and increases the ductility of the sample. High strength extensible inclusion even cancels the softening observed in dense sand behaviour (see Figure 2.7).

From radiographic observations (Jewell and Wroth, 1987), it was concluded that reinforcement increases the
zone of deformation and thus helps resist localized shear. This is reflected in higher vertical displacement in the reinforced sample compared to the unreinforced sample (Figure 2.8). The effect of reinforcement stiffness on the induced tensile force in the reinforcement was also studied using the radiographic technique where the force in the reinforcement was measured by observing the tensile extension of the reinforcement. By comparing the relationship between stiffness and additional shearing resistance with that between stiffness and the maximum induced force which occurs at the plane of shear, it was concluded that the additional shearing resistance is directly related to the induced force, (Figure 2.9).

2.6.2 Deformation of reinforced soil in direct shear test

Gray and Ohashi (1983) have presented a model of reinforced soil in direct shear which emphasises the deformation of the reinforcement during shearing (Figure 2.10). By invoking the principle of limit equilibrium as presented earlier by Jewell and Jones (1981), the additional shear strength due to reinforcement was written in the following form:
Figure 2.7 Influence of various types of reinforcement on behaviour of dense sand (after McGown et al., 1978)

Figure 2.8 Vertical displacement of samples under direct shear test (dilatancy behaviour) (after Jewell and Wroth, 1987)
Figure 2.9 Additional shearing resistance and maximum reinforcement force measured with different reinforcement stiffnesses (after Jewell and Wroth, 1987)
Figure 2.10 Model of deformation of reinforcement in direct shear test (after Gray and Ohashi, 1983)
\[ \tau_{\text{EXT}} = t_r (\cos \theta \tan \phi + \sin \theta) \]  

(2.8)

where \( t_r = \frac{A_r}{A_s} \sigma_r \)

\( \theta \) is the angle of shear distortion equal to \( \tan^{-1} \frac{x_1}{z} \)

\( z \) is the vertical depth of the deformation zone and \( x_1 \)

is the horizontal shear displacement

\( A_r \) is total cross-sectional area of reinforcement(s)

\( A_s \) is the cross-sectional area of the shear plane

\( \sigma_r \) is the tensile stress in the reinforcement at the shear

This equation shows that strength enhancement is related to the size of shear zone. Therefore a study of the size of the shear zone and the parameters that affect it, is important for a fuller understanding of the behaviour of reinforced soil. Shewbridge and Sitar (1989) have conducted investigations on the size of deformation with respect to the type and number of reinforcement strips. They used a specially built shear box in which the shear plane is vertical, (Figure 2.11). The base of the box is made of glass; by embedding some reinforcement in the lower part of the box, the deformation of the reinforcement during shearing can be observed. Appreciable volume change associated with the development of the shear zone was observed, and comparison with unreinforced sand showed that greater volume change occurs in reinforced sand. Typical force-displacement curves from the direct shear tests are
presented in Figure 2.12. It can be seen that reinforced sand has higher strength than unreinforced sand and that the stiffer reinforcement is the more effective. For a given reinforcement concentration, stiffer reinforcement increases the zone of deformation whereupon more soil mass is mobilized to resist the shear.

The effect of the skin friction (or the coefficient of friction between reinforcement and soil) on the shear zone was also studied. This was done by comparing the shear width of a sample reinforced with a certain reinforcement and the shear width of the same sample when this reinforcement had sand glued to its surfaces to increase the skin friction. Results from the tests showed that higher skin friction results in larger shear width. Even though Equation 2.8 suggests that there is a linear relationship between the ratio \( \frac{A_r}{A_s} \) and the strength increase, this was not confirmed by the tests of Shewbridge and Sitar (1989). On the other hand, Gray and Ohashi (1983) found it to apply in their investigation. This could be due to the fact that Gray and Ohashi did not consider skin friction coefficient as a parameter.

The stiffness of the reinforcement and the shear deformation of the sample affect the tensile force induced in the reinforcement; thus Juran et al (1988) included the effect of reinforcement stiffness and soil dilatancy in
Figure 2.11 The direct shear device
(after Shewbridge and Sitar, 1989)

Figure 2.12 Force displacement curves from Tests 2, 3, 4, 5 and 6; notations indicate number of reinforcements and type of material: S = steel; W1 = wood rod; P = parachute cords; B = bungy cords; N = no reinforcement
(after Shewbridge and Sitar, 1989)
their load transfer model of reinforced soil. From numerical simulation of direct shear tests of reinforced sand, they found that the tensile forces generated in the reinforcement are very small in a non-dilatant soil (Figure 2.13). At low shear displacements, the sand tends initially to contract, thus putting the reinforcement in compression rather than tension.

2.6.3 Influence of the inclination of reinforcement on reinforced soil

Theoretical models show that the strength enhancement is dependent on the inclination of the reinforcement with respect to the direction of the shear plane. It has been found that reinforcement is most effective when placed in the tensile strain direction of unreinforced soil. Dyer and Milligan (1984) performed photoelastic analysis of direct shear tests on unreinforced and reinforced sand and a part of their investigation was concerned with the inclination of reinforcement with respect to the direction of the shearing plane. They found that reinforcement is most effective when placed in such an orientation as to be subjected to pure tension. When reinforcement is placed in the compression zone, it is ineffective unless it is stiffer than the soil whereupon it will take up the load from the soil rather than strengthen it. When reinforcement is placed perpendicular to the shearing plane, it acts in
Figure 2.13 Influence of dilatancy property of soil on forces generated in reinforcements (after Juran et al., 1988)
bending as well as tension, rendering it less effective than when working in pure tension.

2.7 Types of Reinforcement and Related Shearing Mechanism at the Interface

There are many types of reinforcement available for reinforcing soil, made from either metal or polymer material. The reinforcement comes in the form of strips, grids or sheets. This section is only concerned with the former two types of reinforcement, most commonly used, the metal strips being either smooth or ribbed (Figure 2.14).

The behaviour of reinforced soil is governed by the interaction of soil with reinforcement; thus an understanding of shear resistance along the soil-reinforcement interface is important. The bonding mechanism at the interface can be classified as follows:

a) direct sliding;

b) bearing against the reinforcement;

c) interlocking of soil particle.

For smooth reinforcement strips, the mechanism of shear resistance mobilization is associated purely with sliding. For ribbed strips, the mechanism involves bearing action on the ribs in addition to sliding, providing extra shearing
Figure 2.14 Types of reinforcement strip
resistance to the system. For grid reinforcement, besides sliding and bearing action, resistance is also provided by the interlocking of soil particle at the openings (see Figure 2.15).

2.7.1 Shearing resistance

The magnitude of shearing resistance can be measured by direct shear test or pull-out test. The shearing resistance is defined as:

\[
\tau = \sigma \tan \psi \\
= \sigma f
\]

(2.9)

where \(\sigma\) is confining pressure
\(\psi\) is the mobilized friction angle at the soil-reinforcement interface

Here \(f\), the friction coefficient, is the apparent friction coefficient \(f'\) (Schlosser and Elias, 1978), obtained from pull-out test, or skin friction when obtained from the direct shear test (Potyondy, 1961). The value of skin friction for smooth metal is normally between 0.4 and 0.65 (Milligan and Palmeiro, 1987). Direct shear tests were performed by Hausmann (1976) using reinforcement material with sand which has internal friction angle of 39° \((\tan \phi = 0.81)\). He found that the skin friction of aluminium and
Figure 2.15 Interaction in bond for different types of reinforcement:
(a) direct sliding in smooth reinforcement
(b) direct sliding and bearing in ribbed reinforcement
(c) direct sliding, bearing and interlocking in grid reinforcement

(after Milligan and Palmeira, 1987)
steel was 0.38 and 0.34, respectively. These results and those of Potyondy (1961) who determined skin friction from direct shear test for various materials on both sand and clay, clearly show that the friction coefficient along a reinforcement-soil interface is less than the friction coefficient of the soil itself. On the other hand, the friction coefficient obtained from a pull-out test is generally higher than that of the soil. Results from Finlay et al. (1984) show that the friction coefficient from pull-out in dense test is, on average, three times higher than the friction coefficient of the soil obtained from direct shear test (see Table 2.2).

2.7.2 Direct shear test and pull-out test

The friction coefficient from direct shear tests may be obtained by using the conventional shear box. The set-up for sliding resistance is shown in Figure 2.16 where the reinforcement is placed on the shearing plane. The set-up for the pull-out tests is shown in Figure 2.17. In their tests, Finlay et al. (1984) used a box with the dimensions: 2m long, 0.42m wide and 0.23m deep containing the fill and 1.12m long reinforcing strip. The force required to pull out the strip can be measured and used to calculate the apparent friction coefficient. The friction coefficient obtained from pull-out tests is generally higher than that obtained from direct shear tests. This is because dilatancy
Table 2.2  Friction Coefficient Obtained from Two Methods of Test and Different Densities of Sand  (after Finlay et al., 1984)

<table>
<thead>
<tr>
<th>Material</th>
<th>Sand</th>
<th>Sand - Strip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sand Shear Box</td>
<td>Direct Shear Test</td>
</tr>
<tr>
<td>Confining Pressure (kPa)</td>
<td>50 to 200</td>
<td>50 to 200</td>
</tr>
<tr>
<td>Friction Coefficient</td>
<td>loose sand (17.2 kN/m³)</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>dense sand (20.1 kN/m³)</td>
<td>1.40</td>
</tr>
</tbody>
</table>
Figure 2.16  Shear test for direct sliding resistance:
(a) sheet or strip reinforcement
(b) grid reinforcement
(after Milligan and Palmeira, 1987)

Figure 2.17  Pull-out test, for all types of reinforcement
(after Milligan and Palmeira, 1987)
in direct shear tests is small or negligible (Juran et al., 1988). Dilatancy increases the shearing resistance of reinforced soil since it causes arching normal to the strip. Schlosser and Elias (1978) have investigated the effect of several factors on the pull-out resistance of reinforcing strip. Among the factors considered were: (a) the type of reinforcement; and (b) the density of the soil. Ribbed strips have higher pull-out resistance than smooth strips since the former create a larger zone of particle movement, thus increasing the dilatancy of the soil. The pull-out resistance of a strip in dense soil is higher than in loose soil due to the phenomenon of dilatancy.

An investigation concerning the friction coefficient obtained from direct shear test and pull-out tests was conducted by Finlay et al. (1984). Direct shear tests performed under applied normal pressures in the range 25 to 105 kPa showed the friction coefficient to be independent of normal pressure. In pull-out test the friction coefficient was high at low normal pressure and decreased with increasing normal pressure. However Ingold and Templeman (1979), who performed direct shear test under applied normal pressures in the range 0 to 115 kPa, found that the skin friction is high at low normal pressure and it decreases with increasing normal pressure to the friction coefficient of the soil. Finlay et al. (1984) also expressed their doubts about the conventional pull-out
test. They suggested that the rigid facing gives resistance when the strip is being pulled out, thus leading to increased normal pressure on the strip and hence a higher friction coefficient. With respect to this, they designed a pull-out set-up where the strip is attached to a facing plate and the whole system is pulled out simultaneously. Friction coefficient obtained from this modified pull-out test is lower than that obtained from the conventional pull-out test. Palmeira and Milligan (1989) performed investigations on the factors that affect the results of pull-out test of grids and found that the friction at the front wall of the pull-out box can result in severe overestimation of the friction coefficient.

2.7.3 Group effect

It is known that there is group effect in piles and, since the arrangement of reinforcement strips in a reinforced earth mass is not unlike a group of piles, there may be some group effect. Guilloux et al. (1979) made a comparison between the force per strip required to pull out a group of reinforcement strips with the force required to pull out a single strip. A coefficient $C_{eff}$ was defined where:

$$C_{eff} = \frac{T_g}{\sum T_i} \quad (2.10)$$
where $T_g$ is the total force required to pull out a group of strips and $\sum T_i$ is the summation of forces to pull out each strip individually.

They studied the forces required with respect to reinforcement strip width, $w$, and spacing, $j$. The results shown in Figure 2.18 indicate that the coefficient, $C_{\text{eff}}$, approaches 1 with $j = 6$ to $8w$. This corresponds to a spacing of 360mm to 480mm for the actual reinforcement strip width of 60mm in the field. However, there may be a scale effect associated with the laboratory tests so the results obtained may not be strictly applicable in the field.

2.8 Concept of Threshold Confining Stress in Reinforced Soil

Direct shear tests by Gray and Ohashi (1983) and triaxial tests by Gray and Al-Refeai (1986) show that there is a threshold confining stress which corresponds to a break in the failure envelopes. Above this threshold confining stress, failure envelopes for different types of reinforced sample are parallel to each other. Gray and Ohashi (1983) found that below the confining stress threshold value, the failure occurs by reinforcement tending to slip out of the soil. They found that this threshold confining stress is affected by the length, skin
Figure 2.18 Group effect in reinforcement strips - Ratio of total force for group pullout to sum of forces for individual pullout versus Ratio of strips spacing to strip width (after Guilloux et al., 1979)
friction and axial stiffness of the reinforcement used. Observations by Long et al. (1972) and Yang (1972), that beyond this threshold value, the reinforcement tends to fail in tension, might suggest that it is also a function of the tensile strength of the reinforcement. It should be clarified that Gray and Ohashi (1983) used reinforcement in the form of extensible fibre, and thus the failure mode of reinforced soil by breakage of the reinforcement was not obtained. Failure of the sample at confining pressure beyond the threshold value was due to failure of the sand.

The strength enhancement of reinforced soil can be described by its shear strength parameters from the failure envelopes. Under low confining pressure the governing parameter is the friction angle. In contrast, for reinforced soil under high confining pressure the governing parameter appears to be the induced cohesion. Thus the strength enhancement may be described by two concepts: the anisotropic cohesion concept and the enhanced friction angle concept.

2.8.1 The anisotropic cohesion concept

This concept was espoused by Schlosser and Long (1973) from results of triaxial tests by Long et al. (1972) on samples of reinforced and unreinforced sand. In these
tests, 100mm diameter cylindrical samples were reinforced with 100mm diameter discs made from aluminium foil. The tests showed that above a threshold value of confining pressure, there is a constant increase, $\Delta \sigma_v$, in applied vertical stress at failure, as illustrated in Figure 2.19. In general, the threshold value for this particular investigation was in the range of 50kPa to 100kPa. Inspection of the failed samples consistently showed that beyond this threshold value, the reinforcement had failed in tension, whereas below this value the reinforcement showed far less sign of tensile failure.

The anisotropic cohesion concept which corresponds to the failure of the reinforced soil by rupture of reinforcement states that, since both the failure envelopes for reinforced and unreinforced soil are parallel beyond the threshold value, thus exhibiting the same angle of shearing resistance, the increase in strength of reinforced soil is imparted by an apparent anisotropic cohesion, $c'$ (see Figure 2.19). The term 'anisotropic' is used because the reinforcement is two-dimensional i.e. the thickness is small relative to other dimensions. The apparent cohesion was derived by defining the failure equation of the reinforced sand in terms of the Rankine coefficient of passive earth pressure, $K_p$, as follows:
Figure 2.19 Failure envelopes for unreinforced and reinforced sand in triaxial test (after Schlosser and Long, 1973)
\[ \sigma_1 = K_p \sigma_3 + \Delta \sigma_1 \quad (2.11) \]

The failure condition for unreinforced soil with cohesion may be written as follows:

\[ \sigma_1 = \sigma_3 K_p + 2c \sqrt{K_p} \quad (2.12) \]

Comparing the two equations:

\[ \Delta \sigma_1 = 2c \sqrt{K_p} \]

\[ c' = \frac{\Delta \sigma_1}{2\sqrt{K_p}} \quad (2.13) \]

Schlosser and Long (1973) have made an analytical derivation relating the apparent cohesion for a reinforced cylindrical sample to the tensile strength, \( T \), and vertical spacing, \( h_r \), of the reinforcement where:

\[ c' = \frac{T \sqrt{K_p}}{2h_r} \quad (2.14) \]

This theoretical result agrees very well with the experimental result obtained by Long et al. (1972) (Figure 2.20).

2.8.2 The enhanced friction angle concept

This concept was introduced by Hausmann (1976) and it
implies that the increase in strength of reinforced soil is imparted by enhancement of internal friction angle from \( \phi \) to \( \phi_r \). As illustrated by Figure 2.19, at lower confining stress the shear envelope for reinforced soil is steeper than for unreinforced soil. Since bond failure or slippage generally occurs at low stress levels, this concept corresponds to reinforced soil in which the mechanism of failure corresponds to slippage i.e. loss of bond. This strength enhancement may also be illustrated with reference to Mohr circles and corresponding failure envelopes for a cohesionless material as shown in Figure 2.21. In formulating his model, Hausmann proposed that reinforcement can be viewed as introducing lateral prestress or confinement into the soil mass. In the model, the friction along the reinforcement is proportional to vertical stress. Thus the lateral prestress \( \sigma_r \) may be written as:

\[
\sigma_r = \sigma_{10} F
\]

where \( F \) is the friction factor.

From the Mohr circle the relationship can be written as:

\[
\sigma_3 + \sigma_r = K_a \sigma_1
\]

Substituting Equation 2.15 into Equation 2.16 gives:

\[
\frac{\sigma_3}{\sigma_1} + F = K_a
\]
Figure 2.20 Comparison of theoretical and experimental results (after Schlosser and Long, 1973)

Figure 2.21 Enhanced friction angle concept (after Hausmann, 1976)
Set $K_r = \frac{\sigma_3}{\sigma_1}$ where $r$ denotes reinforced

$$K_r = K_a - F \quad (2.18)$$

$$\frac{1 - \sin \phi_r}{1 + \sin \phi_r} = K_a - F$$

$$\sin \phi_r = \frac{K_a - F - 1}{F - K_a - 1}$$

$$\phi_r = \arcsin \left( \frac{1 + F - K_a}{1 - F + K_a} \right) \quad (2.19)$$

Hausmann (1976) used this relationship to construct the diagram (Figure 2.22) which relates the enhanced internal friction angle, $\phi_r$, to the unreinforced internal friction angle, $\phi$, and the friction factor, $F$.

2.9 Concluding Remarks on Reinforced Soil Mechanism

Reinforced soil is stronger than unreinforced soil due to an enhanced confining pressure which results from the frictional interaction between the soil and the reinforcement. Two models could be used to explain the strength enhancement: the apparent cohesion concept, which is applicable for reinforced soil under relatively high confining stress; and the enhanced friction angle concept for reinforced soil under relatively lower confining stress.
Figure 2.22 Enhanced internal friction $\phi_r$ as a function of internal friction angle of soil and friction factor, $F$ (after Hausmann, 1976)
The strength enhancement of reinforced soil is dependent on the induced tensile force in the reinforcement and this induced tensile force is a function of several factors. The axial stiffness of reinforcement and the dilatancy of the soil are two important factors. Stiff reinforcement increases the deformation modulus of the composite material and the inclusion of extensible reinforcement makes the composite more ductile.

The mechanism of shear strength mobilization is dependent on the type of reinforcement, and shearing resistance mobilized in laboratory tests is dependent on the type of tests and how the tests are performed.
CHAPTER 3
MECHANICS AND DESIGN OF A REINFORCED EARTH WALL

3.1 Introduction and Scope of this Chapter

The mechanics of a reinforced earth wall is defined here as the behaviour of such a wall during and after its construction and the placement of backfill. This comprises the state of stress and strain of the reinforced earth wall. Knowledge of the mechanics of reinforced earth wall behaviour may be gained by research using (a) model reinforced earth walls, (b) instrumentation of full-scale walls and (c) analytical or numerical analysis.

Design of a reinforced earth wall must be based on an understanding of its behaviour under self load. To the extent that the mechanics is not fully understood, certain assumptions have been made in conventional design practice. In order to understand important assumptions, these conventional design methods are presented briefly in this chapter after reviewing the recent literature on the mechanics of reinforced earth behaviour. Improvement in design practice, however, is outside the scope of this chapter.
3.2 Stresses in Reinforcement

The tensile stress in a reinforcement strip embedded in an earth wall is not uniform. It is zero at the strip's free end, increases to a maximum at a certain distance from the facing, and then decreases towards the facing (see Figure 3.1). The loci of the maximum tension in reinforcement strip, $T_m$, may be regarded as the potential failure surface. This surface divides the reinforced earth wall into two zones: an active zone where the shear stress mobilized at the soil-reinforcement interface acts towards the facing, and a resistant zone where the shear stress acts away from the facing (Figure 3.1). The shear stress developed along each face of a reinforcement strip of width $w$ is given by:

$$\tau = \frac{1}{2w} \frac{dT}{dL} \quad (3.1)$$

in which $dT$ is the change in tensile force over a length, $dL$, of the reinforcement strip. The equation shows that the shear stress is proportional to the derivative or slope of the curve of tensile force distribution along the length of the reinforcement. Both finite element and photoelastic analysis have shown that the shear stress at the bottom face of the reinforcement is higher than at the top face (Schlosser and Long, 1974). Therefore $\tau$ in Equation 3.1 may
Figure 3.1 Force distribution in a reinforcement strip
be regarded as an average value at any location.

The locus of maximum tension has been the subject of research in reinforced earth (Lee et al., 1973; Murray, 1983; Yong, 1983; Laba et al., 1984). It was found that the location and shape of this locus varies to some extent from one wall to another. This is illustrated by Figure 3.2. It can be concluded that the locus is a function of several factors. The method of construction and the safety factor of a wall are two important factors which determine the location and shape of the locus of maximum tension in reinforcement strips. The factor of safety of the wall itself depends on a number of factors such as wall height, shear strength of backfill and the spacing of reinforcement. Therefore, all these factors are important.

3.2.1 Coulomb's failure plane

By assuming that the Rankine active condition is applicable to a reinforced earth wall, the loci of maximum tension may be predicted as the associated Coulomb's failure plane through the toe of the wall. Comparisons have been made between the observed loci of maximum tension and the Coulomb's failure plane and good agreement between them was reported by Lee et al. (1973). However Bolton et al., (1978) reported that the loci of maximum tension formed an almost vertical surface, located at some distance from the
Figure 3.2 The loci of maximum tension for various walls under working stress conditions, based on measurements from full-scale walls (after Yong and McLarin, 1985)
facing. Observations by Yong (1983) of the reinforced earth wall at Mt. Messenger and by Yong and McLarin (1985) of reinforced earth wall at Ngauranga, show that the loci of maximum tension form a much steeper surface than the Coulomb failure planes for two different values of the angle of internal friction (Figure 3.3).

3.2.2 Magnitude of tensile stress

The stresses within the reinforcement strip may be measured by placing strain gauges along the length of the strip. Sargunan and Kalyanasundaram (1980) performed a test to see the effect of reinforcement concentration on the maximum tensile stress developed in the strips. This was done by varying the horizontal spacing of reinforcement strips in 390-mm high reinforced earth model wall and measuring the stresses at different points along the strips. It was found that the relationship between the maximum stress and the inverse of the spacing is almost linear, thus confirming the usual design assumption that the reinforcement strip carries the stress transferred from the soil within its tributary area.

Instrumentation of a full-scale reinforced earth wall by Yong (1983) provided evidence that the tensile force varies along the length of the strip and depends on its overall length. It was found that tension at a given
Figure 3.3  Comparison of observed loci of maximum tension for two walls with theoretical Coulomb failure planes for two different values of $\phi$ (after Yong and McLarin, 1985)
distance behind the facing was much higher for a shorter strip under the same fill height. Based on this, a conjecture was made that a shorter strip would need to develop more traction force for anchorage. Narain et al. (1981) reported that the slope of the curve of strip tension versus distance from facing tends to increase with decrease in length of strips, indicating greater mobilization of soil-reinforcement friction. The size of the active zone which is defined by the loci of maximum tension tends to decrease with decrease in strip length. In other words, for a smaller strip the maximum tension tends to occur closer to the facing than for a longer strip.

Their finding is in agreement with the research experience of Murray (1977) who found that, in a model wall designed with a high factor of safety against adherence failure (as in a wall with long strips), the tension near the facing was very small. On the other hand, for a model wall with factor of safety close to unity (as in a wall with short strips) the tension is generally highest near the facing.

Juran and Chen (1989) have formulated a theoretical model to predict the location and magnitude of maximum tension. In formulating the model, the potential failure surface is associated with a thin shear zone (Figure 3.4). The basis of this theoretical model is the theoretical load transfer model formulated by Juran et al. (1988). Simulations of reinforcement tensile failure using this
Figure 3.4 Schematic analogy between behaviour of reinforced soils in retaining wall and in direct shear box: (a) reinforced soil walls; (b) direct shear analogy

(after Juran and Chen, 1989)
theoretical model show that dilatancy of soil affects the size of the active zone as well as the critical height of a reinforced earth wall (Figure 3.5).

3.3 Lateral Pressure within Reinforced Earth Walls

Many full-scale reinforced earth walls and reinforced earth model walls have been instrumented to measure the lateral pressure within the wall (Lee et al., 1973; Al-Hussaini and Johnson, 1978; Narain et al., 1981; Yong, 1983; Murray and Hollinghurst, 1986). Most of these were to measure the lateral pressure near the facing. The lateral pressure in a reinforced earth wall may be found by two methods: (a) direct method using pressure cells, and (b) indirect method where the strip tension, measured near the facing, is divided by its tributary area. Measurement by Yong (1983) shows that the lateral pressure at the facing is close to the Rankine active earth pressure at the top of the wall while near the base it is much less than the Rankine active earth pressure. A similar conclusion was reached by Lee et al. (1973) from their own experiments. Ingold (1983) analysed the value of the earth pressure coefficient with respect to fill depths for walls of various heights (Figure 3.6). This figure illustrates that, towards the top, the pressure is larger than the active pressure. This may be attributed to stresses arising from compaction effort of the fill. John (1979) proposed that
Figure 3.5 Comparison between predicted and experimental: (a) failure heights; (b) failure surfaces. Prediction based on numerical analysis with different dilatancy coefficient, $R_D$. $\alpha_f$ denotes the inclination of the potential failure surface with respect to the vertical at the toe of the wall. (after Juran and Chen, 1989)
Figure 3.6 Observed variation of earth pressure coefficient with depth, of full-scale reinforced earth walls (after Ingold, 1983)
such distribution is due to the method of construction where a reinforced earth wall is built layer by layer, thus inhibiting the upper part of the wall from yielding. Theoretical work by Terzaghi (1941) shows that such distribution of lateral earth pressure is associated with overturning of the wall from the top (upper edge of the wall), which is in agreement with the type of failure mode of reinforced earth model walls observed by John (1979). Measurements of full-scale reinforced earth by Al-Husaini and Perry (1978) and Finlay and Sutherland (1977) show that the lateral pressure distribution that pertains in these walls is quite similar to that proposed by Terzaghi (1941), as illustrated in Figure 3.7. For completeness it should be added that Ingold (1983) argued that the theoretical failure mode of rotation about the top of the wall contradicts the observed rotation about the toe in some reinforced earth walls. The observed deformation, however, may be due to the externally applied load due to the compaction process.

An interesting observation of the lateral pressure during construction was made by Murray and Hollinghurst (1986). Instrumentation was installed to measure lateral earth pressure within the reinforced zone, as well as the unreinforced zone. The results are shown in Figure 3.8. The pressure quickly reached a maximum value and then remained constant within the unreinforced zone regardless of the
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Figure 3.7 Lateral earth pressure coefficient within depth of wall (after John, 1979)
Figure 3.8  Relations between horizontal (lateral) pressure and depth of fill above pressure cell (after Murray and Hollinghurst, 1986)
3-16

depth of fill above a pressure cell. This indicates that the fill is behaving as an over-consolidated material. This is in contrast with the lateral pressure within the reinforced zone (Figure 3.8 (c)), where the lateral pressure increases almost uniformly with fill height above a pressure cell. This suggests that the presence of reinforcement results in a redistribution of the lateral pressure such that the effect of construction is less localized.

3.4 Vertical Pressure Distribution

Due to the backfill thrust acting within the reinforced zone, the vertical pressure distribution can be expected to be non-uniform. The vertical pressure distributions that may be used in design of a reinforced earth wall are: (a) the uniform distribution; (b) trapezoidal distribution; and (c) Meyerhof's distribution. The first disregards the effect of the backfill thrust while the latter two take into account the effect of the backfill thrust.

3.4.1 Uniform vertical pressure distribution

This is the simplest assumption and can be justified only if backfill thrust on a reinforced earth wall is neglected. The vertical stress at any horizontal plane is
assumed equal to the overburden pressure and is given by:

$$\sigma_v = \gamma h$$  \hspace{1cm} (3.2)

where $\gamma$ is the bulk unit weight of the soil mass

$h$ is the fill height above the level considered

The maximum vertical stress for a wall with height, $H$, will occur at the base where $\sigma_v = \gamma H$.

3.4.2 Trapezoidal distribution

In order to incorporate the effect of the backfill thrust on the vertical pressure within the reinforced earth wall, the active force, $P_a$, is assumed to act at a height of $H/3$ above the base of the wall and the resulting vertical pressure distribution is trapezoidal (Figure 3.9). For simplicity the back of the wall is assumed smooth. The equation for the magnitude of the vertical stress at the base of the wall may be derived following the procedure:

Equating forces and reactions in the vertical direction:

$$W = HL\gamma = L/2(\sigma_a + \sigma_b) \hspace{1cm} \text{thus} \hspace{1cm} \sigma_b = 2\gamma H - \sigma_a$$  \hspace{1cm} (3.3)

where $W$ is the weight of the wall.

Taking the moments of forces and reactions about the centre of the base and noting that $P_a = 1/2K_\gamma H^2$: 
Figure 3.9  Trapezoidal distribution
1/2(σₐ - σ₉) × L × L/6 - 1/2 KₐγH² × H/3 = 0 \hspace{1cm} (3.4)

\frac{L^2}{12} (σₐ - 2γH + σ₉) - \frac{1}{6} KₐγH³ = 0

Simplifying:

σₐ = \frac{KₐγH³}{L^2} + γH

= γH (1 + \frac{KₐH²}{L^2}) \hspace{1cm} (3.5)

and

σ₉ = γH (1 - \frac{KₐH²}{L^2}) \hspace{1cm} (3.6)

3.4.3 Meyerhof's distribution

Another vertical pressure distribution that has been proposed is the Meyerhof Distribution. For this distribution the vertical pressure over a part of any horizontal plane is assumed to be zero (Figure 3.10). The stress distribution is uniform over a width, L-2eₘ, and zero over the remaining width, 2eₘ, where eₘ is the eccentricity of the resultant of forces, Pₐ and W. The magnitude of eₘ can be calculated by taking moments about
Figure 3.10 Meyerhof's distribution
point N as shown in Figure 3.10 (b):

\[ W \times e_m - P_a \times H/3 = 0 \]

\[ \gamma H L e_m - 1/2 K_a H^2 \times H/3 = 0 \]

\[ e_m = \frac{K_a H^2}{6L} \] (3.7)

The magnitude of the vertical pressure at the base of the wall is:

\[ \sigma_v = \frac{\gamma H L}{L - 2e_m} \]

\[ = \frac{H \gamma}{1 - K_a H^2/3L^2} \] (3.8)

3.4.4 Research investigation concerning the vertical pressure distribution

Only a few research investigations have been made concerning the actual vertical pressure distribution within a reinforced earth wall. Bolton et al. (1978) and Murray (1983) investigated the vertical pressure distribution using a reinforced earth model wall while Murray and Hollinghurst (1986) instrumented full-scale reinforced earth wall.
Bolton et al. (1978) measured vertical pressure at the base of model walls by using 10mm diameter pressure cells. The model walls were subjected to various centrifugal accelerations in order to simulate loads encountered in full-scale walls. The results are shown in Figure 3.11. In general, the vertical pressure was less than overburden pressure near the facing, and higher than overburden pressure at some distance behind the facing, beyond which the pressure was close to the overburden pressure.

Murray (1983) investigated the behaviour of a reinforced earth model wall. The dimension of the model wall was one metre in height, 1m wide and 2m long. The configuration of the reinforcement strips was non-uniform; the length of the strip varied from a maximum at the top of the wall to a minimum at the base. The wall was built on a bed of sand about 150mm deep, which formed the foundation. Pressure cells, 25mm in diameter were placed along the width, at the base of the wall to observe the vertical pressure distribution. The pressure distribution is illustrated in Figure 3.12. It was found that, at the front of the wall, the vertical pressure was much higher than the overburden pressure. The pressure then decreased towards the free end of the bottom-most strip. At a distance of 0.4m from the face and beyond, the vertical pressure was fairly constant. This was in close agreement with the theoretical trapezoidal vertical pressure distribution when
Figure 3.11  Vertical pressure distribution at base of model wall under various centrifugal acceleration $N$, which gives the number of gravities (after Bolton et al., 1978)
Figure 3.12 Vertical pressure distribution at the base of reinforced earth model wall (after Murray, 1983)
the back of the wall (the vertical interface between reinforced zone and backfill) was assumed smooth. Murray (1983) also constructed a series of reinforced earth model walls where the length of strips was uniform throughout the depth of the wall and he found that the observed experimental pressure distribution was in closer agreement to the theoretical trapezoidal distribution when the back of the wall was assumed rough rather than smooth. The analytical method (Murray, 1983) was based on the concept that the friction at the interface will induced the backfill thrust, \( P \) (resultant thrust), to act at \( \phi_m \) inclined to the horizontal, where \( \phi_m \) is the mobilized friction angle at the interface (Figure 3.13). The result gave the maximum vertical stress at the base as:

\[
\sigma_a = \gamma H \left\{ 1 + K \frac{H}{L} \cos \phi_m (\frac{H}{L} - \tan \phi_m) \right\} \quad (3.9)
\]

and the minimum vertical stress at the base as:

\[
\sigma_b = \gamma H \left\{ 1 - K \frac{H}{L} \cos \phi_m (\frac{H}{L} - \tan \phi_m) \right\} \quad (3.10)
\]

where \( K \) is the general earth pressure coefficient
(note: \( P \) have lateral and vertical components)

Murray (1983) had also observed the vertical pressure distribution in full-scale reinforced earth walls. In contrast to that in model walls, the distribution of
Figure 3.13 Influence of friction at the back of the wall on backfill thrust
vertical stress was fairly uniform in full-scale walls. In order to explain the discrepancy from the observed trapezoidal distribution in his reinforced earth model walls, Murray considered the factor of safety of the model and full-scale walls against collapse as well as the geometry of the walls (in terms of their L/H ratios). The L/H ratio for the reinforced earth model wall was 0.5 while the L/H ratio for full-scale walls was from 0.7 to higher than 1. Since a wall with small L/H ratio is more prone to overturning, a large differential pressure as in trapezoidal distribution is expected to develop in the model walls. Murray (1983) also looked at the results of the centrifugal model tests by Bolton et al. (1978) who reported that the vertical pressure was uniform at low stress level and, at high stress levels, the vertical pressure distribution approached the trapezoidal distribution. Thus Murray (1983) concluded that the vertical pressure distribution for a wall which is near collapse is trapezoidal. According to Murray (1983) a trapezoidal distribution can be anticipated for a wall with a low L/H ratio (L/H < 0.7) or for a wall approaching collapse, even if its L/H ratio is relatively large. For a wall with relatively high L/H ratio (L/H ≥ 0.7), or for a wall with a high safety factor against failure, a fairly uniform vertical pressure distribution may be expected.

In their investigation, Murray and Hollinghurst (1986)
instrumented a reinforced earth retaining wall to measure the vertical pressure distribution at the base of the wall. The results obtained were then compared with the trapezoidal distribution as shown in Figure 3.14. It can be seen that vertical pressure near the facing is very small and the vertical pressure at some distance behind the facing is higher than the overburden. This may be due to the effect of the backfill thrust where the overturning moment is inducing a trapezoidal vertical pressure distribution. This hypothesis was supported by the close similarity between the observed and calculated slopes of the vertical pressure distribution curves, at between 2m and 4m from the face (see Figure 3.14). It could be hypothesised that the actual pressure at the facing is much less than the calculated pressure because during the process of construction, the fill near the facing cannot be compacted as well as for the rest of the wall; thus the bulk density of the fill near the wall face is less than assumed. Since there is no real evidence of a trapezoidal vertical pressure distribution Murray and Hollinghurst (1986) suggested that if this is indicative of reinforced earth wall, a safe and more economical design would be one based on a uniform pressure distribution. This conclusion was considered valid on the premise that the observed pressure distribution was indicative of the real behaviour of a reinforced earth wall. In order to really appreciate this context, the design outcome resulting from the
Figure 3.14 Observed vertical pressure distribution at base of full-scale reinforced earth wall compared to calculated trapezoidal distribution (after Murray and Hollinghurst, 1986)
assumption of different vertical pressure distributions is presented at a later stage of this chapter.

3.5 Deformation of Reinforced Earth Wall

Instrumentation is often extremely useful to observe the deformation of a reinforced earth wall. Instrumentation is generally installed at the facing. Yong and McLarin (1985) used inclinometers to measure the tilt of a wall and nominal 40mm alkathene (plastic) tubes to measure the settlement of the foundation. The profiles of the outward face deformation as the wall was being built are shown in Figure 3.15. At full height of the wall, the top three concrete facing panels had tilted back, thus giving the bulging appearance. The maximum outward tilt was 1.0% of the wall height. Vertical settlement of 150mm to 230mm with differential settlement of 50mm to 60mm were recorded during the construction period; however, this was accommodated by the structure without any damage.

Murray and Hollinghurst (1986) measured the horizontal movement of the facing by means of theodolite observations onto coned studs placed on selected facing units. The vertical movement was measured by levelling onto settlement rods, fixed to the footings and brought through the fill in casings. The observed horizontal movement of the wall face is shown in Figure 3.16. Movements were only recorded to a
Figure 3.15 Outward horizontal movement of the wall facing as the reinforced earth wall was being built (after Yong and McLarin, 1985)
Figure 3.16 Horizontal movement of facing of a fully constructed reinforced earth wall (after Murray and Hollinghurst, 1986)
height of 4m since above this level the facing units were not connected to the reinforcement strips. The maximum movement of 13mm was recorded towards the top of the wall. Observation of vertical settlement of the footing at the wall face showed a settlement of 70mm. The settlement was continuing at a rate of 2-3mm per year.

3.6 Main Considerations for Design of a Reinforced Earth Wall

In the design of a reinforced earth wall, consideration has to be given to the internal stability as well as the external stability. Internal stability consideration involves designing the wall against potential pull-out and tensile failures of the reinforcement strips. External stability consideration involves taking the reinforced earth as a gravity block and designing it against potential overturning, sliding, bearing capacity failure and slip circle failure mode.

3.7 Design for Internal Stability

Failure of reinforced earth wall can result from tensile rupture of reinforcement or pull-out failure of the reinforcement. Designing a reinforced wall involves quantifying the tensile force that will develop in each reinforcement strip and ensuring that the tensile strength of the reinforcement and the frictional resistance at the
soil-reinforcement interface is adequate to prevent failure. Several methods are available to quantify the tensile force. These methods consider either the local stability of each reinforcement strip, as in the Rankine Method, or the overall stability of a wedge as in the Coulomb Method (Lee et al., 1973). The wall is in an active state and the vertical pressure distribution at any depth is assumed to be uniform (i.e. vertical pressure equal to the overburden pressure).

3.7.1 The Rankine Method

The Rankine Method is a local equilibrium method where the stability of each reinforcement strip is considered (Figure 3.17). The strip in the ith layer must support the active thrust between the (i - 1)th layer and ith layer: the total active thrust per metre length of wall is calculated based on its tributary area determined by the vertical spacing, $S_v$, of strips. The active thrust that the strip must resist is:

$$T_i = K_a \sigma_{vi} S_v + 0.5 K_a \gamma S_v^2 \tag{3.11}$$

where $\sigma_{vi}$ is the vertical stress acting on the (i - 1) layer $S_v$ is vertical spacing of strips

Hence the vertical stress, $\sigma_{vi}$, may be equated to the overburden pressure, Equation (3.11) may be written as:
\[ T_1 = (i - 0.5)K_a\gamma S_v^2 \]  \hspace{1cm} (3.12)

The maximum tensile force per metre length of the wall occurs at the bottom layer where its magnitude is:

\[ T_{\text{max}} = (n - 0.5)K_a\gamma S_v^2 \]  \hspace{1cm} (3.13)

When \( n \) is large as in the real case, Equation (3.13) can be written as:

\[ T_{\text{max}} = nK_a\gamma S_v^2 = K_a\gamma HS_v \]  \hspace{1cm} (3.14)

3.7.2 The Coulomb Force Method

In the Coulomb Force Method the equilibrium of a wedge is considered (Figure 3.18). The total lateral thrust, \( P_a \), is given by:

\[ P_a = 0.5\gamma H^2\cot\theta \tan(\theta - \phi) \]  \hspace{1cm} (3.15)

The maximum value of the expression on the right hand side may be obtained by equating to zero its derivative with respect to \( \theta \) which gives the well known result:

\[ \theta = 45^0 + \phi/2 \]  \hspace{1cm} (3.16)
Figure 3.17 The local consideration in the Rankine Method

Figure 3.18 Equilibrium of a wedge in the Coulomb Method
Substituting for $\theta$ in equation (3.15) gives the maximum value of lateral thrust and this must be equal to the sum of the tensile forces in the reinforcement strips, $\sum T$. This can be written as follows:

$$\sum T = 0.5\tan^2(45^\circ - \phi/2)yH^2$$

$$= 0.5K_a\gamma H^2$$

(3.17)

By assuming a triangular distribution with no tension at the top layer, the tension per metre length of wall at the $i$th layer is:

$$T_i = \frac{i}{n + 1} K_a\gamma HS_v$$

(3.18)

The maximum tension per metre length of wall will be at the bottom layer where:

$$T_{\text{max}} = \frac{n}{n + 1} K_a\gamma HS_v$$

(3.19)

When $n$ is large, Equation 3.19 assumed the form of Equation 3.14.

In both these methods, the actual distribution of vertical stress was not taken into consideration. The implicit assumption was made that the vertical stress on any horizontal plane is uniform and equal to the overburden
pressure. A more general form of equation (3.14) would be as follows:

\[ T_{\text{max}} = K_a \sigma_v' S_v \]  

(3.20)

where \( \sigma_v' \) is the actual vertical stress to distinguish it from the overburden pressure, \( \sigma_v = \gamma H \), acting at the base of the reinforced earth wall. It should be noted that the maximum value of vertical stress, determined from any one of the vertical pressure distributions must be substituted for \( \sigma_v' \).

3.8 Consideration of Vertical Pressure Distribution in the Design of a Reinforced Earth Wall

By comparing the magnitude of the maximum vertical normal stress predicted from trapezoidal distribution (Equation 3.5) or Meyerhof's distribution (Equation 3.8) with the uniform pressure distribution where \( \sigma_v' \) is equal to overburden pressure, \( \sigma_v \), it can be seen that uniform pressure distribution will give the lowest maximum tensile force when \( \sigma_v' \) is substituted into Equation 3.20. Hence, besides being simple, uniform pressure distribution will result in more economical (but not necessarily as safe as assumed) design.
3.8.1 Stability against tensile failure

The stability of a reinforced earth wall against tensile failure can be obtained by comparing the developed tensile force in the reinforcement strip with its tensile strength. From Equation 3.20 the maximum tensile force for a reinforced earth wall with horizontal spacing of strips, \( S_H \), is:

\[
K_a \sigma_v S_v S_H
\]  
(3.21)

Thus the factor of safety against tensile failure is:

\[
FS_y = \frac{\sigma_y W t_e}{K_a \sigma_v S_v S_H}
\]  
(3.22)

where \( \sigma_y \) is the allowable tensile stress in the strip 
\( W \) is the width of the strip 
\( t_e \) is the thickness of the strip 
\( S_v \) is the vertical spacing of strips 
\( S_H \) is the horizontal spacing of strips

As can be seen from Equation 3.22, the computed factor of safety will depend on the vertical pressure distribution assumed in determining \( \sigma_v' \). Assumption of a uniform vertical pressure distribution will result in a higher value of factor of safety over assumptions of trapezoidal of Meyerhof's distributions.
3.8.2 Adherence stability or stability against bond failure

The adherence stability of the reinforced wall may be assessed according to local stability of a strip or the overall stability of the wall. The frictional resistance of a single strip of reinforcement is defined as:

\[ F_R = 2wL \sigma_v f \]  \hspace{1cm} (3.23)

where \( w \) is the width of the strip
\( L \) is the length of the strip
\( f \) is the coefficient of skin friction of the strip
\( \sigma_v \) is the vertical stress acting on the strip

The factor of 2 arises because both top and bottom surfaces of the strip contribute to the frictional resistance.

The frictional resistance opposes the tendency for a strip to pull-out under a tensile force in the strip. For local consideration, the adherence stability of a reinforced earth wall is governed by the bottom-most strip. The local adherence stability of this strip, assuming a uniform vertical pressure distribution may, therefore, be quantified in terms of a factor of safety, \( F_{S\phi} \), which is the ratio of resisting force, \( F_R \), to the disturbing force, \( K_a \gamma H S_v S_h \). Cancelling \( \sigma_v \) in the numerator with \( \gamma H \) in the denominator, the factor of safety is:
For the overall adherence stability of the wall the effective length of each strip is required. The actual extension of the effective length of strips has been a matter of some controversy. According to some, the whole length of the strip could be considered as the effective length, $L_a$. Other researchers suggest that only the part of the strip which is in the resistant zone constitutes its effective length. If the latter assumption is adopted, the potential failure surface (or the loci of maximum tension, $T_w$) has to be defined. For the Coulomb method, Lee et al. (1973) suggested that, for overall adherence stability of the wall, all of the reinforcement strip need not extend into the resistant zone (Figure 3.19). The frictional resistance, $F_{WR}$, per metre length of wall, assuming uniform vertical pressure distribution, is:

$$F_{WR} = 2\gamma w f N S_v \sum_{i=m}^{n} i [ L - (n-i)S_v \tan(45^\circ - \phi/2) ] \quad (3.25)$$

where $N$ is number of strip per metre length of wall

$m$ is the serial number of the first layer of reinforcement to extend into the resistant zone (counted from the top).
Figure 3.19 The overall adherence stability considerations:
(a) all reinforcement strips extend into resistant zone;
(b) not all reinforcement strips extend into resistant zone
(after Lee et al., 1973)
The factor of safety against pull-out failure for the Coulomb Force Method is obtained by dividing the frictional resistant with the total active thrust:

$$FS_\phi = \frac{4hfNS_v}{K_aH^2} \sum_{i=m}^{n} i \left[ L - (n-i)S_v\tan(45^\circ - \phi/2) \right] \quad (3.26)$$

When the trapezoidal vertical pressure distribution and force equilibrium are considered, the safety factor against adherence failure given by Murray (1983) in terms of overall adherence is:

$$FS_\phi = \frac{2fh}{K_a} \left[ \frac{L}{H} (n + 1) - \frac{\sqrt{K_a}}{3} (n - 1/n) \right] \quad (3.27)$$

This equation was derived assuming that the effective adherence length of each strip is the length that extends beyond the Coulomb failure plane.

The safety factor for overall adherence, assuming Meyerhof's vertical pressure distribution, is:
Schlosser (1977) derived this equation assuming local stability of the strip where the whole length of the strip is considered effective in resisting pull-out. Thus it is to be expected that the analysis the using Equation 3.28 will yield higher factor of safety than Equation 3.27.

### 3.9 An Empirical Method for the Design of a Reinforced Earth Wall

#### 3.9.1 Coherent Gravity Method

The design methods discussed so far are based on the concept of limit equilibrium. In contrast, the coherent gravity method is an empirical method based on the working stress state. It was developed from observation of reinforced earth walls (McKittrick, 1978) with some later modifications suggested by Arenicz and Fredlund (1987). In this method the local equilibrium of individual reinforcement strips against tensile failure and adherence failure is considered. Empirical formulae for factors that affect the design of the wall were developed. These factors are: (a) the soil-reinforcement friction coefficient; (b)
the shape and position of maximum tension line; and (c) the
distribution and intensity of the lateral stress. The coherent gravity method is based on the following assumptions:

(a) the wall is in a state of safe equilibrium;
(b) the loci of maximum tension separate the wall into active and resistant zones;
(c) only that portion of length of each strip which is located in the resistant zone contributes to the frictional resistance of the strip;
(d) the vertical pressure distribution is the Meyerhof distribution;
(e) the apparent friction coefficient, $f^*$; the loci of maximum tension, $x$; and the coefficient of lateral earth pressure, $K$, are given by the following empirical formulae (see also Figure 3.20):

$$f^* = \begin{cases} 
0.4 & \text{for smooth strip} \\
 f_o^*(1 - y/6) + y/6\tan\phi & \text{for } y\leq6\text{m}; \text{ribbed strip} \\
 \tan\phi & \text{for } y>6\text{m}; \text{ribbed strip} 
\end{cases} \quad (3.29)$$

where $y$ is the depth of fill above the level considered $f^*$ is the apparent friction coefficient $f_o^* = (1.2 + \log Cu)$ is the governing value for apparent friction coefficient
Figure 3.20 Design assumptions for Coherent Gravity Method
Cu is the uniformity coefficient of the soil

\[ x = \begin{cases} 
0.3H & \text{for } y \leq 0.5H \\
0.6(H - y) & \text{for } y > 0.5H 
\end{cases} \quad (3.30) 
\]

where \( x \) is the horizontal distance from the facing to the maximum tension line.

\[ K = \begin{cases} 
K_o + (K_a - K_o)y/6 & \text{for } y \leq 6m \\
K_a & \text{for } y > 6m 
\end{cases} \quad (3.31) 
\]

where \( K_o \) is coefficient of at-rest earth pressure.

The equations for the safety factor against tensile failure and adherence failure which are based on the above assumptions may be derived considering the local equilibrium of a strip at level \( i \). The tensile force in the reinforcement at this level is:

\[ T_i = K_o \sigma_v S_v S_h \quad (3.32) \]

Thus the basic equation for factor of safety against tensile failure is:
where \( A \) is the cross-sectional area of reinforcement strip
\( \sigma_y \) is the allowable tensile stress in the reinforcement

The basic equation for the factor of safety against adherent failure is:

\[
FS_y = \frac{A \sigma_y}{T_1} \quad (3.33)
\]

where \( L_a \) is the length of the reinforcement strip that is in the resistant zone.

By combining Equations 3.29, 3.30 and 3.31 into Equations 3.32, 3.33 and 3.34, the empirical formulae for design are derived. The factor of safety against tensile failure for both smooth and ribbed strip is:

\[
FS_y = \frac{2WL_a \sigma_y f^*}{T_1} \quad (3.34)
\]

\[
\text{for } \frac{y^2}{3L^2} \quad 0 \leq y \leq 6m
\]

\[
\gamma [K_o + (K_a - K_o)y/6)yS_vS_H
\]

\[
FS_y = \frac{A \sigma_y (1 - \frac{y^2}{3L^2})}{\gamma K_a yS_vS_H} \quad (3.35)
\]

\[
\text{for } y \geq 6m
\]
Due to the variation in the apparent friction coefficient for smooth and ribbed strip the formulae for the specific types of strip have been derived. The factor of safety against adherence failure for smooth strip is:

\[
FS_\phi = \begin{cases} 
\frac{0.8w(L - 0.3H)}{[K_o + (K_a - K_o)y/6]S_vS_H} & \text{for } 0 \leq y \leq 0.5H \\
0.8w(L - 0.3H) & \text{for } 0 \leq y \leq 0.5H \leq y \leq 6m \\
\frac{0.8w(L - 0.6(H - y))}{K_aS_vS_H} & \text{for } 0.5H \leq y \leq H \\
0.8w[0.6(H - y)] & \text{for } 0.5H \leq y \leq H \geq 6m \\
\end{cases}
\] (3.36)
The factor of safety against adherence failure for ribbed strip is:

\[
FS = \frac{2w(L - 0.3H)[f_o(1 - y/6) + y/6\tan\phi]}{[K_o + (K_a - K_o)y/6]S_v S_H} \quad \text{for } 0 \leq y \leq 0.5H \quad 0 \leq y \leq 6m
\]

\[
FS = \frac{2w(L - 0.3H)\tan\phi}{K_a S_v S_H} \quad \text{for } 0 \leq y \leq 0.5H \quad y \geq 6m
\]

\[
FS = \frac{2w[L - 0.6(H - y)](f_o(1 - y/6) + y/6\tan\phi)}{[K_o + (K_a - K_o)y/6]S_v S_H} \quad \text{for } 0.5H \leq y \leq H \quad 0 \leq y \leq 6m
\]

\[
FS = \frac{2w[L - 0.6(H - y)]\tan\phi}{[K_o + (K_a - K_o)y/6]S_v S_H} \quad \text{for } 0.5H \leq y \leq H \quad y \geq 6m
\]

By using these formulae the cross-section of the reinforcement strip and its length may be determined for any level of \( i \).

3.10 External Stability

In designing for external stability, the reinforced zone is treated as a coherent rigid block and designed as a conventional retaining wall. In order to simplify the design, the friction acting on the back of the reinforced zone and the passive pressure acting near the toe of the wall are both ignored. The failure modes that have to be considered are:

- overturning of the wall
- sliding of the wall
- bearing failure of the foundation
In addition, the overall stability of the wall with respect to the supporting foundation and the adjacent retained fill have to be considered. For checking overall stability, The Department of Transport, (United Kingdom) recommended in their Technical Memorandum (BE 3/78), which deals with the design of reinforced earth structures, that all potential slip surfaces should be considered including those which may pass through the structure. The factor of safety against overall instability should be not less than 1.5 for both short and long term (Brown and Rochester, 1979).

For consideration of potential bearing failure, the analyses are usually based on the assumption that the vertical pressure distribution is trapezoidal. In order to take into account the flexibility of the reinforced earth wall to absorb differential settlement, the allowable bearing pressure on the foundation has been increased from not more than a third of the ultimate bearing capacity of the foundation for conventional wall, to not more than half. In checking for sliding, the factor of safety should be not less than 2. These are the British design recommendations and it should be noted that specifications for external stability may vary from one country to another.
The overturning consideration is rarely critical (Boyd, 1978) since the dimensions of the wall (which determine overturning resistance) is governed by other factors (i.e. bearing capacity failure or internal stability). Nonetheless, the safety factor recommended by the Reinforced Earth company (1985) in Australia is not less than 1.5. The differential settlement should be not more than 2%.

3.11 Concluding Remarks

Investigations on reinforced earth walls have been carried out at limit state and at working stress level with the intention to better understand the mechanics of reinforced earth. The main areas of interest are the development of the tensile stress in the reinforcement strips (including the locus of maximum tension) and the lateral and vertical pressure distributions. Discrepancy of observed results between various walls suggested that the state of stress and strain of reinforced earth wall is dependent on the safety factor of the wall, the geometry and the construction method.

In the design of reinforced earth wall, the reinforcement strip is assumed to carry the lateral stress within its tributary area. Thus the developed tensile stress in the reinforcement strip has to be quantified. The developed tensile stress is affected by the assumed
vertical pressure distribution. Assumption of a uniform vertical pressure distribution results in the least maximum tension, whereas a trapezoidal distribution yields the largest maximum tension in the strip. Since the frictional resistant of a strip is dependent on the vertical pressure that acts on it, the vertical pressure distribution has an effect on the analysis of the adherence stability of the entire wall.

One of the main concerns of this thesis is the vertical pressure distribution in reinforced earth wall. With respect to this, experimental and numerical investigations which are related to vertical pressure distribution in reinforced earth are presented in Chapters 4, 5 and 8.
Chapter 4

The Effect of Backfill Thrust on Vertical Pressure Distribution in Reinforced Earth Model Walls

4.1 Introduction

In the design of reinforced earth walls, the magnitude and distribution of the vertical pressure may be calculated from several theoretical models (Jones, 1985) such as uniform, trapezoid and Meyerhof's; the latter being most commonly used at present. Although each of them is based on the assumption of the reinforced zone being a coherent rigid mass, they differ regarding the effect of (unreinforced) backfill thrust on the distribution of vertical stress in the adjacent reinforced zone. There is a growing recognition that such an effect takes place in most, if not all, reinforced earth structures, and hence ought to be investigated and quantified to assess validity of the theoretical stress distributions used in design. An extensive computer search of reinforced earth publications has failed to find any report on investigations of this particular problem.

The uniform pressure theory neglects any effect of backfill thrust and hence vertical stress \( \sigma_v \) at any depth \( h \) is calculated from the well-known expression:

\[
\sigma_v = \gamma h
\]

(4.1)

where \( \gamma \) represents the bulk unit weight of reinforced soil.
The trapezoid theory assumes that the back (vertical boundary) of the reinforced zone is smooth and that lateral pressure on it increases linearly with depth. A moment balance and further assumption that \( \sigma_v \) may vary linearly along any horizontal plane at depth \( h \) (measured from the top surface of the wall), yields:

\[
\begin{align*}
\sigma_v &= \gamma h (1 + K_a h^2/L^2) \quad \text{at the face of reinforced zone} \quad (4.2) \\
\sigma_v &= \gamma h (1 - K_a h^2/L^2) \quad \text{at the back of reinforced zone} \quad (4.3)
\end{align*}
\]

where \( K_a \) is coefficient of active earth pressure

\( L \) is reinforcement length (extent of reinforced zone)

As assumed the vertical pressure will vary linearly between these two limits and this implies that the reinforced zone responds in a manner similar to a rigid foundation.

Meyerhof’s theory assumes that \( \sigma_v \) is constant at any level considered and that the confining pressure near the back of reinforced zone is zero. Hence, a moment balance which takes into account the backfill thrust yields:

\[
\begin{align*}
\sigma_v &= \begin{cases} 
\gamma h (1 - K_a h^2/3L^2)^{-1} & \text{for } 0 \leq x \leq \frac{K_a H^2}{3L} \\
0 & \text{for } \frac{K_a H^2}{3L} \leq x \leq L
\end{cases} \quad (4.4)
\end{align*}
\]

where \( x \) is a horizontal coordinate taken from the face of the reinforced zone towards its back and \( H \) is the wall height.
The comparison between the three vertical pressure distributions is shown in Figure 4.1.

The aim of the investigation presented herein was two-fold:

a) firstly, to find the actual effect of backfill thrust on the distribution of $\sigma_v$ from a series of laboratory tests of reinforced earth model walls, and
b) secondly, to assess the validity of the three theoretical distributions of $\sigma_v$ by comparison with the results of the laboratory tests.

4.2 Experimental Study

Series of laboratory tests were conducted on instrumented reinforced model walls built within the confinement of a specially constructed rigid box. Pressure transducers were placed at the base of the box to measure vertical pressure along the reinforced zone. In order to determine the effect of backfill thrust on the vertical pressure, measurements were taken before and after the backfill thrust on the reinforced zone was introduced.

Initially, the reinforced zone was separated from the backfill by a 3.4-mm thick steel plate extending over the whole width and depth of the model box. Rigidity of the partition was high enough to prevent yielding and hence the adjacent soil was in a state of rest and the lateral pressures on each side of the partition were in the opposite direction. When the partition was lifted off,
Figure 4.1 Theoretical vertical pressure distribution

- **a) UNIFORM DISTRIBUTION**
- **b) TRAPEZOID DISTRIBUTION**
- **c) MEYERHOF'S DISTRIBUTION**
active lateral pressure of the backfill was exerted on the reinforced zone (see Figure 4.2). The partition was in place before construction of the model wall and, during construction, sand was placed layer by layer on both sides of the partition separating the reinforced zone from the backfill. When the wall reached the desired height, the readings of the pressure transducers were taken. The backfill thrust was gradually introduced to the reinforced zone by lifting the partition up very slowly. The readings of the pressure transducers were again taken when the partition was completely removed. The details of the experimental set-up, materials and procedures are given below.

4.2.1 Model box

The reinforced earth model walls were built within a glass box 800mm wide, 1000mm long and 600mm deep. The glass sides minimized friction and along with the fact that the ratio of the model wall height (300mm) over its width was 2.67, which is well over the value of 1.25 recommended by Hausmann (1978), arching of stresses caused by the side walls was considered to be minimal.

Two accessories were used in the model walls construction: a motorized hopper that ran along the length of the box to deposit the sand fill and a facing element support frame. The frame consisted of a square hollow section sliding up and down on two steel rods (Figure 4.3).
Figure 4.2 Lateral pressure at partition
Figure 4.3  Facing element support frame
The section provided temporary support for a facing element while the adjacent 50-mm layer of sand was being placed. The base of the support frame formed a 15mm berm to a model wall. The overall view of the model box is shown in Figure 4.4.

4.2.2 Reinforcement

The reinforcement strips were smooth shim steel 0.2mm thick, 10mm wide and 370mm long. The tensile strength of the shim steel is 580MPa and since the objective is to build stable reinforced earth model walls, this ensured the reinforced earth model walls will not fail due to tensile failure of reinforcement strips.

The length of the reinforcement was chosen to exceed 0.8 times the final height of model walls (300mm) - a widely accepted empirical requirement for stability of reinforced earth walls (Ingold, 1982). The width of the strip was chosen to provide adequate surface for sand-reinforcement interaction while maintaining a low stiffness of the reinforcement strips. Low stiffness was desirable in order to simulate non-rigid strip reinforcement of full-scale walls.

The ratio of reinforcement length over the height of the model was 1.2, thus according to Petrik (1979) the strain field of the backfill has maximum homogeneity.
Figure 4.4 General view of model box
4.2.3 Fill

Dried local beach sand with uniformity coefficient of 2.0 was used as a construction material. Its grain size distribution is illustrated by Figure 4.5. The sand was rained through a hopper which travelled along the length of the wall and remained at the height of 600mm from the base of the wall. No compaction of the sand was attempted. The raining process gave a fill with the unit weight of 15.66kN/m³ and void ratio of 0.63. This method of placing the fill has been used by many researchers for reinforced earth model wall tests, e.g., Narain et al. (1981), Murray (1983) and Nagel (1985). The relative density of the sand was 61%; thus it can be classified as medium density fill (Lambe and Whitman, 1979). The relative density test was done according to ASTM standard test procedures (Bowles, 1978) where the minimum and maximum densities obtained were 14.56 and 16.44kN/m³, respectively.

The internal friction angle determined from a direct shear test was 35.4° (Appendix E). The test was done using a 60mm square shear box and the shearing rate applied was 1.2mm/min. The range of applied normal pressure was 14.51kPa to 28.14kPa. The density of the sand was 15.6kN/m³ and the initial void ratio was 0.63. Low confining pressures were applied in the test to simulate, as close as possible, the low confining pressures within the reinforced earth model wall.
<table>
<thead>
<tr>
<th>Sand</th>
<th>Coarse to medium</th>
<th>Fine</th>
<th>Silt</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S. standard sieve sizes</td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>No.4</td>
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<td>No.10</td>
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<td>No.100</td>
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<tr>
<td>No.200</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.5 Grain size distribution of fill material
4.2.4 Facing elements

The facing elements, each extending over the entire width of the wall, were made from galvanised steel (Figure 4.6). The height and thickness of each element was 50mm and 1.46mm, respectively. Its ends were lined with felt so that sand was contained behind the facing but no part of the facing element touched the glass sides of the box. Each element had a horizontal slit at its centre to allow attachment of the reinforcement strips and washers at its bottom to allow connections between facing elements. This type of connection allowed rotation while preventing translation.

4.2.5 Instrumentation

Six pressure transducers (PT1 to PT6) were placed at the centre line of the base of the wall where the readings were less affected by the glass sides of the box. The box itself was secured to a rigid timber table serving as a base on which the model walls were built. The transducers were TYCO AB with measuring range of 0 to 6.7 kPa. The diameter of the measuring surface was 19mm. The position of the transducers within a fully constructed reinforced earth model wall is shown in Figure 4.7.

The transducers had been calibrated with water where the measuring surface was fitted to a 1.5m long clear plastic tube which was held vertically on a wall. Various
Figure 4.6  The facing of reinforced earth model wall showing facing elements
Figure 4.7  Configuration of model wall showing position of pressure transducers
depths of water were poured into the tube to attain pressures within the range which will be encountered in the model walls. The calibration charts are shown in Appendix E2. The manufacturer's specifications are in Appendix E1. During calibration and in actual tests of reinforced earth model wall, the transducers were supplied with 5V direct current and the output voltages were read by a voltmeter.

In the model wall tests, the pressure transducers were mounted to the base of the wall using modelling clay (Plasticine) whereby a good positioning of the pressure transducers was achieved. The pressure transducers were equally spaced and the measuring surface of each pressure transducer was levelled at 45mm from the base of the wall. Inspection after the model wall was dismantled revealed that the position of the pressure transducers remained unchanged.

4.2.6 Test procedures

A total of eleven tests were conducted in four series. The procedure involved in the first series (test A1 to A3) consisted of the following steps:

1) placing the pressure transducers and the partition;
2) aligning the first facing element and temporarily holding it vertically using the supporting frame;
3) raining 25-mm high layer of sand into both parts of the box separated by the partition;
4) taking a reading of the pressure transducers;
5) placing four reinforcement strips on the sand bed at 200mm distance apart (centre to centre);
6) attaching reinforcement strips to the facing element;
7) raining another 25mm high layer of sand;
8) taking reading of the pressure transducers;
9) placing another facing element on top of the previously laid one and sliding the supporting section to this element;
10) repeating procedures (3) to (9) until the wall reached a height of 300mm;
11) lifting the partition up very slowly and then taking the final readings of the pressure transducers.

(During construction of reinforced earth model walls, the pressure transducers readings were taken every 25mm increment of fill.)

In tests A1 to A3, the spacing of pressure transducers was 70mm. This arrangement gave a distance of 420mm from the facing and a gap of 50mm from the free end of the reinforcement strip to the surface of the partition. The purpose of these tests was to measure the vertical pressure distribution before and after the backfill thrust was applied to the reinforced zone.

In the subsequent series of tests, the spacing of pressure transducers was reduced to 63mm since the partition was relocated to a new position being 42mm closer
to the facing unit. It was found that the 50-mm gap could be reduced to 8mm, still adequate to prevent the reinforcement strip from moving or coming into contact with the partition when it was being lifted. The smaller gap resulted in a better representation of the reinforced zone. The locations of the pressure transducers are denoted as gauged points PT1, PT2, PT3, PT4, PT5 and PT6. The distance of these gauged points from the facing for all the series of tests are given in Table 4.1.

Table 4.1 Positions of Gauged Points with respect to Wall Face

<table>
<thead>
<tr>
<th>Gauge Point</th>
<th>Distance from facing for tests A1 to A3</th>
<th>Distance from facing for other tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>0.094L = 35mm</td>
<td>0.085L = 31mm</td>
</tr>
<tr>
<td>PT2</td>
<td>0.284L = 105mm</td>
<td>0.255L = 94mm</td>
</tr>
<tr>
<td>PT3</td>
<td>0.473L = 175mm</td>
<td>0.426L = 157mm</td>
</tr>
<tr>
<td>PT4</td>
<td>0.662L = 245mm</td>
<td>0.596L = 220mm</td>
</tr>
<tr>
<td>PT5</td>
<td>0.851L = 315mm</td>
<td>0.766L = 283mm</td>
</tr>
<tr>
<td>PT6</td>
<td>1.040L = 385mm</td>
<td>0.936L = 346mm</td>
</tr>
</tbody>
</table>

In the second series of tests (B1 to B4), the spacing of the transducers was 63mm (as explained above) and the test procedure was the same as described in the first series. The purpose of the tests was the same as that of Series A (i.e. to measure the vertical pressure before and after the application of backfill thrust). Only the spacing of the transducers was varied.
In the third series (tests C1 and C2), the spacing of pressure transducers was again 63mm apart and the backfilling was only done after the reinforced zone reached the height of 300mm. The partition was lifted when the backfill reached the same height (300mm). Otherwise the test procedure remained the same. This type of construction simulated those of the gravity concrete wall where the backfill is placed only after the concrete structure is fully erected. The purpose of these tests was to check the effect of different construction procedures on the vertical pressure distribution.

Finally, the fourth series (test D1 and D2) was conducted as the third one but the partition was not lifted. The test was to observe the vertical pressure within the reinforced block as the backfill was gradually placed. The result from this test as will be seen later, will be used to validate the concept used to analyse the results of the first three series.

4.3 Test Results

The pressure readings for each test is presented in Appendix A. For each series the average readings were taken as representative of that series.

Based on the test results, pressure distribution diagrams between PT1 and PT6 were drawn for each series (Figures 4.8 to 4.10). The diagrams were not extrapolated
Figure 4.8 Pressure distribution diagrams for First and Second series
Figure 4.9 Pressure distribution diagrams for Third series
Figure 4.10 Pressure distribution diagrams for Fourth series
beyond the transducers. However, since the distance from
the wall face to PT1 and from PT6 to the end of the strip
were relatively small, the diagrams drawn were considered
to be a good representation of pressure distribution within
the reinforced zone of model walls.

The nonlinear shape of the vertical pressure
distribution prompted a check on the pressure transducers
at the early stage of the experiments: to see if there was
any mechanical or electrical faults. The test done was
similar to the one which will be described in Chapter 5,
Section 5.3.1, and the result showed that all of the
pressure transducers were in good working order.

4.3.1 First Series

The pressure readings for this series are tabulated in
Tables A.1 to A.4 in Appendix A. The pressure distributions
obtained from test A1 to A3 are illustrated by Figure 4.8a.
Under a fill height \( d^* \) of 100mm, the pressure distribution
was fairly uniform. Under subsequent fill heights of 150mm,
200mm, 250mm, 300mm (before lifting the partition) and
300mm (after lifting the partition) the pressures at the
middle of the reinforced zone were lower than those at the
boundaries (the facing and the rear). Under the fill
heights of 200mm, 250mm, 300mm (before lift) and 300mm
(after lift) the pressures at PT1 were noticeably higher
than at the rest of the gauged points. The increase in
pressure was fairly constant for the incremental fill
height of 50mm. Under the fill heights of 150mm, 200mm,
250mm, 300mm (before lift), pressures at PT6 were lower than at PT5. However when the partition was lifted, pressure at PT6 was higher than at PT5.

* Note that fill height \( d \), is measured from the base, thus the overburden depth, \( h \), above the measured plane or surface is: \( d - 45\text{mm} \).

The tests show that the backfill thrust increases the vertical pressure along the reinforced zone. This effect lessens with increasing distance from the rear end of the reinforced zone.

4.3.2 Second Series

The pressure readings for this series are tabulated in Tables A.5 to A.9 in Appendix A. The pressure distributions obtained from test B1, B2, B3 and B4 are illustrated by Figure 4.8b. Under the fill height of 100mm the pressure was fairly uniform. Under the fill heights of 200mm, 250mm, 300mm (before lift) and 300mm (after lift) the pressure at PT1 were higher than at other gauged points and the pressure in the middle of the reinforced zone was lower than that elsewhere in the zone. Under the fill heights of 200mm, 250mm and 300mm (before lift) the pressure at PT6 was lower than at PT5. When the partition was removed the pressure at PT6 was higher than at PT5.

The tests show that the backfill thrust increases the vertical pressure at all points within the reinforced zone. This influence lessens with increasing distance from the
rear end of the reinforced zone or the back of the reinforced earth model wall.

4.3.3 Third Series

The pressure readings for this series are tabulated in Tables A.10 to A.15 in Appendix A. The pressure distributions obtained from tests C1 and C2 are shown by Figures 4.9a and 4.9b. Figure 4.9a shows the pressure distributions as the reinforced zone was being built. Under the fill height of 100mm the pressure was fairly uniform with a slightly lower pressure at the middle of the zone. A similar curve was found under fill height of 150mm, however the pressures at the middle were much lower than at the two boundaries. Under the fill heights of 150mm, 200mm, 250mm and 300mm the pressure at PT1 were higher than at PT6. Under the fill heights of 200mm, 250mm and 300mm the pressure at PT6 was lower than at PT5. Pressures at PT1 and PT5 were similar under all the fill heights.

The pressure readings as the zone beyond the partition was being filled (tests C1 and C2, respectively) are tabulated in Tables A.11 and A.13 respectively. The average result is tabulated in Table A.15. The result shows that the pressures at PT1, PT2, PT3, PT4 and PT5 increase while the pressure at PT6 decreases with increasing height of the fill beyond the partition. The pressure at PT6 was smallest when the backfill height was equal to the height of the reinforced zone. After the partition was lifted, the
pressures at all gauged points increased and the pressure at PT6 was quite similar to when there was no backfill beyond the partition (see Table A.15).

4.3.4 Fourth Series

The pressure readings for tests D1 and D2 are presented in Tables A.17 to A.21 in Appendix A. The average results from these two tests are taken as representative of the series. The pressure distributions are shown by Figures 4.10a and 4.10b.

Figure 4.10a shows that, as the reinforced zone was being built, the pressure under the fill height of 100mm was fairly uniform. Under the fill height of 150mm the pressure at the two boundaries was quite similar and there was lowering of pressure around the middle of the reinforced zone. Under the fill heights of 200mm, 250mm and 300mm the pressures at PT1 and PT5 were quite similar and the pressure at PT6 was lower than at PT5.

Table A.21 in Appendix A, and Figure 4.10b show the pressure distribution as backfilling was done behind the reinforced zone. It shows that with increasing backfill height the pressures at PT2, PT3, PT4 and PT5 increase while the pressures at PT1 and PT6 decrease. The pressure at PT6 was reduced up to 33% and the pressure at PT1 was reduced up to 3%. The maximum increase of 37% occurred at PT4.
The total vertical force as the backfilling was being done behind the reinforced zone was analysed and presented herein, since its result will later form an important concept in the analysis of the effect of the backfill thrust. It was found that as the backfilling was being done, the total vertical force within the reinforced zone increases with the height of the backfill $S$, (Table 4.2). This may be attributed to frictional force mobilized at the soil-partition interface since the partition was not perfectly smooth. As the backfill was gradually built, the lateral thrust tends to overturn the reinforced block outwards (about the toe) thus the frictional force acts downward to resist the motion, therefore increasing the total vertical force. This explanation was proven by a check (Figure 4.11): when an assumed value of mobilized friction angle $\phi_m$ of $30^0$ (a value less than $\phi$ of $35.4^0$), was mobilized along the partition, the frictional force which is $P \tan \phi_m$ (or $P \mu$) was found to be $0.078 \text{kN/m}$, a value which is exactly the difference between the the total vertical force when $S = 300\text{mm}$ and $S = 0$ (see Table 4.2).
Table 4.2 Influence of Backfill Height on Total Vertical Force in Fourth Series (average from test D1 and D2)

<table>
<thead>
<tr>
<th>Backfill height S (mm)</th>
<th>Total vertical force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.1787</td>
</tr>
<tr>
<td>100</td>
<td>1.2042</td>
</tr>
<tr>
<td>150</td>
<td>1.2153</td>
</tr>
<tr>
<td>200</td>
<td>1.2367</td>
</tr>
<tr>
<td>250</td>
<td>1.2442</td>
</tr>
<tr>
<td>300</td>
<td>1.2562</td>
</tr>
</tbody>
</table>

Backfill thrust tends to tilt the wall

Frictional Force $F_w = P_a \mu = P_a \tan \phi_m$

assuming $\phi_m = 30^\circ$, $F_w = 1/2 \times 0.266 \times 15.66 \times 0.255^2 = 0.078$ kN/m

Figure 4.11 Friction force mobilized at the back of the wall
4.4 Analysis of the Experimental Results

The pressure distribution curves for all the tests show that the pressure was highest near the facing. A common feature of these curves was that the pressure in the middle of the reinforced zone was always lower than at the two boundaries. This phenomenon was more distinctive with the increase in fill height. It can be seen that the pressure at the middle part of the reinforced zone was lower than the overburden, while at the boundaries it was higher than the overburden. This could possibly be attributed to a kind of arching phenomenon across the dimension L. However, since this feature also occurred when there was no backfill behind the partition, as during construction of the reinforced zone in (complementary) tests Cs and Ds, it can be concluded that the arching was not caused by the backfill thrust. It is shown later (Chapter 8), that the finite element analysis of reinforced earth wall indicates the existence of stress concentration at the toe of the wall. At the facing, the vertical normal stress in these analyses (Finite Element Method) is higher than overburden while being lower than overburden around the middle region of the reinforced zone.

The total vertical force from the measured pressure
distribution (before lift) was found to be generally less than the total weight of the overburden (see Table B.1 in Appendix B). This can only be attributed to some kind of arching across the dimension B. It should be noted from Figure 4.7 that the transducers give only the picture of vertical stress at one position i.e. the middle of width B. Secondly, the total vertical force from measured pressure distribution (after lift) are larger than the total measured vertical force (before lift). The explanation for this observation is that frictional force was mobilized at the back of the wall (interface between the reinforced and the unreinforced zones) when the backfill thrust was introduced. The backfill thrust tends to overturn the wall outwards hence the frictional force acts in a downward direction to resist this movement. Therefore the total vertical force consisted of the frictional force and the weight of the wall. This phenomenon has been proven in the fourth series (tests D1 and D2) where the total vertical force within a reinforced earth block increases with the height of the backfill. For analytical proof, the theoretical trapezoid vertical pressure distribution assuming the reinforced earth wall acts as a rigid body and that the back of the wall is frictional, is derived below and illustrated in Figure 4.12.

\[ W = \gamma HL, \quad P_a = \frac{1}{2} K_a \gamma H^2, \quad F_w = \mu P_a \]

From equilibrium of vertical forces:

\[ \gamma HL + \mu P_a - \frac{1}{2}(\sigma_a + \sigma_b)L = 0 \]

\[ \sigma_a + \sigma_b = 2\gamma H + \frac{2\mu P_a}{L} \] (1)
(a) A free body diagram of a reinforced earth wall (assume to act as a rigid body)

(b) Components of Vertical Stress

Figure 4.12 Derivation of vertical stress (assuming reinforced earth wall acts as a rigid body)
From equilibrium of moment about the centre of the wall:

\[ \frac{1}{2}(\sigma_a - \sigma_b) \times L \times \frac{L}{6} - P_a \times \frac{H}{3} + F_w \times \frac{L}{2} = 0 \]

\[ \sigma_a - \sigma_b = \frac{4P_a H}{L^2} - \frac{6\mu P_a}{L} \]  

(2)

Summation of (1) and (2):

\[ \sigma_a = \gamma H + \frac{2P_a H}{L^2} - \frac{2\mu P_a}{L} \]  

(4.5)

Substitute Equation 4.3 into (1):

\[ \sigma_b = \gamma H - \frac{2P_a H}{L^2} + \frac{4\mu P_a}{L} \]  

(4.6)

In both equations for vertical stress or pressure at the facing, and at the back of the wall, the first term is the component of stress due to overburden; the second term is the effect of the lateral thrust while the third term is the effect of friction.

From the pressure diagrams it can be seen that the effect of the lateral thrust is to reduce the vertical pressure at the back of the wall and increase the vertical pressure at the facing, while the frictional force tends to increase the vertical stress at the back of the wall and reduce the vertical stress at the facing. The resulting vertical pressure distribution is the summation of the stresses due to the three components: weight of the wall, lateral thrust and frictional force.
Going back to the experimental results, it was considered that the effect of frictional force at the back of the reinforced earth model wall should be separated from the effect of the lateral (horizontal) overturning force. From here onwards the term 'lateral thrust' is used to define the action of the backfill thrust alone i.e. excluding the frictional effect. To do this a procedure was adopted as shown in Appendix B. In effect the vertical pressure at the back was adjusted for friction. The frictional force was calculated as the difference $\Delta p$, in the areas of the observed pressure diagram after and before lifting the partition. This force was then assumed to act over a width of $0.07\text{mm}$ (distance between PT5 and PT6) for the first series (tests A2 and A3) and a width of $0.063\text{mm}$ (distance between PT5 and PT6) for the second and third series (tests B1 to B4, C1 and C2). The pressure at PT6 was corrected on this basis. From the corrected pressure, the percentage decrease due to lateral thrust alone was calculated. At all other parts the increased or decreased pressure was based on uncorrected before and after observed values. This led to Figure 4.13.

The average result for the first series (tests A2 and A3) is tabulated in Table C.2 and it is illustrated by curve As in Figure 4.13. It can be seen that the lateral thrust decreases the vertical pressure by 30% at the end of the reinforced zone and increases the vertical pressure within the remainder of the reinforced zone. The maximum increase of 14% occurred at the vicinity of PT5 and the
Figure 4.13 An interpretation of stress change due to lateral thrust based on correction for friction at PT6
increase in pressure was reduced to 3% at the facing. The percentage increase or decrease in vertical pressure along the reinforced zone for tests B1 to B4 is tabulated in Table C.1 in Appendix C and the average result for this series is tabulated in Table C.2. The result is illustrated by curve Bs in Figure 4.13. It can be seen that the lateral thrust decreases the vertical pressure by 33% at the rear end of the reinforced zone and increases the vertical pressure elsewhere. The maximum increase of 20% occurred at the vicinity of PT5 and the increase in pressure reduces to 2% at PT1. The result for the third series (tests C1 and C2) is shown in Table C.2, Appendix C, and illustrated by curve Cs in Figure 4.13. It can be seen that the lateral thrust reduces the pressure at PT6 by 55% while increasing the pressure at PT1, PT2, PT3 and PT5. There was no increase at PT4.

The effect of the lateral thrust on the vertical pressure distribution for the first and second series was similar. This was expected since the only difference between the two series was the spacing of the pressure transducers. The lateral thrust reduced the pressure at the rear end of the reinforced zone and increased the pressure within the remainder of the reinforced zone. In the vicinity of the facing, the effect of the lateral thrust on the vertical pressure distribution was minimal.

Results of the third series also confirmed the above observation even though the recorded increases in vertical
pressure were not the same as in the first and second series, which may be attributed to the different construction method used.

The analysis shows that if the influence of friction is separated, the backfill thrust decreases the vertical pressure at the rear end of the reinforced zone and increases the pressure at the remaining part of that zone. The influence of the backfill thrust on the vertical pressure lessens with increasing distance from the rear end of the reinforced zone.

4.5 Comparison with Theoretical Vertical Pressure Distributions

The following comparison is based on results from the second series (tests B1 to B4). In these tests the actual construction procedure of reinforced earth walls was simulated (as it was in the first series, As). The results obtained from this series were chosen for comparison with theoretical pressure distributions - uniform, trapezoidal, and Meyerhof's - described in the introduction to this chapter. The comparison is shown in Figure 4.14 (see also Appendix D). It can be seen that near the boundaries of the reinforced zone the recorded vertical pressure was higher than that predicted by the theories and in the middle of the zone the pressure was lower than that predicted theoretically. Generally, the distribution of the vertical pressure during the tests was definitely nonlinear as opposed to the theoretical linearity.
Figure 4.14 Comparison of test results with theoretical vertical pressure distributions
The most prominent and consistent feature of the observed vertical pressure distribution was the arching in the middle of the reinforced zone. In order to check whether this arching was due to the presence of reinforcement strips, results obtained from another series of tests were used where a model wall was built without using any reinforcement strips. The facing elements were taped to a rigid partition which served as the front of the wall. The partition was externally supported by a metal frame. The result of these additional tests shows that even without reinforcement there was a dip in pressure at some distance behind the facing. (This test is referred to again in Chapter 5).

The effect of the backfill thrust (lateral thrust) on the vertical pressure distribution within the reinforced earth model wall is shown by the hatched area in Figure 4.15. It shows that the backfill thrust reduces the vertical pressure at the rear end and increases the vertical pressure in the remaining part of the reinforced zone. The positive hatched area is approximately the same as the negative one. It indicates clearly that the backfill thrust causes redistribution of vertical pressure within the soil structure. The diagram also shows that the maximum effect of the backfill thrust occurs near the free end of the reinforcement while towards the facing this effect is diminishing.
Figure 4.15 Observed effect of backfill thrust on vertical pressure distribution after correction for friction force at PT6 based on author's proposed approximation
4.6 Implied Effect of Backfill Thrust on Wall Stability
(Considering moments about the toe)

It is of interest to consider the moment (at a particular point) due to a vertical pressure distribution. The point taken was gauge point PT1 which can be considered as the toe of the wall. The aim of the analysis was to find the effect of the backfill thrust by comparing the moment due to the observed vertical pressure distribution when there was no backfill thrust, with the moment due to the vertical pressure distribution when there was backfill thrust. (Note: friction effect has been separated).

The pressure diagrams that were used in this analysis are found in Figure 4.15. The moments about PT1 were calculated from the vertical pressure distribution when there was no backfill thrust (before lift) and when there was backfill thrust (after lift). The moment (clockwise is positive) about the toe, per linear metre of the reinforced earth model wall, when there was no backfill thrust is 191.44Nm while the moment when there was backfill thrust is 185.25Nm. It can be seen that the backfill thrust caused a reduction of 3.2% of the moment about the toe. This would imply that the backfill thrust reduces the overturning resistance about the toe of the wall. Since the backfill thrust is an additional horizontal force considered, it obviously reduces the sliding resistance of the wall as well.
4.7 Comparison of Moments from Observed and Theoretical Vertical Pressure Distributions

The moment analysis, as described in Section 4.6, can also be used for validation (or otherwise) of the theoretical vertical pressure distribution (trapezoidal theory) which consider the effect of the backfill thrust. The pressure diagrams used for this analysis are those in Figure 4.14. The moment about PT1 was calculated for the uniform, trapezoidal, and experimental distributions. Before calculation of the moment were made, the area of the pressure diagram for uniform and trapezoidal distribution had to be adjusted so that it is equal to that of the experimental diagram. This had to be done so that any difference in the moment, calculated from the various pressure diagrams can only be attributed to the shape of the diagram, where the shape of the diagram is the vertical pressure distribution. The area of the diagrams (Figure 4.14) which give the total vertical force for the trapezoidal and the uniform distribution are 1254.5N/m and 1257.8N/m respectively; and these diagrams had to be adjusted so that the area is equal to that of experimental's which is 1242.9N/m. During the adjustment, the ratio of the vertical pressure at PT1 and PT6 was kept similar to that before the adjustment so that the shape of the pressure diagram remains unaffected. This is the reason why the analysis will not and could not be made on the Meyerhof's distribution since the adjustment will make it equal to the uniform distribution. The reason being that the width 2e_m...
is 15.6mm, hence it is outside the gauged length.

For the uniform distribution, the vertical normal stress was initially constant at 3.993kPa. Adjustment to the uniform distribution resulted in a constant vertical normal stress of 4.075kPa while, for trapezoidal distribution, the vertical normal stress at PT1 changed from 4.412kPa to 4.371kPa; and at PT6 the vertical normal stress changed from 3.553kPa to 3.520kPa. These adjusted values were then assumed for the calculation of moment about PT1, with the results given in Table 4.3.

Table 4.3 Moment about the Toe (after eliminating effect of differences in total vertical force)

<table>
<thead>
<tr>
<th>Vertical pressure distribution</th>
<th>Moment (Nm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>uniform</td>
<td>195.7</td>
</tr>
<tr>
<td>trapezoidal</td>
<td>188.7</td>
</tr>
<tr>
<td>experimental</td>
<td>185.2</td>
</tr>
</tbody>
</table>

Note: the absolute value of moment depends on wall size (H and L)

As stated earlier in this chapter, the uniform pressure distribution does not take into account the effect of the backfill thrust, while the trapezoidal distribution does include such an effect. Comparison of the moments resulting from these two distributions show that the backfill thrust reduces the moment about the toe by 3.6%. This is very close to the value obtained experimentally.
by considering the moments before and after removal of partition in the second series (Bs).

It can be seen that the moment due to the trapezoidal distribution is almost the same as that from the experimental distribution. It can be concluded that although the distribution from the trapezoid theory was not fully reproduced in experimental model walls, the overall effect on stability is similar considering overturning about the toe. The effect of the backfill thrust can be seen as reducing the overturning resistance about the toe.

4.8 Friction at the Back of the Reinforced Earth Wall and the Backfill Thrust (Basic Analytical Consideration)

Theoretical analysis based on the trapezoid theory was made on the effect of the backfill thrust on the vertical pressure distribution within the reinforced earth wall when friction at the back of the wall is considered. Equation 4.5 was used for the calculation of the vertical normal stress at the facing, $\sigma_a$, while Equation 4.6 was used for the calculation of the vertical normal stress at the back of the wall, $\sigma_b$. In order to study the effect of friction, comparison was made of the trapezoidal distribution when the back of the wall was assumed smooth, with the trapezoidal distribution when friction at the back of the wall is considered. (Note: Equations 3.9 and 3.10 developed by Murray (1983) would have given similar results where $K \cos \phi_m$ is equal to $K_a$)
An analysis was undertaken to find the vertical pressure distribution at the base of a reinforced earth model wall similar to the experimental model wall. The dimensions L was 370mm and H was 300mm. The coefficient of earth pressure $K_a$ was 0.266, corresponding to an angle of internal friction $\phi$ of 35.4° and the unit weight $\gamma$ is 15.66kN/m². A mobilized friction angle at the back of the wall $\phi_m$ of 30° was assumed. This was also the value assumed by Murray (1983). The vertical normal stress at the facing $\sigma_a$, and vertical normal stress at the back of the wall $\sigma_b$ (and resulting total vertical force) are presented together with the values obtained when the back of the wall was assumed to be smooth i.e. $\phi_m = 0$ in Table 4.4. These values are to be compared with the vertical normal stress assuming a uniform distribution, where the vertical normal stress along the base of the wall is constant at 4.698kPa (giving total vertical force of 1.738kN/m).

Table 4.4 Theoretical Vertical Normal Stress and Total Vertical Force (based on trapezoid theory)

<table>
<thead>
<tr>
<th>Position</th>
<th>Vertical stress (kPa)</th>
<th>$\phi_m = 0$</th>
<th>$\phi_m = 30^\circ$</th>
</tr>
</thead>
<tbody>
<tr>
<td>at facing</td>
<td>4.866</td>
<td>4.283</td>
<td></td>
</tr>
<tr>
<td>at the back</td>
<td>4.530</td>
<td>5.696</td>
<td></td>
</tr>
<tr>
<td>Total vertical force</td>
<td>1.738kN/m</td>
<td>1.846kN/m</td>
<td></td>
</tr>
</tbody>
</table>

The result obtained was used to draw Figure 4.16. It
Figure 4.16 Theoretical vertical pressure distribution showing the effect of friction at the back of the wall (for a particular case)
can be seen that the friction at the back of the wall increases the vertical stress at the back of the wall and reduces the stress at the facing. Maximum vertical stress occur at the back of the wall and minimum stress is at the facing, as opposed to minimum stress at the back, and maximum stress at the facing, when there is no friction at the back of the wall. The magnitude of the maximum stress was higher when there was friction at the back of the wall. This would have an effect on the consideration of bearing capacity of the foundation soil, and maximum tensile stress that will be induced in the reinforcement strips. These results are for a particular case. Obviously if other values for the parameters were used e.g. a smaller value of $\mu$, the results would be different.

4.9 Conclusion

The laboratory tests conducted on reinforced earth model walls with adjacent unreinforced backfill, which simulate reinforced earth walls on a rigid foundation, indicate that even before backfill thrust comes into play the pressure distribution within the reinforced zone is nonlinear. When the backfill thrust comes into play, the vertical pressure increases throughout, with maximum increase near the back of the wall and this effect lessens with distance away from the back. If the influence of friction along the back is separated, the effect of lateral thrust alone is to decrease the pressure at the back. The reduction in pressure at this rear end conforms only qualitatively to trapezoid theory. However maximum increase
in pressure does not occur at the facing as predicted by that theory. This is not surprising because reinforced earth obviously does not act as a perfectly rigid body.

An arching phenomenon appears to exist in reinforced earth walls where the vertical pressure in the middle of the wall is distributed to the sides, resulting in stress concentration particularly at the front. Vertical pressure at the middle is much less than that predicted by theories. It can be concluded therefore that the vertical pressure distribution in reinforced earth walls is influenced by at least two factors: backfill thrust and arching in the direction of length \( L \). The backfill thrust (lateral thrust) has an effect of reducing the overturning resistance about the toe of the reinforced earth model wall. This effect is qualitatively similar to that predicted by the trapezoid theory.

The total vertical force based on observed stresses is found to be somewhat less than that based on theoretical static equilibrium consideration i.e. less (an average of 7\%) than the quantity \( yhL \). Obviously, this must be due to arching in the direction of width \( B \). It is important to recognise this because of the significance of the arching phenomenon in understanding the observed behaviour.

The accuracy of the measured results was validated by comparing the total downward force based on observed
vertical pressures before building up the backfill behind the reinforced block (fourth series) and after building the backfill to its maximum height (300mm), where the difference should be equal to the vertical friction force. It was found to be exactly equal to that quantity.

Basic analytical consideration using the trapezoid theory show that, as expected, friction at the back of the wall increases the total vertical force. The particular analysis performed showed that, when frictional force was mobilized, the maximum stress does not necessarily occur at the facing.
CHAPTER 5
THE EFFECT OF REINFORCEMENT STRIP DIMENSIONS ON THE VERTICAL PRESSURE DISTRIBUTION IN REINFORCED EARTH WALLS

5.1 Introduction

In the design of reinforced earth walls, the determination of vertical stress is required. Besides the use of the vertical normal stress to calculate the horizontal thrust on the wall face, the vertical normal stress distribution is required for the determination of the frictional resistance of the reinforcement strips. The general equation for the frictional resistance of a strip is:

\[ F_R = 2wL_a \cdot \sigma_v \cdot f \]  \hspace{1cm} (5.1)

where \( w \) is width of strip
\( L_a \) is effective length of the strip
\( f \) is friction coefficient of soil-reinforcement
\( \sigma_v \) is vertical normal stress

Considering a unit length of a reinforcement strip, the width of the reinforcement strip determines the area on which the vertical stress acts and where the shearing stress develops. It also controls the axial stiffness of the reinforcement strip. The effect of these factors on the behaviour of reinforced earth as a composite is unknown. Thus, the focus of this investigation was the determination
of the effect of the strip width on the vertical normal stress.

5.2 Investigative Procedure

Several series of experiments were conducted. In each series, several reinforced earth model walls were built, each instrumented by pressure transducers placed at the base of the reinforced earth model wall to measure the vertical normal stress or pressure due to self-weight. The difference between the series of experiments was in the geometry of the reinforcement strips employed in the construction of model walls. A total of seven series of experiments were conducted. Of these, five series were conducted to determine the effect of the strip width on the vertical pressure:

Series A reinforced earth model walls with strips of 300mm long and 10mm wide
Series B reinforced earth model walls with strips of 300mm long and 13.5mm wide
Series C reinforced earth model walls with strips of 300mm long and 20mm wide. An additional test was made in this series to find the effect of external load (strip load) on the vertical pressure distribution.
Series D reinforced earth model walls with strips of 300mm long and 25mm wide
Series E reinforced earth model walls with strips of
300mm long and 32mm wide

The remaining two series of experiments were conducted as controls:

Series F  unreinforced model walls

Series G  reinforced earth model walls with strips of 370mm long and 10mm wide

Except for Series A, the walls were constructed to a safe height of 300mm. Walls in Series A failed before they reached this height. The vertical and horizontal spacings of the reinforcement strip were kept constant at 50mm and 200mm respectively. The number of reinforcement strips were kept constant at 24 strips, where four reinforcement strips were placed in each lift of soil. The material used as the reinforcement was 0.20mm thick shim steel. In Series F the walls were built without any reinforcement. The backfill was contained by a rigid steel plate placed at the front of the wall. This series of tests were conducted to determine the vertical pressure distribution within the wall when there was no reinforcement.

5.2.1 Model box

The walls were built within a glass box 800mm wide, 1000mm long and 600mm deep as used in the experiments described in Chapter 4. The glass sides, apart from minimizing friction, allowed visual observation of model
behaviour during construction. Two accessories were used in model wall construction: a motorized hopper that ran along the length of the box to deposit the sand fill and a frame to support the facing elements. The frame consisted of a square hollow section sliding up and down on two steel rods. The section provided temporary support for a facing element while the adjacent 50mm layer of sand was being placed.

5.2.2 Reinforcement

Except for Series F, the reinforcement strips were cut out of 0.20mm thick shim steel. The modulus of elasticity of the shim steel was found to be 70,000MPa and the tensile strength was 580MPa. These properties were obtained through tensile test, Appendix G. The strength of the reinforcement ensured that the walls would not fail from tensile failure of the strips.

5.2.3 Fill

Dried local beach sand as described in section 4.2.3 was used as construction material. The sand was rained through a hopper which travelled along the length of the wall and remained at the height of 600mm above the base of the wall. The raining process resulted in an even placement of sand (see Figure 5.1). No compaction of the sand was attempted. The raining process gave a fill with the unit weight of 15.66kN/m$^3$, relative density of 61% and void ratio of 0.63.
The in-situ density of the sand was determined by placing a 250mm square and 300mm deep aluminium container within the model box and raining sand from the hopper into the container. The thickness of the container walls was very small at 0.30mm, this ensured that the sand fell straight into the container.

5.2.4 Facing elements

The facing elements extended over the entire width of the wall. The height and thickness of each element was 50mm and 1.46mm respectively. These facing elements have been described in detail in Section 4.2.4.

5.2.5 Instrumentation

Nine pressure transducers (PT1 to PT9) were placed at the centreline of the base of the wall where the readings were less affected by the glass sides of the box, Figure 5.2. The box itself was secured to a rigid timber table, serving as a base on which the model walls were built. The spacing of the pressure transducers was 70mm. The distance of the pressure transducers from the facing is tabulated in Table 5.1. The measuring surface of the transducers was 45mm from the base of the wall.

The transducers were TYCO AB with a measuring surface of 19mm. The transducers had been calibrated with water where the procedure was as described in section 4.2.5. The calibration charts are shown in Appendix H. The transducers were supplied with 5V direct current and the output
Figure 5.1  Reinforcement strips on an even bed of sand

Figure 5.2  Placement of pressure transducers
<table>
<thead>
<tr>
<th>Pressure transducer or gauge point</th>
<th>Distance from facing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>35</td>
</tr>
<tr>
<td>PT2</td>
<td>105</td>
</tr>
<tr>
<td>PT3</td>
<td>175</td>
</tr>
<tr>
<td>PT4</td>
<td>245</td>
</tr>
<tr>
<td>PT5</td>
<td>315</td>
</tr>
<tr>
<td>PT6</td>
<td>385</td>
</tr>
<tr>
<td>PT7</td>
<td>455</td>
</tr>
<tr>
<td>PT8</td>
<td>525</td>
</tr>
<tr>
<td>PT9</td>
<td>595</td>
</tr>
</tbody>
</table>
voltages which were proportional to the physical changes being measured were read by a voltmeter.

5.2.6 Test procedure

The procedures involved in all the series except Series F consisted of the following steps:

1) placing the pressure transducers;
2) aligning the first facing element and temporarily holding it vertically using the supporting frame;
3) taking initial reading of the pressure transducers;
4) raining 25-mm high layer of sand into the box;
5) taking a reading of the pressure transducers;
6) placing four reinforcement strips on the sand bed at 200mm distance apart (centre to centre);
7) attaching the strips to the facing element;
8) raining another 25mm high layer of sand;
9) taking reading of the pressure transducers;
10) placing another facing element on top of the previously laid one and sliding the supporting section to this element;
11) repeating procedures (4) to (10) until the wall reached a height of 300mm.

Even though each of the pressure readings were taken after the hollow section of the supporting frame was slid up and off the wall, the base of the frame formed a 15mm berm to the wall. In order to determine the effect of the hollow section on the vertical pressure, comparisons of the
pressure readings when the section was in place and when it was slid off were made. These additional readings were taken in tests B1 and B2 in Series B. The pressure readings were used to find the total vertical force from the area of the pressure diagram. The result of these supplementary tests are presented in Appendix L. It can be seen that when the 100mm reinforced earth model wall was supported, 16% of the vertical force was transferred to the support system and for the wall of 300mm high, only 4% of the total vertical force was transferred to the support system.

In Series F, the walls were built without any reinforcement. After the pressure transducers were put in place, the model wall was constructed by first placing a rigid plate in front of the box. The plate was held in place at the top by two flanges which rested on the top edges of the model box and at the bottom it was supported by the berm. Additional support was given by the hollow section positioned at 360mm from the base of the wall. Additionally, six of the facing elements were connected to each other and they were taped to the rigid plate on the side which was to be in contact with the sand fill. This was to ensure that surface friction at this boundary was the same as in other walls used for the investigation. After the initial readings of the pressure transducers were taken, the box was ready to be filled with sand. Sand was dropped from the hopper and readings of the pressure transducers were then taken for every 25mm increment of sand. The wall was constructed to a depth of 300mm.
It should be noted that the boundary conditions for this type of wall are not the same as for the reinforced earth model walls since there is no deflection at the front of the wall normally allowed by the flexible facing.

5.3 Experimental Study

5.3.1 Series A

Two reinforced earth model wall tests were conducted in this series. There were tests A1 and A2. The shim steel reinforcement strips used in the construction of the walls were 0.2mm thick, 300mm long and 10mm wide. The total mass of reinforcement was 126g.

Visual observation showed that as the wall was being constructed there was forward deflection of the facing with zero movement at the toe of the wall and maximum deflection around the mid-way of its height. The deflection increased with the increase in fill height. The movement of the wall mobilized the frictional resistance of the reinforcement strips and the wall remained stable.

Wall A1 failed at a height of 240mm. the mode of failure was rotation at the toe of the wall with a forward movement of the fill near the facing, Figure 5.3. Wall A2 reached the height of 250mm. At this height the maximum deflection was 20mm at the height of 150mm from the base. An attempt to increase the height further resulted in failure of the wall. The failure mode was the same as wall A1. Both
Figure 5.3  Failure of reinforced earth model wall

(note: value on tape does not indicate actual height; e.g. 230mm on tape, show 130mm height above base)
walls failed from adherence failure of the reinforcement.

The pressure readings under various fill heights for both tests are tabulated in Tables I.1 to I.3 in Appendix I. The average readings from these two tests were taken as representative for Series A. The vertical pressure distribution in the reinforced earth walls in series A is illustrated by Figure 5.4.

It can be seen that the vertical pressure distribution is not uniform within the reinforced zone and also outside the reinforced zone. In order to describe the pressure distribution under various fill heights, the pressure at each gauged point was divided by the pressure at PT1, giving the pressure ratio at each gauged point. This was done for all fill heights, resulting in the dimensionless values of pressure ratio tabulated in Table 5.2.

The trend of the pressure distribution at any fill height was similar with highest pressure being at PT1 (near the facing). The pressure decreased from the facing to PT4 and then increased again from PT4 to PT5. The pressures at PT5 and PT6 were similar. The pressure decreased again from PT6 to PT9.

There were two regions in the reinforced earth model walls: the reinforced zone and the backfill. The pressure beneath the reinforced zone was measured by PT1, PT2, PT3 and PT4 and the pressure beneath the backfill was measured by PT5, PT6, PT7, PT8 and PT9. The experimental readings
Figure 5.4  Pressure distribution diagram for Series A

Table 5.2  Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill Height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1</td>
<td>0.50</td>
<td>0.46</td>
<td>0.36</td>
<td>0.53</td>
<td>0.53</td>
<td>0.32</td>
<td>0.24</td>
<td>0.21</td>
</tr>
<tr>
<td>150</td>
<td>1</td>
<td>0.52</td>
<td>0.44</td>
<td>0.40</td>
<td>0.57</td>
<td>0.57</td>
<td>0.47</td>
<td>0.41</td>
<td>0.38</td>
</tr>
<tr>
<td>200</td>
<td>1</td>
<td>0.48</td>
<td>0.37</td>
<td>0.33</td>
<td>0.52</td>
<td>0.52</td>
<td>0.48</td>
<td>0.40</td>
<td>0.35</td>
</tr>
<tr>
<td>250</td>
<td>1</td>
<td>0.54</td>
<td>0.32</td>
<td>0.26</td>
<td>0.50</td>
<td>0.51</td>
<td>0.44</td>
<td>0.33</td>
<td>0.28</td>
</tr>
<tr>
<td>300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Not available</td>
</tr>
</tbody>
</table>
indicate a lowering of pressure in the middle region of the reinforced zone. Within the backfill the pressure decreases with increasing distance from the reinforced zone. It should be noted that the distance from PT9 to the end wall of the model box is 335mm which is considered substantial enough to eliminate any end wall frictional effect.

The tests show that the vertical pressure distribution within the wall is not uniform. The distribution within the reinforced zone is nonlinear however the distribution within the backfill appears to be close to a linear one.

The non-linearity of the vertical pressure distribution prompted a check made on the pressure transducers to determine whether there was a mechanical or electrical fault. The check was done by placing a known weight on the centre of the measuring surface of one of the pressure transducers. The difference in voltage reading was divided by the calibration factor of the pressure transducer to obtain the measured pressure. The procedure was repeated on other transducers using the same weight. Theoretically, the measured pressure should be the same for all transducers. The result of the test is presented in Appendix J. It can be seen that there was only a small difference between the readings of the pressure transducers thus it was concluded that the pressure readings in the tests with reinforced earth model walls were accurate.
5.3.2 Series B

Two reinforced earth model wall tests were conducted in this series. They were tests B1 and B2. The shim steel reinforcement strips used in the construction of the walls were 0.20mm thick, 300mm long and 13.5mm wide. The total mass of the reinforcement was 171g.

Both walls were built to a safe height of 300mm. During and after construction, there was a very small deflection of the wall facing. At the final height, the maximum deflection for both wall B1 and wall B2 was 2mm at the height of 100mm from the base.

The pressure readings under various fill heights for both tests are tabulated in Tables 1.4 to I.5 in Appendix I. The average readings from these two tests are taken as representative of Series B and the vertical pressure distribution is illustrated by Figure 5.5.

In order to describe the vertical pressure distribution under the various fill heights, the pressure reading at each gauged point was divided by the reading at PT1. The pressure ratios are tabulated in Table 5.3.

The vertical pressure distributions at all fill heights are similar with the arching between PT1 and PT5 more pronounced with the increase in fill height. This is shown by the decrease in pressure ratio at PT3 and PT4 and the increase in pressure ratio at PT2. The pressure ratio at PT5 to PT9 remained unchanged with fill height. The
Figure 5.5 Pressure distribution diagram for Series B

Table 5.3 Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Series Height (mm)</th>
<th>Fill</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1</td>
<td>0.66</td>
<td>0.56</td>
<td>0.56</td>
<td>0.67</td>
<td>0.63</td>
<td>0.49</td>
<td>0.46</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>1</td>
<td>0.70</td>
<td>0.50</td>
<td>0.50</td>
<td>0.69</td>
<td>0.61</td>
<td>0.50</td>
<td>0.46</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>1</td>
<td>0.74</td>
<td>0.47</td>
<td>0.47</td>
<td>0.70</td>
<td>0.60</td>
<td>0.54</td>
<td>0.46</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>1</td>
<td>0.80</td>
<td>0.46</td>
<td>0.44</td>
<td>0.69</td>
<td>0.59</td>
<td>0.56</td>
<td>0.46</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>1</td>
<td>0.87</td>
<td>0.49</td>
<td>0.43</td>
<td>0.69</td>
<td>0.61</td>
<td>0.60</td>
<td>0.48</td>
<td>0.34</td>
<td></td>
</tr>
</tbody>
</table>
lowest pressure ratio was at PT9.

It can be concluded that the vertical pressure distribution within the reinforced earth model wall is not uniform. Within the reinforced zone there is lowering of pressure in the middle region. The pressure within the backfill decreases with increasing distance from the back of the reinforced zone. The tests show that within the reinforced zone the vertical pressure distribution is nonlinear and it is close to linear in the backfill.

5.3.3 Series C

Two reinforced earth model tests were conducted in this series. They were tests C1 and C2. The shim steel reinforcement strips used in the construction were 0.20mm thick, 300mm long and 20mm wide. Total mass of reinforcement was 253g.

The walls were built to a safe height of 300mm. There was no visible deflection of the wall facings. Beside taking the pressure readings under various overburden heights, additional testing was performed where the walls were loaded with strip load when they reached the height of 300mm. The purpose of this test was to determine the effect of external load on the vertical pressure distribution in reinforced earth model walls. Jackson and Jones, (1988) have shown that the vertical pressure distribution, due to strip load applied near the facing, is a complex subject. This additional test was performed as a point of interest, and it shall not form a major concern of this thesis which
is solely on the behaviour of reinforced earth walls under self-weight.

The procedure of the additional testing included:

1) placing a rectangular steel section at a distance \( d_x \) from the facing, on top and across the 300mm high reinforced earth model wall. The steel section acted as a loading plate and it extended through the entire width of the model box. The section was 65mm wide, 20mm thick, 780mm long, with a 10mm clearance at each of the side walls of the model box.

2) taking readings of the pressure transducers;
3) placing weight at the centroid of the steel section top surface and taking readings of the pressure transducers;
4) placing additional weight at the centroid of the steel section top surface and taking readings of the transducers;
5) repeating procedure (4) until the wall fails.

In deciding on the distance \( d_x \) between the wall face and the point of application of the strip load, the two following conflicting factors have to be considered:

a) The placement of the load should result in a significant effect on the soil-reinforcement interaction. Since Kennedy et al. (1980) have found that no significant forces are generated in the reinforcement strips when the surcharge strip loads are applied outside the top extremity of the potential failure wedge, the loading block should be
positioned close to the wall facing.

b) Since the lateral thrust on the facing due to strip is dependent on its distance from the wall face, the loading block should not be too close to the facing so that a large additional load could be sustained by the reinforced earth wall before failure.

When considering a), for convenience the potential failure plane was assumed to be Rankine wedge, Figure 5.6. It should be appreciated however, that although Lee et al. (1973) observed the Rankine failure plane in their reinforced earth model wall, most observed potential failure surfaces in both model and full-scale walls are curvilinear (Yong and McLarin, 1985; Arenicz and Chowdhury, 1987). In adopting a linear failure plane, the reinforced zone was assumed as a homogenous material which was supported by the fact that the thickness of the reinforcement strip was small; hence the reinforcement strip would not change the Rankine failure plane as in a layered soil. The internal friction angle $\phi$ of the sand used in the analysis was 35.4°. As can be seen in the figure, the failure plane cuts the top surface of the wall at 155mm from the facing.

Balancing requirements a) and b), with requirement b) having more weight, the distance 'd' was chosen to be 117.5mm. This means that the entire width of the loading plate is within the potential failure wedge.
Figure 5.6 Rankine failure wedge
The pressure readings for both tests are tabulated in Tables I.7 to I.9 in Appendix I. The average readings from this tests were taken as representative of Series C. The vertical pressure distribution under various fill heights is illustrated in Figure 5.7. The pressure ratios are tabulated in Table 5.4.

The trend in pressure is similar for all fill heights, with lowering of pressure in the middle region of the reinforced zone becoming more pronounced with the increase in fill height. It can be seen that the maximum pressure was at PT1 and the minimum pressure was at PT9. These tests show that the vertical pressure distribution in the wall is not uniform and nonlinear both in the reinforced zone and the backfill.

The results for the additional tests are tabulated in Tables K.1 to K.3 in Appendix K, where the value of the surcharges applied are also given. For test C2 (Table 2), there was a decrease in pressure at PT1 under Load Plate (LP) and Surcharge Load 1 (SL1). For both tests C1 and C2 (Tables 1 and 2) there was no change in pressure at PT5 under SL2 and SL3. There was no change in pressure at PT8 and PT9 under SL2, SL3 SL4 and SL5 for both tests. The average results from both tests (Table 3) were then taken to assess the increase in vertical pressure due to the surcharge strip load, as illustrated in Figure 5.8. The negative distance indicates: distance towards the facing from the centre of the load plate while the positive
Figure 5.7  Vertical pressure distribution diagram for Series C

Table 5.4  Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>Series</th>
<th>Pressure Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>100</td>
<td>C</td>
<td>1</td>
</tr>
<tr>
<td>150</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>250</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>300</td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>
Figure 5.8  Change in vertical pressure due to strip load applied at top surface (H = 300mm)
distance indicates distance towards the back of the wall from the centre of the load plate.

It can be seen from the figure that under the load of LP, there was a decrease in vertical pressure near the facing with a small increase in vertical pressure at points near the load plate. There was no change in pressure at points far from the load plate. Under larger strip load (SL4 and SL5), there was a significant increase in pressure near the facing however the highest increase in pressure was at 45mm from the load plate and towards the facing. At the same distance but towards the back of the wall, the increase in vertical pressure was much less. The increase in pressure at this point is similar to that of the point under the loading plate. From the loading plate to the back of the wall, the increase in vertical pressure decreases with distance.

Theoretically, assuming no eccentricity of the external load, the increase in the vertical pressure along the affected area should be uniform. The area affected by the strip load can be found by assuming some form of vertical pressure diffusion. In this analysis a vertical pressure diffusion of 2:1 was assumed. This diffusion ratio was chosen after Kennedy et al. (1980) who has shown that the theoretical and experimental results agree very well when this ratio was assumed. The calculation of the vertical pressure due to the strip load is shown in Figure 5.9a. As it can be seen from Figure 5.9b, the affected area
The vertical pressure or stress at a depth \( h \), due to strip load is:

\[
\Delta \sigma_v = q \frac{B_x}{(B_x + h)} \quad \text{when } h \leq 2d_x
\]
or

\[
= q \frac{B_x}{(B_x + d_x + h/2)} \quad \text{when } h > 2d_x
\]

(after Kennedy et al., 1980)

(a)

Figure 5.9 (a) Theoretical vertical pressure due to strip load

(b) Author's model wall configuration
at the level of the pressure transducers' measuring surface is 310\text{mm} while, in the tests, the gauged area was 560\text{mm}. The vertical pressure at this level due to the surcharge strip load can be calculated as:

\[ \sigma_v = q \left( \frac{B_x}{B_x + h} \right) \]
\[ = q \left( \frac{65}{65 + 255} \right) \]
\[ = 0.203q \] \hspace{1cm} (5.2)

The theoretical vertical pressure under the various applied surcharge loads is tabulated in Table 5.5.

Test results show that the pressure due to the surcharge strip load is not uniformly distributed as predicted by theory and that, under larger loads SL4 and SL5, the affected area is more than the theoretical values. In the test the decrease in pressure under initial applied load shows a redistribution of the vertical pressure. The test also shows that more vertical pressure was distributed towards the facing. This behaviour may have some connection with the observed pressure distribution due to overburden where maximum pressure was near the facing.

5.3.4 Series D

Two reinforced earth model wall tests were conducted in this series. They were tests D1 and D2. The shim steel reinforcement strips were 0.20\text{mm} thick, 300\text{mm} long and 25\text{mm}
Table 5.5  Theoretical Vertical Pressure at Measured Level due to Strip Load

<table>
<thead>
<tr>
<th>LOAD</th>
<th>Surcharge ( q ) (kPa)</th>
<th>Theoretical pressure ( 0.203q ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LP</td>
<td>1.500</td>
<td>0.304</td>
</tr>
<tr>
<td>SL1</td>
<td>1.998</td>
<td>0.406</td>
</tr>
<tr>
<td>SL2</td>
<td>2.496</td>
<td>0.507</td>
</tr>
<tr>
<td>SL3</td>
<td>2.994</td>
<td>0.608</td>
</tr>
<tr>
<td>SL4</td>
<td>4.989</td>
<td>1.013</td>
</tr>
<tr>
<td>SL5</td>
<td>6.984</td>
<td>1.418</td>
</tr>
</tbody>
</table>
wide. The total mass of reinforcement was 316g.

At the final height of 300mm, there was no wall deflection of both wall facings. The results of both tests are tabulated in Tables I.10 to I.12 in Appendix I. The average results from the tests are illustrated in Figure 5.10. The pressures were made dimensionless and the pressure ratios are tabulated in Table 5.6.

Under a fill height of 100mm, the vertical pressure distribution is fairly uniform although the pressure at PT5, PT6 and PT7 was slightly higher than at PT1. Under fill height 150mm the pressure at PT5 and PT6 was higher than at PT1. Under all fill heights the pressure decreased from PT5 to PT9 with the pressure at PT9 being the lowest along the gauged length. The lowering of pressure in the middle of the reinforced zone is clearly visible under fill heights 150mm, 200mm, 250mm and 300mm.

The tests show that the vertical pressure distribution is not uniform within the wall. In general, it is nonlinear within both the reinforced zone and the backfill. Under lower fill heights however, the vertical pressure distribution within the backfill is close to a linear one.

5.3.5 Series E

Two reinforced earth model wall tests were conducted in this series. They were tests D1 and D2. The shim steel reinforcement strips were 0.20mm thick, 300mm long and 32mm
Figure 5.10 Vertical pressure distribution diagram for Series D

Table 5.6 Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill Height (mm)</th>
<th>PT1 Pressure Ratio</th>
<th>PT2 Pressure Ratio</th>
<th>PT3 Pressure Ratio</th>
<th>PT4 Pressure Ratio</th>
<th>PT5 Pressure Ratio</th>
<th>PT6 Pressure Ratio</th>
<th>PT7 Pressure Ratio</th>
<th>PT8 Pressure Ratio</th>
<th>PT9 Pressure Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 D</td>
<td>0.76</td>
<td>0.96</td>
<td>1.05</td>
<td>1.35</td>
<td>1.28</td>
<td>1.03</td>
<td>0.76</td>
<td>0.59</td>
<td></td>
</tr>
<tr>
<td>150 D</td>
<td>0.69</td>
<td>0.62</td>
<td>0.74</td>
<td>1.13</td>
<td>1.02</td>
<td>0.84</td>
<td>0.64</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>200 D</td>
<td>0.74</td>
<td>0.53</td>
<td>0.59</td>
<td>0.98</td>
<td>0.90</td>
<td>0.76</td>
<td>0.58</td>
<td>0.41</td>
<td></td>
</tr>
<tr>
<td>250 D</td>
<td>0.77</td>
<td>0.49</td>
<td>0.51</td>
<td>0.80</td>
<td>0.85</td>
<td>0.72</td>
<td>0.56</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>300 D</td>
<td>0.82</td>
<td>0.47</td>
<td>0.45</td>
<td>0.82</td>
<td>0.81</td>
<td>0.68</td>
<td>0.53</td>
<td>0.38</td>
<td></td>
</tr>
</tbody>
</table>
wide. The total mass of the reinforcement was 405g.

At the final height of 300mm there was no visible deflection of the wall facings. The pressure readings under various fill heights are tabulated in Tables I.13 to I.15 in Appendix I. The average readings from both tests were taken as representative for this series. The resulting vertical pressure distribution is illustrated in Figure 5.11. The readings were made dimensionless and the pressure ratios within the gauged length under various fill height are tabulated in Table 5.7.

Under a fill height of 100mm, the vertical pressure is fairly uniform. Under fill heights 150mm to 300mm the lowering of pressure within the middle of the reinforced zone was quite obvious. Under these fill heights the pressure from PT5 to PT7 were increased. The pressure then decreased from PT7 to PT9. Under all the fill heights, the lowest pressure was at PT9.

The tests show that the vertical pressure distribution within the wall is not uniform and it is nonlinear both in the reinforced zone and the backfill zone.

5.3.6 Series F

Two unreinforced model wall tests were conducted in this series. They were tests F1 and F2. The procedures involved in these tests have been described before. The purpose of these tests was to check whether the nonlinearity of the vertical pressure distribution in
Figure 5.11 Vertical pressure distribution diagram for Series E

Table 5.7 Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>Press. Ratio</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>E 100</td>
<td></td>
<td>1</td>
<td>0.84</td>
<td>0.73</td>
<td>0.76</td>
<td>0.96</td>
<td>0.92</td>
<td>0.87</td>
<td>0.73</td>
<td>0.64</td>
</tr>
<tr>
<td>150</td>
<td></td>
<td>1</td>
<td>0.65</td>
<td>0.54</td>
<td>0.58</td>
<td>0.78</td>
<td>0.79</td>
<td>0.81</td>
<td>0.59</td>
<td>0.52</td>
</tr>
<tr>
<td>200</td>
<td></td>
<td>1</td>
<td>0.61</td>
<td>0.44</td>
<td>0.48</td>
<td>0.69</td>
<td>0.74</td>
<td>0.82</td>
<td>0.56</td>
<td>0.48</td>
</tr>
<tr>
<td>250</td>
<td></td>
<td>1</td>
<td>0.63</td>
<td>0.41</td>
<td>0.41</td>
<td>0.62</td>
<td>0.69</td>
<td>0.76</td>
<td>0.51</td>
<td>0.42</td>
</tr>
<tr>
<td>300</td>
<td></td>
<td>1</td>
<td>0.66</td>
<td>0.41</td>
<td>0.39</td>
<td>0.62</td>
<td>0.69</td>
<td>0.78</td>
<td>0.51</td>
<td>0.41</td>
</tr>
</tbody>
</table>
reinforced earth model walls were due to the presence of reinforcement.

Since the facing was rigid and supported there was no deflection of the walls. The pressure readings from these tests are tabulated in Tables I.16 to I.18 in Appendix I. The average readings from the two tests were taken as representative of this series. The pressure distribution under various fill heights is illustrated in Figure 5.12 and the pressure ratios within the gauged length are tabulated in Table 5.8.

Under all the fill heights the vertical pressure distribution is not uniform and the shape of the pressure distribution is similar. Highest pressure was recorded at PT1 and lowest pressure was at PT3. The pressure decreased, from PT1 to PT3, and increased from PT3 to PT6, and decreased again from PT6 to PT9.

The tests show that the pressure within a unreinforced model wall is nonuniform and nonlinear. It shows that the peculiar nonlinear pressure distribution found in reinforced earth model walls is not unique to reinforced earth. However in the region corresponding to the unreinforced zone (in the reinforced earth model wall test) the pressure distribution was found to be fairly uniform.

When the vertical pressure distribution in the unreinforced model wall (Table 5.8) was compared with the vertical distribution within a reinforced earth model wall
Figure 5.12 Vertical pressure distribution diagram for Series F

Table 5.8 Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.79</td>
<td>0.43</td>
<td>0.65</td>
<td>0.70</td>
<td>0.82</td>
<td>0.56</td>
<td>0.56</td>
<td>0.58</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>0.72</td>
<td>0.42</td>
<td>0.57</td>
<td>0.66</td>
<td>0.75</td>
<td>0.62</td>
<td>0.60</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>0.67</td>
<td>0.37</td>
<td>0.48</td>
<td>0.65</td>
<td>0.71</td>
<td>0.62</td>
<td>0.61</td>
<td>0.51</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>0.65</td>
<td>0.35</td>
<td>0.44</td>
<td>0.61</td>
<td>0.67</td>
<td>0.60</td>
<td>0.59</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0.70</td>
<td>0.36</td>
<td>0.43</td>
<td>0.63</td>
<td>0.68</td>
<td>0.63</td>
<td>0.62</td>
<td>0.48</td>
<td></td>
</tr>
</tbody>
</table>
(Table 5.7) it can be seen that there was a large difference in the vertical pressure distribution as expected.

5.3.7 Series G

Two reinforced earth model wall tests were constructed in this series. They were tests G1 and G2. The shim steel reinforcement strips were 0.20mm thick, 370mm long and 10mm wide. The total mass of reinforcement was 156g. The purpose of this series was to determine whether the vertical pressure distribution was affected by the length of the reinforcement. The result from this series will be compared with the result from Series A.

At the final height of 300mm there was only a small deflection of the wall facings. The maximum deflection of the facing was 2mm at 100mm from the base for both walls. The pressure readings under various heights are tabulated in Tables I.19 to I.21 in Appendix I. The average readings from both tests are taken as representative of the series. The pressure distributions are illustrated in Figure 5.13 and the pressure ratios within the gauged length are tabulated in Table 5.9.

The pressure distributions under all fill heights have a similar trend. The pressure was highest near the facing and, except for fill heights 100mm and 150mm, the lowest pressure was at PT4. Under fill heights 100mm and 150mm the lowest pressure was at PT9. The pressure decreased from PT1
Figure 5.13 Vertical pressure distribution diagram for Series G

Table 5.9 Pressure Ratios Along The Gauged Length

<table>
<thead>
<tr>
<th>Fill Series height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 G</td>
<td>1</td>
<td>0.73</td>
<td>0.61</td>
<td>0.49</td>
<td>0.77</td>
<td>0.87</td>
<td>0.65</td>
<td>0.54</td>
<td>0.46</td>
</tr>
<tr>
<td>150 G</td>
<td>1</td>
<td>0.67</td>
<td>0.55</td>
<td>0.48</td>
<td>0.55</td>
<td>0.78</td>
<td>0.49</td>
<td>0.43</td>
<td>0.35</td>
</tr>
<tr>
<td>200 G</td>
<td>1</td>
<td>0.78</td>
<td>0.54</td>
<td>0.40</td>
<td>0.65</td>
<td>0.76</td>
<td>0.69</td>
<td>0.53</td>
<td>0.49</td>
</tr>
<tr>
<td>250 G</td>
<td>1</td>
<td>0.78</td>
<td>0.51</td>
<td>0.39</td>
<td>0.66</td>
<td>0.76</td>
<td>0.72</td>
<td>0.55</td>
<td>0.48</td>
</tr>
<tr>
<td>300 G</td>
<td>1</td>
<td>0.81</td>
<td>0.50</td>
<td>0.37</td>
<td>0.64</td>
<td>0.75</td>
<td>0.74</td>
<td>0.56</td>
<td>0.48</td>
</tr>
</tbody>
</table>
to PT4, increased from PT4 to PT6, and decreased again from PT6 to PT9.

The tests show that the vertical pressure distribution within the walls was not uniform. The distribution was nonlinear within the reinforced zone and it was linear in the backfill.

5.4 Comparison Between Series

5.4.1 The effect of reinforcement strip length on the vertical pressure distribution

The reinforcement strip length reflects the size of the reinforced zone, often defined as the ratio of the reinforcement length over the height of the reinforced earth wall. In order to determine whether the vertical pressure distribution is affected by the length of reinforcement strip, a comparison was made of the pressure readings in walls having the same heights, number and widths of strip, but a different length of reinforcement. Comparison was made between Series A and Series G.

In Series A the length of the reinforcement strip was 300mm, making the ratio of reinforcement length over wall height to be 1.0 while in Series G the length of the reinforcement strip was 370mm, making the ratio to be 1.23. In Series A, pressure transducers PT1 to PT4 were within the reinforced zone and pressure transducers PT5 to PT9 were within the unreinforced backfill zone. In Series G the
By comparing the pressure diagrams shown in Figure 5.4 and Figure 5.13, it can be seen that the shape of the pressure curves within the reinforced zone are similar in both series. Likewise the shape of the curves within the unreinforced zone are also similar. For both series the trend of pressure is similar from PT1 to PT4. In Series A the pressure increased from PT4 to PT5, the latter being located near the back of the reinforced zone. In Series G, the pressure increased from PT4 through PT5 to PT6, the latter being located near the back of the reinforced zone. The pressure then decreased from PT6 to PT9.

The tests show that the shape of the pressure curves in the reinforced zone and unreinforced zone are similar for both series. The pressure curve for both series are characterized by the almost identical pressure at PT2 and PT5 for Series A and almost identical pressure at PT2 and PT6 for Series G. It can be concluded that within the two reinforced zones of different size (governed by the length of reinforcement, L) the vertical pressure distribution was similar. In Chapter 7 it will be shown that numerical analysis by Shen et al. (1976) gave a similar result. The overall conclusion that can be made from these series is that the length of reinforcement (L) influenced the magnitude of vertical pressure at any particular point and
that the vertical pressure distribution within a reinforced zone is similar no matter what the length of the reinforcement.

5.4.2 The effect of reinforcement strip width on vertical pressure distribution in reinforced earth model walls.

In order to determine the effect of reinforcement width on the vertical pressure distributions in reinforced earth model walls, the pressure distribution from Series B, C, D and E were compared. The comparison was made of the vertical pressure distributions under the maximum fill height of 300mm since obviously, the pressure readings under this fill were the most reliable. Series A could not be included since the walls were built only up to 250mm. The pressure distribution from Series F was included as control.

The vertical pressure distributions are illustrated in Figure 5.14. The pressure distribution for all the series are not uniform, both in the reinforced zone and the backfill zone and it can be seen that the pressure distribution between the series of reinforced earth model walls are different. At a particular distance from the facing, the vertical pressure is different between the series and there is also a difference in the trend (increase or decrease) of the pressure. An example of this difference in trend is illustrated by Series B (strip width of 13.5mm) and Series E (strip width of 32mm). From gauged point PT1 to PT4 the trend is similar (pressure
Figure 5.14 Comparison of vertical pressure distribution between series of tests (effect of width of strips for Series B = 13.5mm to Series E = 32mm)
The pressure for Series E increases from PT4 to PT7 and decreases from PT7 to PT9 while for Series B the pressure increase from PT4 to PT5 and decreases from PT5 to PT9. The difference in the trend gave the difference in the vertical pressure distribution for each series. Another example to show the difference in the vertical pressure distribution can be seen by comparing the pressure diagram for Series B (Figure 5.5) with the pressure diagram for Series C (5.7). The trend in the vertical pressure from PT5 to PT6 for Series B is different for that of Series C, thus making the vertical pressure distribution for the two series different.

It can be concluded that the width of the reinforcement strip affect the trend in the vertical pressure and hence the vertical pressure distribution. For the same number and length of reinforcement, the width of the reinforcement affects the vertical pressure distribution within the reinforced zone and the backfill.

It can also be seen from Figure 5.14 that for all the series, the vertical pressure near the facing is higher than the calculated overburden pressure and the pressure within the rest of the wall is lower than the calculated overburden pressure. This shows that there was arching of vertical pressure.

As in the second stage of this analysis, the vertical forces acting in a reinforced earth model wall, at various heights of fill, was analysed for each series. The vertical
force is the area of a pressure diagram. The vertical force per linear metre of the wall under various heights of fill is tabulated in Table 5.10. The tabulated values were used to plot Figure 5.15. The figure shows the measured vertical force versus strip width for the various fill heights. The term 'measured' is used since the force is derived from the area of the pressure diagram, which was obtained through instrumentation of model walls.

Although data points were plotted for a width strip of zero, where the results from unreinforced model wall test were used, discussion at this stage will be on the results of the reinforced earth model wall tests which are illustrated by the bold curves in the figure. Since the walls in Series A were only built up to 250mm, there is no data point for the strip width of 10mm for fill height of 300mm. The shape of the curves is consistent under various fill heights which reflect that the accuracy of the tests performed was good.

It is clearly shown in Figure 5.15 that the relationship between the measured vertical force and width of the reinforcement strip is nonlinear. It can be seen that the measured vertical force was smallest for reinforced earth with the smallest strip width of 10mm. Not considering the curve for fill height of 300mm, the measured vertical force increases for strip width of 10mm to 20mm. The measured force decreased for strip width of 20mm to 25mm and increased again for strip width of 25mm to 30mm.
Table 5.10  Measured Vertical Force

<table>
<thead>
<tr>
<th>Series</th>
<th>Width of strip (mm)</th>
<th>Measured vertical force (kN/m) at fill height</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100mm</td>
<td>150mm</td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>10</td>
<td>0.2456</td>
</tr>
<tr>
<td>C</td>
<td>13.5</td>
<td>0.4330</td>
</tr>
<tr>
<td>D</td>
<td>20</td>
<td>0.4344</td>
</tr>
<tr>
<td>E</td>
<td>25</td>
<td>0.3940</td>
</tr>
<tr>
<td>F</td>
<td>32</td>
<td>0.4431</td>
</tr>
<tr>
<td>G</td>
<td>unreinforced</td>
<td>0.4308</td>
</tr>
</tbody>
</table>

Note: OF(d) is theoretical Overburden Force (kN/m) at fill height d. Note that the overburden depth above transducer is h = d - 45mm.

Figure 5.15  Measured vertical force over a length of 560mm versus strip width (theoretical force is shown by dotted line)
Under all the fill heights, the measured vertical force was lower than the theoretical overburden force, and it can be seen from the figure that the difference increases with increasing fill height. This was observed for all the series and it prompted the consideration whether:

1) the horizontal displacement of the facing resulted in the sand mass above PT1 having lower density thus lower vertical stress measured by PT1 and
2) whether the pressure transducers underregister.

The first consideration can be eliminated since the calculation shown that the amount of vertical displacement of the facing needed to bring about the difference in the vertical force had to be extremely large (200 mm), and this was not observed in the experiment. The second consideration leads to the conclusion that if underregistration of the pressure transducer did happen, it would have the same factor in all of the series and it would have no effect on the comparison of vertical forces between the series of tests.

It was found that the reinforcement strip width affect the measured vertical force in the reinforced earth model walls. In order to explain this behaviour or find the significance of it, we should look at the plan of the model test shown in Figure 5.16. It shows the position of the pressure transducers and the reinforcement strips. Each
pressure transducer measures the load above it and we shall assume that the measured volume is defined by a cylindrical funnel of the same diameter as the transducer's measuring surface, Figure 5.17.

The total vertical force in the walls is the overburden force and, since the measured volume is constant, the difference between the measured force and the overburden force could be attributed to arching of vertical pressure across the wall. This arching phenomenon has been describe by Guilloux et al. (1979). Figure 5.18 shows the arching phenomenon after Guilloux. It should be noted that the shape of the arch is hypothetical.

In order to substantiate the findings that the arching of the vertical pressure occurs across the reinforced earth model walls, the measured vertical force in an unreinforced model walls was plotted in Figure 5.15 under zero strip width. The values used were the experimental results corrected by a factor of 1.16 to account for the fact that the front of the walls were supported. The factor of 1.16 was obtained from the supplementary test (Appendix L), performed in order to find the effect of the support system on the vertical force where the condition in the unreinforced wall tests were represented by the supported 100mm high reinforced earth model wall. It can be seen that the measured vertical force was close to the overburden pressure (see also Table 5.11). The small difference was considered to be negligible and can be attributed to
Plan of model test

Figure 5.16 Plan of the model wall test

Cross-section of model test

Figure 5.17 Pressure transducer and its measured load
Arching across reinforcement strips

Figure 5.18  Arching phenomenon (after Guilloux et al., 1979)
experimental limitation.

In total, the experimental results obtained and their subsequent analysis, as presented above, indicate that:

1) In unreinforced model walls there is no arching across the walls.
2) In reinforced earth model walls, arching develops across the walls.
3) The arching in reinforced earth model walls is influenced by the strip width.

The analysis also shows high accuracy and reliability of the tests conducted.

5.5 The Influence of Strip Width on Arching of Vertical Pressure

The arching phenomenon is incorporated into the findings that the strip width affects the measured vertical force. The test results show that arching is affected by the strip width. In order to explain this statement, the curves for fill height of 250mm were taken to illustrate the distribution of the vertical pressure across the walls with strips of 10mm (lowest measured force) and 20mm strips (highest measured force). It can be seen that, for both types of wall, the measured vertical force was lower than the overburden force which means that the vertical pressure over the transducers was lower than over the adjacent strips. On the other hand, the pressure in the vicinity of
the 10mm reinforcement strips were higher than for the 20mm strips, Figure 5.19. Since the measured vertical force in all the remaining types of reinforced earth model wall, (strip width of 10mm to 32mm) was lower than the overburden force, the vertical pressure distribution is as illustrated in Figure 5.19 although the magnitude of the pressure varies.

It can be concluded that the presence of reinforcement strips modifies the vertical pressure distribution across the model wall. There is arching of the vertical pressure where the pressure over the strip is affected by the width of the strip. This pressure is more than the overburden. The arching is also influenced by the fill height.

5.6 Conclusion

The experiments performed show that the vertical pressure distribution at the wall base level within both the reinforced earth and the unreinforced backfill is nonlinear. Maximum vertical normal stress occurs at the toe of the wall and there is lowering of vertical normal stresses within the middle of the reinforced zone. Vertical normal stresses in an unreinforced wall is also nonlinear.

Although the vertical pressure at any particular point is affected by the length of reinforcement, the vertical pressure within the reinforced zone i.e. the shape of the pressure diagram is not affected by this parameter.
Figure 5.19 Arching in model walls with different strip widths

(a) Arching in reinforced earth model wall with 10mm strips

(b) Arching in reinforced earth model wall with 20mm strips
The investigation has shown that the width of the reinforcement strip has an effect on the vertical pressure in a reinforced earth model wall. It influences the vertical pressure distribution along the length of the wall, although the trend of the distribution remains the same for both the reinforced and unreinforced zones. Moreover, if one compares the total measured vertical force with the theoretical overburden, it can be concluded that the distribution across the wall (width B direction) is also influenced by the width of the reinforcement strips. This interpretation is based on occurrence of arching across the reinforcement strips.
CHAPTER 6
DIRECT SHEAR TESTS ON REINFORCED SAND

6.1 Introduction

Direct shear tests were performed on reinforced sand samples to understand more clearly the soil-reinforcement interaction in reinforced soil. The tests were conducted using a purpose-built Direct Shear Apparatus. The samples simulated the configuration of a reinforced earth wall. Thus the investigation allows the study of the global behaviour of reinforced earth at a unit cell level. Several series of investigations were performed, with the following aims:

1) to study the influence of reinforcement strip width on reinforced soil
2) to study the behaviour of soil reinforced with wide strips
3) to study the effect of different types of reinforcement on reinforced soil
4) to study the behaviour of reinforced soil submerged in water

The findings of these investigations highlighted the fact that the soil-reinforcement mechanism is very much influenced by the dimension and type of the reinforcement strip and that the reinforcement is effective in strengthening soil submerged in water.
6.2 The Direct Shear Apparatus

The Direct Shear Apparatus was purpose built for investigation of reinforced soil. It consisted of several parts:

1) internal box
2) external box
3) bottom and top platens
4) serrated upper and lower plates
5) hand-operated jack loading mechanism
6) load hanger
7) three LVDT (Linear Voltage Displacement Transducer) and a dial gauge
8) proving ring

Figures 6.1 and 6.2 show the Direct Shear Apparatus. The internal box was constructed from 30mm brass plates and has internal dimensions (sample size) of 300mm square and a depth of 170mm. The serrated bottom plate has 11 slots of 240mm wide by 2mm, spaced at a distance of 25mm between each other to allow reinforcement strips to be secured to the slots (Figure 6.3). This arrangement allows the reinforcement strips to be placed perpendicular to the bottom plate and different spacings of reinforcement may be tested.

The internal box was placed inside the external box where the massive weight of the brass internal box ensured that there was no relative movement of the external and the
(a) Components of shear box (plan):
  foreground: bottom and top platens
  background: serrated upper and lower plates

(b) Shear box ready for testing

Figure 6.1  The Direct Shear Apparatus
Figure 6.2  Direct Shear Apparatus showing loading system

Figure 6.3  Serrated bottom plate with slots
internal boxes. After the sample was prepared, the vertical load was applied by means of weights attached to the load hanger. The load was uniformly distributed over the sample through the top platen.

The shearing load was applied by a hand-operated jack. The shear resistance of the sample was measured by the proving ring. A HP 7DCDT-100 LVDT was connected to the proving ring to allow electronic datalogging of the shear load. The test was strain controlled where the horizontal or shear displacement was measured by a HP 7DCDT-1000 LVDT. Another HP 7DCDT-100 was installed to measure the vertical displacement of the samples during the test. The datalogging was done using the Keithley/Soft 500 System connected to a personal computer. Figure 6.4 shows the layout of the testing system.

Due to its massive weight, it was not viable to have the upper part of the brass shear box separated from the lower part during shearing, as with a conventional small shear box (60mm square). Due to its weight and contact with the lower part of the shear box, the upper part of the shear box has its own resistance to shear displacement, thus shear resistance readings from the test had to be corrected for this additional resistance. The box resistance was assessed by shearing the unfilled shear box. This test was repeated several times and the shear resistance versus the shear displacement curve was plotted (Figure 6.5). Regression analysis was performed on the data and the resistance of the shear box was found to be:
Figure 6.4  Layout of the testing system

Figure 6.5  Shear resistance of shear box without any sample (i.e. upper part of box touches lower part)
\[ F_{\text{Box}} = 343.736 + 2.1509x_1 \]  

(6.1)

where \( x_1 \) is horizontal shear displacement

This resistance was incorporated in the FOTRAN program which transformed the measurement into a comprehensible data form, Appendix P2. The resistance can also be defined in terms of the friction angle \( \phi_{SB} \). This was found by comparing the uncorrected friction angle of the sample with the corrected friction angle. The mean \( \phi_{SB} \) was found to be 0.50, Appendix M. It can be seen that the resistance of the shear box gave an added friction angle to the sample being sheared.

6.3 Data Acquisition System

In order to achieve greater accuracy, maximize test data and minimize the manpower required to run the test, the measurements of test data was automated, where an electronic data acquisition system, the Keithley/Soft 500 was installed to run with the Direct Shear Apparatus. The Keithley system together with the LVDTs and a 640K RAM Sperry personal computer formed the measuring system of the shear displacement, vertical displacement and the shearing force during the tests.

The Linear Voltage Displacement Transducers were supplied with an input voltage and the output voltages were proportional to the physical changes being measured and the function of the Keithley/Soft 500 was to monitor these voltages. The Keitley was run by a program which has BASIC
and Soft 500 routines, Appendix N.

The program calls the Keithley subroutines. INIT is called to commence the Soft 500 routines. The IONAME subroutine opens the channel to each of the transducers and the instantaneous voltage of each of the transducers is read by ANREAD.

The data acquisition was started and stopped by a test operator thus the number of manpower to conduct the test was two: one to turn the wheel of the jack system and one to manage the data acquisition.

6.4 Sample Preparation

Once the arrangement of the reinforcement strips was decided, the strips were fixed to the serrated bottom plates. This was achieved by bending 10mm of one end of the strips through an angle of 90°. Thus when the strip was pushed through the slot in the serrated plate, free end first, the bend formed a catch with the plate. The strip was then restrained by masking tape. The serrated plate with attached reinforcement strips was then placed on top of the bottom plate in the internal box, Figure 6.6a.

Sand was poured into the internal box using a scoop. In order to ensure that the relative density of each sample was the same for all tests, the sand was poured from a fixed level and filled layer by layer. The level chosen was the top of the internal box and four scoops of sand were poured for each layer: one scoop of sand for each corner of
the box. The mass of sand used for each sample was 24.4kg. When all the sand had been poured, the surface of the sample was levelled. It was then ready to be loaded. The schematic diagram of the sample is shown in Figure 6.6b.

The sand was dried local beach sand with a uniformity coefficient of 2.0. It was the same sand as used in the reinforced earth model walls described in Chapters 4 and 5. The sand was reused for up to fifty sample tests, after which it was replaced by another batch of sand. This precaution was taken to ensure that there was no significant buildup of fines due to crushing during shear.

Since the mass and the volume of the sand was large, the weight and volume of the reinforcement were negligible, thus the sample preparation resulted in a sample density of 15.64kN/m³. The initial void ratio was 0.63 and the relative density (Appendix 0) was 60%. The sample can be classified as medium dense (Lambe and Whitman, 1979).

6.5 Reinforcement Strips

The material used as the reinforcement strips throughout the investigations was cut out of aluminium cans. It was an aluminium alloy with the modulus of elasticity of 69000MPa. Its thickness was 0.12mm. The yield strength was 248MPa and the ultimate tensile strength was 283MPa. The tensile strength and the cross section of the strips ensured that the strips would not break in tensile failure.
Figure 6.6  (a) Plan view showing orientation of reinforcement strips in shear box; (b) The schematic diagram of the sample which is ready for testing.
The length of the reinforcement strips used was kept constant at 160mm, Figure 6.7. The 10mm bend was for connection to the serrated bottom plate as described earlier.

6.6 Direct Shear Test

6.6.1 Test procedure

The procedures for the direct shear test were:

1) prepare the sample
2) prepare the data acquisition system
3) place the serrated top plate on the prepared sample
4) place the top platen
5) place the hanger onto the top platen and attach the required weights to the hanger
6) start the test using the hand operated jack by turning the wheel clockwise at a constant rate. Simultaneously start the data acquisition
6) stop the test and data acquisition when the sample has reached its residual strength.

The average shearing rate was 0.82mm/sec (49.7mm/min) with a standard deviation of 0.10mm/sec. In order to determine when to stop the test, a mechanical dial gauge was attached to the proving ring. When the residual strength of the sample was reached the needle remained at a constant reading. The test was stopped at this point.
A reinforcement strip

Figure 6.7  A reinforcement strip
The data obtained was used to plot the shear force versus the shear displacement of the sample. This plot was used to determine the force-displacement behaviour of the sample. The data was also used to determine the peak shear resistance needed in plotting the graph of shear stress versus applied normal stress from which the shear strength parameters, \(c\) (cohesion) and \(\phi\) (internal friction angle) were obtained.

6.6.2 Test data

An example of the voltage measurements acquired is shown in Appendix P1. These are the voltage readings of each transducer. The first column shows the time. It can be seen that readings were taken every 0.3 second. In order to convert this voltage reading to a comprehensible form, a FORTRAN program (Appendix P2) was written to read the data and calculate the shear displacement, vertical displacement and the shear force. One set of average readings which appears in the final product is taken from five sets of readings. The final product is shown in Appendix D3 where the maximum shear force reading was used to plot the graph of shear stress versus applied normal stress.

It should be noted that the transducers had been calibrated and their calibration factors were used in the FORTRAN program. The calibration chart for each of the transducers is shown in Appendix Q. It should also be noted that even though the facility to measure the vertical displacement was installed, in the actual tests the
vertical displacement was not measured. During a few test runs where the LVDT to measure the vertical displacement was placed at the centroid of the top platen, the vertical displacement readings were erratic thus it was decided to do away with measurement of vertical displacement. The erratic reading was due to tipping of the top platen. Tipping will be discussed in another section.

During the test, observation of the mechanical dial gauge showed that the force-displacement behaviour of all the samples can be generalized by curve A in Figure 6.8. It shows that the shearing resistance increased with shear displacement, reached a maximum point and remained at this level. Thus the peak resistance was also the residual strength of the sample. The data obtained from electronic measurements, however, show that the general force-displacement behaviour is consistent with Curve B in Figure 6.8.

It can be seen that towards the end of the test, even though the mechanical dial gauge registered a constant reading, the LVDT attached to the proving ring still registered an increase in shear resistance but at a diminishing rate. This was because the transducer was more sensitive and accurate than the mechanical dial gauge. For this reason at all the shear tests, the electronic datalogging was stopped as soon as the mechanical dial gauge reached a constant reading.
Figure 6.8 Force-Displacement curves during direct shear test obtained through mechanical (A) and electronic (B) gauges.
6.6.3 Tipping

It was observed during the tests that there was tipping of the top platen. This was because the Direct Shear Apparatus has the conventional arrangement where the top platen is free to rotate. The tipping is illustrated by Figure 6.9. The amount of tipping, \( \alpha \), was observed to vary from 0.4\(^{0}\) to 1.5\(^{0}\).

The tipping mechanism (Figure 6.10) was explained by Jewell (1989). In direct shear test, most of the shear stress is transmitted to the soil from the side walls. This load transfer subjects the upper part of the box and the soil sample to an equal and opposite force forming a couple. In order to restore equilibrium, the sample rotates to mobilize stresses between the soil and the side walls thus giving the required balancing moment.

The consequence of tipping is that it creates a non-uniformity of stresses across the centre of the sample. In order to achieve greater uniformity of stresses, the top loading plate could be secured to the top half of the box so that the upper half of the box moves as a unit during shearing. With this arrangement the test is symmetrical about the central plane and a non-uniform vertical stress distribution is generated across the top of the sample to balance the couple.

It should be noted that although in the investigations conducted all of the samples were subjected to tipping, it
Figure 6.9  Tipping of top platen
Definitions for direct shear test

Conventional arrangement - free top platen

(a)  
(b)  
(c)  

Improved arrangement - fixed top platen

(d)  
(e)  
(f)  

Forces in direct shear test showing couple applied by shear force. With free top platen (a to c) this results in non-uniform distribution of stress on central plane; symmetrical test with fixed top platen (d to f) provides more uniform stress distribution on central plane.

Figure 6.10 The tipping mechanism (after Jewell, 1989)
had no effect on the final results since the investigations were of a comparative nature.

6.7 Background Study

6.7.1 Theoretical model

The theoretical model of the direct shear test of reinforced soil, presented in Chapter 2, is repeated in this section to clarify the work performed in this chapter.

In direct shear, the interaction of soil and reinforcement results in a force, $P_R$, being induced in the reinforcement, Figure 6.11. This force has a normal component:

$$P_R \cos \theta$$

and a tangential component:

$$P_R \sin \theta$$

The normal component reduces the shear stress which the soil must support to:

$$\tau = \tau_{yx} - \frac{P_R}{A_{er}} \sin \theta$$

(6.2)

and increases the normal effective stress in the sand to:

$$\sigma = \sigma_{yy} + \frac{P_R}{A_{er}} \cos \theta$$

(6.3)
Figure 6.11 Equilibrium analysis of reinforced soil (repeated from Figure 2.5)
This allows additional frictional shearing resistance to be mobilized. Thus the effect of the reinforcement is to increase the shearing resistance by an amount:

$$\tau_{ext} = \frac{P_R}{A_{sr}} \left( \cos \theta \tan \phi + \sin \theta \right) \quad (6.4)$$

In the author's investigation, the reinforcement strips were placed perpendicular to the shearing plane where the bottom ends were fixed to the serrated bottom plate, thus the initial $\theta$ was zero. When the sample was sheared, there was deformation of the reinforcement strip which changed the value of $\theta$. The theoretical model for this system is shown in Figure 6.12. It is based on the model used by Gray and Ohashi (1983).

Results from limit equilibrium analysis can be applied in this case where the shear strength increase, due to the presence of reinforcement, is defined by Equation 6.4 where $P_R$ is the product of the tensile stress developed in the reinforcement strip and the cross sectional area of the strip.

There are two likely possibilities of the tensile stress distribution along the strip: linear or parabolic distribution, with tensile stress being maximum at the shear plane and decreasing to zero at the reinforcement ends (Gray and Ohashi, 1983). However Jewell and Wroth (1987) found from measurements that the reinforcement force varied approximately linearly from a maximum at the shear
Figure 6.12 Model of deformation of reinforcement in direct shear test (after Gray and Ohashi, 1983) (repeated from Figure 2.10)
plane to zero at both ends of the reinforcement.

6.7.2 Stress and strain of reinforced soil in direct shear test

Jewell and Wroth (1987) had conducted several series of direct shear tests on reinforced soil. The work of these researchers have broadened our knowledge on the state of stress and strain of reinforced soil. The main part of their investigation was concerned with the force induced in the reinforcement. They have shown that the effect of reinforcement on soil shearing resistance depends directly on the mobilized reinforcement force. The extension in the reinforcement and hence the mobilized reinforcement force, depends on the strain in the soil.

Radiographic observation shows that the reinforcement causes more sand to deform and help resist localized shear deformation. It was found that additional shearing resistance increased with axial stiffness of the reinforcement. When reinforced soil is sheared the soil will deform as it mobilizes additional shearing resistance where the soil close to the reinforcement will try to deform with respect to the reinforcement. If the reinforcement is stiff, rough and lying in the direction of tensile incremental strain in the soil, it will resist the deformation. Since there is shearing force, the principal axes of stress will rotate so that the soil can come into equilibrium.
Their investigation shows that plastic soil strain rather than elastic soil strain governs the force generated in the reinforcement and that placing reinforcement close to the direction of principal incremental tensile strain in the soil and hence perpendicular to the direction of compressive stress will give rise to the maximum rate of increase in force. Since the ratio of the principal incremental tensile and compressive strains increases with the angle of dilation, considerably less deformation is required to generate reinforcement forces in dense sand than in loose sand.

In the shear test, the reinforced sample will reach the peak shearing resistance when there is slip at the soil-reinforcement interface or when the direction of zero incremental strain is aligned with the reinforcement whereupon the sand will continue to shear as in the unreinforced sample.

6.8 Current Study

Four individual investigations had been performed using the Direct Shear Apparatus. They are:

1) Experiment A - to find the influence of reinforcement width

2) Experiment B - to find the behaviour of soil reinforced with wide strips
3) Experiment C - to find the influence of strip type on reinforced soil

3) Experiment D - to find the behaviour of submerged reinforced soil

In order to avoid any possible confusion, the experiments will be presented in individual sections, denoted as 6A, 6B, 6C and 6D for Experiments A, B, C and D, respectively.
SECTION 6A

EXPERIMENT A

The Effect of Strip Width on Reinforced Soil

6A.1 Aim

The aim of this investigation is to find the influence of the reinforcement strip width on the behaviour of reinforced soil in a shear box test. The term 'behaviour' encompasses the force-displacement behaviour and the strength of the reinforced soil.

6A.2 Introduction

The reinforcement strip width has been found to affect the apparent friction coefficient (Schlosser and Elias, 1978) and the author's present investigation on reinforced earth model walls has shown that the vertical pressure distribution is influenced by the strip width. The strip width affects the area of soil-reinforcement interface and the stiffness in axial extension and bending of the strip. In the author's investigation, direct shear tests were performed where the varying parameter was the reinforcement strip width. Throughout the series the ratio of the cross sectional area of reinforcement strip over the shearing area or reinforcement density was kept constant at 0.04%. Four series of direct shear test were conducted where a series comprised of tests on sand reinforced with a
particular width of strip. The reinforced samples shall be described as a composite.

The widths tested were 10mm, 15mm, 20mm, 30mm and 60mm. The arrangement of reinforcement strips within the shear box for each series is shown in Figure 6.A1. Group effect (Guilloux et al., 1979), if there is any, would not influence the comparison between series since the width of strip over spacing ratio is constant at 0.2 for all the series. In each series the samples were tested under eight different confining pressures. The applied normal pressures were 6.74kPa, 7.96kPa, 8.98kPa, 10.08kPa, 11.16kPa, 12.26kPa, 13.34kPa and 14.44kPa. At these low confining pressures the strength enhancement of the reinforced soil can be explained using the enhanced angle concept (Hausmann, 1976; Ingold, 1982). The low confining pressure ensured that the reinforced soil failed from slip at the interface rather than tensile rupture of the reinforcement.

Under each confining pressure, the peak shear resistance was determined. The peak shearing resistance was reached when there was slip at the reinforcement interface or when the direction of the zero deformation was aligned with the direction of the reinforcement. In the latter, the reinforcement was deemed as no longer effective and the sand sheared as it was unreinforced. Graphs of the peak shearing resistance versus the applied normal stress were drawn to obtain the shear strength parameters according to the Coulomb's equation (Terzaghi and Peck, 1967):
Figure 6A.1  Arrangement of reinforcement strips within shear box for constant reinforcement density $A_r/A_x$ of 0.04%
\[ \tau_s = c + \sigma \tan \phi \]  

(6A.1)

where \( \tau_s \) is shear strength  
\( c \) is cohesion  
\( \sigma \) is confining pressure  
\( \phi \) is internal friction angle of soil

A series of direct shear tests were also performed on unreinforced sand to act as a control. For reinforced sand the internal friction angle obtained is designated by \( \phi_r \) - the internal friction angle of the composite.

6A.3 Test Results

6A.3.1 Force-Displacement behaviour

The behaviour of the samples during shearing is shown in Figures 6A.2, 6A.3, 6A.4, 6A.5, 6A.6 and 6A.7. The figures show the force-displacement behaviour of the samples of unreinforced sand, composite with 10mm strips, composite with 15mm strips, composite with 20mm strips, composite with 30mm strip and composite with 60mm strips respectively; at various applied normal pressures. It can be seen that, for all samples, the shearing resistance is only mobilized after some initial shear displacement; this displacement ranges from 1mm to 5mm or 0.3\% to 1.7\% of the length of the shear box (300mm). There is no correlation between this displacement and the applied confining pressure or the type of sample. It may be suggested that,
Figure 6A.2 Force-displacement behaviour of unreinforced sand

Figure 6A.3 Force-displacement behaviour of composite with 10mm strips
Figure 6A.4 Force-displacement behaviour of composite with 15mm strips

Figure 6A.5 Force-displacement behaviour of composite with 20mm strips
Figure 6A.6 Force-displacement behaviour of composite with 30mm strips

Figure 6A.7 Force-displacement behaviour of composite with 60mm strips
at initial displacement, only a small mass of the sample is resisting the shear and after further displacement, the sample acts as whole body to resist the shear.

The force-displacement plot for unreinforced sand shows that at applied normal pressure of 6.74kPa, the shear resistance of the sample was mobilized once the sample was sheared and then the resistance tapered off. At normal pressures of 8.98kPa and 14.44kPa, at low shear displacement, the shear resistance mobilized was small but higher shear resistance was mobilized with increasing strain. The rate of mobilization of shear resistance then tapered off. The plot illustrates that the maximum shear resistance increases with an increase in applied normal pressure.

The force-displacement plot for composite with 10mm strips shows that at applied normal pressures of 6.74kPa, 8.98kPa and 14.44kPa, only a small shear resistance was mobilized initially but at around shear displacement of 5mm, the shear resistance of the composite was mobilized at a high rate which then tapered off with a further increase in shear displacement. At normal pressure of 6.74kPa, at shear displacement of 10mm, the shear resistance dropped but increased again with further shear displacement. The plot shows that the peak shear resistance increases with applied normal pressure.

The force-displacement plot for composite with 15mm strips shows that at applied normal pressures of 6.74kPa,
8.98kPa and 14.44kPa, at displacement of 3mm the shear resistance of the composite was mobilized at a high rate. The rate then tapered off. At applied pressure of 6.74kPa, at shear displacement of 10mm the shear resistance dropped but increased again with further shear displacement. The plot shows that peak shear resistance increases with an increase in applied normal pressure.

The force-displacement plot for composite with 20mm strips shows that for applied normal pressures of 8.98kPa and 14.44kPa, the rate of mobilized shear resistance was highest between shear displacement of 5mm to 10mm then the rate tapered off. The plot shows that for a given shear displacement higher shear resistance was mobilized when the applied normal pressure was higher.

The force-displacement plot for composite with 30mm strips shows that at applied normal pressure of 6.74kPa and 8.98kPa, the slope of the stress-strain curve are identical. At applied pressure of 14.44kPa, after shear displacement exceeded 3mm there seemed to be three transitions on the stress-strain curve: the rate of mobilized shear resistance increased from shear displacement of 3mm then tapered off. At 14mm displacement the rate increased again and then tapered off. The rate increased again at 21mm shear displacement and then tapered off. The plot shows that for a given shear displacement, higher resistance was mobilized with increases in applied normal pressure.
The force-displacement plot for composite with 60mm strips show that at applied pressure of 6.74kPa and 8.98kPa, the rate of mobilized shear resistance was high after shear displacement of 4mm. The rate then quickly tapered off. At 14.44kPa there were three distinctive transitions on the stress-strain curve where at each transition the rate of mobilized shear resistance increased then tapered off. The plot shows that higher shear resistance was mobilized with higher applied pressure.

In order to determine whether the discontinuities which are very distinctive for composite with 60mm strips have any significance, the force-displacement curves for all the samples were closely examined. It was found that generally the curves are composed of many small transitions or discontinuities. These transitions are also present in the curve at applied pressure of 14.44kPa for unreinforced sand. The transitions in sand with 60mm strips are more distinctive because there were no data points between displacement 18mm to 23mm.

By tranposing the figures on each other it was found that the force-displacement curves for composite with 20mm strips and composite with 30mm were almost identical.

All the samples tested, both reinforced and unreinforced, has a similar characteristic in their force-displacement behaviour. In general the shearing resistance of the sample was mobilized at a high rate after an initial shear displacement. The rate quickly decreased
and then tapered off to the peak shear resistance.

The force-displacement curves show that all the samples obey the Coulomb strength equation where the peak resistance increases with increase in applied normal pressure henceforth the shear strength parameters of the different type of samples may be obtained from this test.

Observation of all the force-displacement curves show that there seem to be no correlation between applied normal pressure and the shear displacement at peak shear resistance. There is also no correlation between the shear displacement at peak shear resistance and the different types of samples. The range of shear displacement of which the samples were observed to reached peak shear resistance was 15mm to 28mm.

In order to determine whether the stiffness of the reinforced sample is affected by the width of the reinforcement strip, the test results from various reinforced samples were compared. It should be noted that tangent modulus is usually taken from a plot of stress versus strain. Here one has to use force-displacement plots. Therefore the average tangent modulus was taken as the average slope of the curve between shear displacement of 5mm to 15mm. This range of displacement was chosen after observing that in all the force-displacement curves available, the rate (force/displacement) of mobilized shear resistance was fairly constant for displacement between 5mm and 15mm. Although referred to in this chapter as a tangent
modulus, it should be regarded as only a qualitative index of stiffness. The tangent modulus of the reinforced samples when sheared under applied normal pressure of 14.44kPa is tabulated in Table 6A.1.

Table A1 Tangent Modulus of Samples at Normal Stress of 14.44kPa

<table>
<thead>
<tr>
<th>Width of strip (mm)</th>
<th>Tangent Modulus (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td>51</td>
</tr>
<tr>
<td>10</td>
<td>70</td>
</tr>
<tr>
<td>15</td>
<td>77</td>
</tr>
<tr>
<td>20</td>
<td>57</td>
</tr>
<tr>
<td>30</td>
<td>58</td>
</tr>
<tr>
<td>60</td>
<td>55</td>
</tr>
</tbody>
</table>

It can be seen that unreinforced sand had the lowest tangent modulus and the composite with 15mm strips had the highest tangent modulus. The tangent modulus of composite with 20mm strip, 30mm strips and 60mm strips were approximately the same. This result shows that the presence of reinforcement stiffened the sand by upto 51%.

6A.3.2 Shear strength parameters

Graphs of shear stress versus applied normal stress were plotted for each series of experiments. They are presented in Appendix R. Regression analyses were performed
on the data to determine the shear strength parameters $c$ and $\phi$. The strength parameters obtained are tabulated in Table 6A.2

Table 6A.2 Shear Strength Parameters

<table>
<thead>
<tr>
<th>Width of strip (mm)</th>
<th>$c$ (kPa)</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td>1.64</td>
<td>41.0°</td>
</tr>
<tr>
<td>$w = 10$</td>
<td>1.76</td>
<td>47.9°</td>
</tr>
<tr>
<td>$w = 15$</td>
<td>1.04</td>
<td>49.6°</td>
</tr>
<tr>
<td>$w = 20$</td>
<td>2.48</td>
<td>46.0°</td>
</tr>
<tr>
<td>$w = 30$</td>
<td>3.99</td>
<td>42.8°</td>
</tr>
<tr>
<td>$w = 60$</td>
<td>1.31</td>
<td>46.6°</td>
</tr>
</tbody>
</table>

The results are illustrated in Figure 6A.8. It can be seen that all of the composites have higher internal friction angle than unreinforced sand and there is an optimum width between 10mm to 30mm which resulted in a composite with the highest internal friction angle. The internal friction angles for composite with 30mm and composite with 60mm are approximately the same. When their force-displacement curves were transposed onto each other it was found that from zero to 15mm shear displacement their force-displacement curves were almost identical.

For completeness, the difference in the internal friction angle of unreinforced sand obtained in the above
investigation (41°) and that from standard 60mm square shear box (35.4°) described in Section 4.2.3 shall be discussed. In both investigations the initial void ratio of the sample was the same. The higher internal friction angle obtained in the investigations presented in this section could be attributed to the low confining pressures applied: 6.74kPa to 14.44kPa while in the results described in Section 4.2.3 the applied confining pressures were 14.51kPa to 28.14kPa. This reasoning is supported by the finding of Soydemir and Espinosa (1978), from a series of direct shear tests performed in the 60mm square box. The tests were to find the internal friction angle \( \phi \) of unreinforced sand, the soil-reinforcement friction angle \( \psi \), and their relationship with applied confining pressures. As can be seen from Figure 6A.9 the friction angles obtained at a low vertical stress or applied confining pressure are high compared to that obtained at higher confining pressures.

6A.4 Analysis of the Results

The results of the tests were analysed using the enhanced friction angle concept (Hausmann, 1976; Ingold, 1982), where the apparent cohesion was disregarded.

Schlosser and Long (1974) have shown through triaxial tests that, at low confining pressure, the shear strength of reinforced soil is characterized by an apparent internal friction angle which is larger than the internal friction angle of unreinforced soil and the ultimate strength is
Figure 6A.8 Coulomb failure envelopes for various types of samples

Figure 6A.9 Mobilized friction under various confining pressures (after Soydemir and Espinosa, 1978)
governed by the adherence failure of the soil and reinforcement. At high confining pressure the strength of the reinforced soil is characterized by an apparent cohesion and the ultimate strength of the reinforced soil is governed by tensile rupture of the reinforcement, Figure 6A.10. The transition in the failure envelope has also been observed in the results of direct shear tests on reinforced sand performed by Gray and Ohashi (1983).

Since the maximum applied normal pressure used in the investigation presented in this section was only 14.44kPa, it is justifiable to characterize the strength enhancement of the reinforced sand purely as an enhanced internal friction angle. The decision to disregard the c parameter is further justified by the low value of apparent cohesion obtained for each series which can be attributed to limitation of test accuracy.

This reasoning is supported by the value of cohesion of 1.64kPa for cohesionless dry unreinforced sand. Bowles (1978) points out that, for cohesionless material, a small apparent internal friction should be neglected unless it is more than 10kPa to 15kPa.

One of the factors influencing the test accuracy was the self-weight of the sample. In the tests, the self-weight of the samples were disregarded. However if the normal pressure of 1.25kPa which is due to self-weight (acting on the shearing plane) is included, the c value of the unreinforced dry sand will be very small at 0.55kPa.
Unreinforced

Reinforced

 Applied Confining Pressure $\sigma_3$

Figure 6A.10 Transition in strength enhancement of reinforced soil (after Schlosser and Long, 1974) (repeated from Figure 2.19)
The internal friction angle remains unaffected.

Having established that the strength of reinforced sand is characterized by an enhanced internal friction angle, the experimental results can be illustrated by Figure 6A.11.

By comparing the tangent modulus and the internal friction angle of the various composite and unreinforced sand samples, it can be seen that the unreinforced sand has the lowest stiffness and the lowest internal friction angle. The composite with strips of 15mm has the highest stiffness and the highest internal friction angle.

The result for composite with 60mm strips is disregarded in the analysis since it digresses from the optimum width observed when sand was reinforced with narrower strips. Since there was a large gap between the largest width of 60mm and the second largest width of 30mm, this decision was justified. It was suspected that there was some mechanism that becomes significant once the difference in width was large, thus another separate series of investigations were conducted to determine the behaviour of reinforced soil with wide reinforcement strips.

Further analysis of the results was based on efficiency factor defined as:
Figure 6A.11 Experimental results showing enhanced friction concept
\[ e_r = \frac{\tan \phi_r}{\tan \phi} \] (6A.2)

where \( \phi_r \) is the internal friction angle of the composite

\( \phi \) is the internal friction angle of unreinforced sand

The efficiency factor shows the strength of the composite as compared to unreinforced sand, thus the efficiency factor of unreinforced sand is 1. The efficiency factor for composite with each width of reinforcement is tabulated in Table 6A.3.

Table 6A.3  Efficiency Factors for Various Samples

<table>
<thead>
<tr>
<th>Width of strip (mm)</th>
<th>Unreinforced sand</th>
<th>( e_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w = 0 )</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>( w = 10 )</td>
<td></td>
<td>1.27</td>
</tr>
<tr>
<td>( w = 15 )</td>
<td></td>
<td>1.35</td>
</tr>
<tr>
<td>( w = 20 )</td>
<td></td>
<td>1.19</td>
</tr>
<tr>
<td>( w = 30 )</td>
<td></td>
<td>1.06</td>
</tr>
</tbody>
</table>

The influence of the strip width on the strength of sand is illustrated by Figure 6A.12. Comparison of the efficiency values shows that the reinforcement strip of width 15mm is most efficient in strengthening the soil. The composite has 35\% more shear strength than unreinforced sand.
Figure 6A.12  Effect of strip width on strength of soil
6A.5 Conclusion

The tests show that the presence of reinforcement increases the stiffness and the strength of the sand. The stiffness of the sand was increased by upto 51% and its strength were enhanced by upto 35%.

For a given concentration of reinforcement, the width of the reinforcement affects the strength of the reinforced soil. There is an optimum width which results in a reinforced soil with the highest strength.
SECTION 6B
EXPERIMENT B
The Behaviour of Reinforced Soil with Wide Strips

6B.1 Introduction

In Experiment A, because of the necessity to maintain the $A_r/A_s$ constant, there was a large gap between the largest (60mm strips) and the second largest (30mm strips) width tested. The composite with the 60mm strip have larger strength than the composite with 30mm strips and this behaviour digressed from the result obtained for narrow strips. It can be hypothesised that the reason the composite with 60mm strips have higher strength than composite with 30mm strips is because the 60mm strip have higher axial stiffness.

Jewell and Wroth (1987) found that increasing the axial stiffness of the reinforcement increases the strength of the composite proportionally. They found that the mobilized reinforcement force increases with an increase in axial stiffness. If the result from Jewell and Wroth is to be applied directly to the investigation presented here, one would expect the strength of the composite with the 'narrow' strips to be increased with increasing width of the strips used. However this was not observed, which suggests that there is other factor (or factors) governed by the width of the strip that influence the strength of the composite.
It should be noted that the shape of reinforcement used by Jewell and Wroth was different from the shape of the strip used in the author's investigation; where the strips were thin and flat. Due to the shape of the strips combined with the fact that the spacing of strips along the shear box is kept constant throughout the investigation, varying the width of strips would affect the stress across the shear box.

Experiment A suggests that the strips used can be categorized as 'narrow' strip and 'wide strips'. Experiment B was conducted to determine the effect of strip width on the strength of the composite when the strips are wide.

It could be argued that the presence of reinforcement in the sample gave an added strength to the soil due to the rigidity of the reinforcement, where the strip resists bending during shear, hence the bending stiffness of the strip should be considered. In devising the investigation, however, the author has assumed that the strips were thin enough to offer little if any resistance to bending. Thus, the samples sheared as a unit which can be proven by examining the tangent moduli of composite with 30mm strips and composite with 60mm strips.

Individually, the bending stiffness of the 60mm strip was twice that of the 30mm strips, but the stiffness moduli of the two composites are almost identical. This proves that reinforced samples act as a unit during shearing; in other word the reinforced samples are true composites.
6B.2 Experimental Set-up

A total of five series of direct shear tests were performed on reinforced sand. The difference between each series was in the width of the strips used. In each series, six reinforcement strips were used and the arrangement is shown in Figure 6B.1. The $A_r/A_s$ ratio, or reinforcement density, was not kept constant to allow a close interval between the widths tested. The widths of the strips tested were 30mm, 35mm, 40mm, 45mm and 50mm. The resulting $A_r/A_s$ were 0.040%, 0.047% 0.053% 0.060% and 0.067 %, respectively.

In each series the applied confining pressures were 6.74kPa, 7.86kPa, 8.98kPa, 10.08kPa, 11.16kPa, 12.26kPa, 13.34kPa and 14.44kPa. For each series of tests the maximum shearing resistance at each applied confining pressure was used to plot the Coulomb failure envelope to find the shear strength parameters. The low confining pressures used allowed the strength enhancement due to reinforcement to be characterized by an enhanced angle of friction.

6B.3 Test Results

6B.3.1 Force-Displacement behaviour

The behaviour of the samples during shearing is shown in Figures 6B.2, 6B.3, 6B.4, 6B.5 and 6B.6. They show the force-displacement behaviour of the samples of composite with 30mm strips, composite with 35mm strips, composite with 40mm strips, composite with 45mm strips and composite
Figure 6B.1 Arrangement of reinforcement strips in shear box for each series
Figure 6B.2  Force-displacement behaviour of composite with 30mm strips

Figure 6B.3  Force-displacement behaviour of composite with 35mm strips
Figure 6B.4 Force-displacement behaviour of composite with 40mm strips

Figure 6B.5 Force-displacement behaviour of composite with 45mm strips
Figure 6B.6 Force-displacement behaviour of composite with 50mm strips
with 50mm strips, respectively, at various applied normal pressures.

It can be seen that, for all the composites, the trends in the force-displacement curves are similar. After some initial shear displacement the shearing resistance was mobilized at a high rate. The rate then quickly tapered off to a peak shear resistance. The peak shear resistance increased with the increase in applied normal pressure.

The force-displacement curves at applied normal pressure of 14.44 kPa were used to find the tangent modulus of each type of sample. The average tangent moduli were taken between shear displacement of 5mm to 15mm. The tangent moduli along with the shear deformation (as percentage of the sample size) at peak shear resistance, at applied normal pressure of 14.44 kPa are tabulated in Table 6B.1.
Table 6B.1 Tangent Modulus and Relative Deformation at Peak Shear Resistance

<table>
<thead>
<tr>
<th>Width of strip (mm)</th>
<th>Tangent Modulus (N/mm)</th>
<th>Relative deformation at peak shear resistance (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>w = 0</td>
<td>51</td>
<td>6.2</td>
</tr>
<tr>
<td>w = 30</td>
<td>62</td>
<td>8.6</td>
</tr>
<tr>
<td>w = 35</td>
<td>70</td>
<td>9.9</td>
</tr>
<tr>
<td>w = 40</td>
<td>67</td>
<td>9.9</td>
</tr>
<tr>
<td>w = 45</td>
<td>67</td>
<td>9.6</td>
</tr>
<tr>
<td>w = 50</td>
<td>59</td>
<td>8.4</td>
</tr>
</tbody>
</table>

It can be seen that the stiffness of any type of the composite is higher than the stiffness of unreinforced sand and that even though the composite with 50mm strips had the highest concentration of reinforcement, its stiffness was the lowest among the composites. Comparison of tangent modulus with relative deformation at peak shear resistance indicate that relative deformation at peak shear resistance, increases with the increase of stiffness of the sample.

6B.3.2 Shear Strength Parameters

Graphs of shear stress versus applied normal stress were plotted for each series of experiments. They are presented in Appendix S. Regression analyses were performed on the data to determine the shear strength parameters. The strength parameters obtained is tabulated in Table 6B.2.
Table 6B.2  Shear Strength Parameters

<table>
<thead>
<tr>
<th>Width of strip (mmm)</th>
<th>c (kPa)</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$w = 0$</td>
<td>1.64</td>
<td>41.0$^\circ$</td>
</tr>
<tr>
<td>$w = 30$</td>
<td>3.99</td>
<td>42.8$^\circ$</td>
</tr>
<tr>
<td>$w = 35$</td>
<td>-0.25</td>
<td>53.4$^\circ$</td>
</tr>
<tr>
<td>$w = 40$</td>
<td>-0.41</td>
<td>55.0$^\circ$</td>
</tr>
<tr>
<td>$w = 45$</td>
<td>2.23</td>
<td>49.6$^\circ$</td>
</tr>
<tr>
<td>$w = 50$</td>
<td>2.51</td>
<td>44.8$^\circ$</td>
</tr>
</tbody>
</table>

The cohesion parameter shall be disregarded in the analysis. The justification for this is the same as in Experiment A. The experimental result can then be illustrated by Figure 6B.7. The efficiency factor which has been introduced in Experiment A (Equation 6A.2) was calculated for each type of composite. The results are tabulated in Table 6B.3.

Table B3  Efficiency Factors for Various Samples

<table>
<thead>
<tr>
<th>Width of strip (mm)</th>
<th>efficiency factor $e_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td>1.00</td>
</tr>
<tr>
<td>30</td>
<td>1.06</td>
</tr>
<tr>
<td>35</td>
<td>1.55</td>
</tr>
<tr>
<td>40</td>
<td>1.64</td>
</tr>
<tr>
<td>45</td>
<td>1.35</td>
</tr>
<tr>
<td>50</td>
<td>1.14</td>
</tr>
</tbody>
</table>

It can be seen that the composite with 40mm strips is 64% stronger than unreinforced sand and that a composite
with wider strips of 50mm is only 14% stronger than unreinforced sand. The efficiency versus strip width is plotted in Figure 6B.8. It can be seen that there is an optimum strip width which produces the strongest composite. This trend is similar to that observed for 'narrow' strips. This behaviour deviates from prediction of the increase in composite strength with an increase in axial stiffness of reinforcement and an increase in reinforcement concentration.

The observation of an optimum width implies that there is a width of reinforcement strip at which the soil reinforcement interaction in terms of adherence is strongest. It may be suggested that, after the optimum width is reached, a further increase in the width of the strip would result in formation of slip planes; this can be supported by the fact that the soil-reinforcement friction coefficient is less than soil-soil friction coefficient.

6B.4 Conclusion

The presence of reinforcement enhances the strength of reinforcement and the strength enhancement is not directly proportional to the concentration of reinforcement.

The presence of reinforcement increases the stiffness of the sand and the relative deformation at peak shear resistance increases with increases in stiffness of the composite.

There is an optimum width of reinforcement strips
Figure 6B.7 Experimental results showing Coulomb failure envelopes

Figure 6B.8 Effect of strip width on strength of soil
which result in a composite with the highest strength. This behaviour is similar to sand reinforced with a narrow strip.

Experiment A and Experiment B have shown that the reinforcement width has an effect on the strength of reinforced sand. The strips can be categorized as 'narrow' strips and 'wide' strips and for each category of strips, there is an optimum width that produces a composite with the highest strength.
SECTION 6C
EXPERIMENT C
Effect of Reinforcement Type on Behaviour of Reinforced Soil

6C.1 Aim

The aim of this experiment is to determine the effect of reinforcement strip type on the behaviour of reinforced soil. Two types of reinforcement were tested: smooth strips and ribbed strips.

6C.2 Introduction

It has been found from pull-out tests that the apparent friction coefficient of ribbed strip is larger than that of smooth strips. As can be seen from Figure 6C.1, the limiting value for ribbed strips is close to the friction coefficient at the soil-reinforcement interface, as obtained from a direct shear test (Schlosser and Elias, 1978). The limiting value of the apparent friction coefficient for ribbed strips is close to friction coefficient, \( \tan \phi \), of the soil. The figure also illustrates the fact that, generally, the friction coefficient of the soil-reinforcement is lower than the friction coefficient of soil-soil. The soil-reinforcement friction coefficient is typically \( 0.5\tan \phi \) (Smith and Pole, 1980). The apparent friction coefficient from pull-out test is generally higher than the skin friction obtained from a direct shear test because of the dilatancy phenomenon during pull-out.
Figure 6C.1 The apparent friction coefficient (after Schlosser and Elias, 1978)
Smooth reinforcement generates two dimensional sample dilation at the interface which is significantly smaller than dilatancy of the soil itself whereas the pull-out of ribbed strips produces three dimensional particle movement which results in a significant tendency of the soil at the interface to dilate. Dilatancy is an important factor in frictional resistance at the soil-reinforcement interface. The influence of density and overburden pressure on the apparent friction coefficient can be related to dilatancy (Guilloux et al., 1979; Schlosser and Elias, 1978; Juran et al., 1988).

The skin coefficient for a reinforcement material can be found from a direct shear test where the reinforcement material is mounted on a block which occupies the lower part of the shear box and soil fills the other half of the box, (Potyondy, 1961; Finlay et al., 1984). There have been doubts over the use of the friction coefficient obtained from this method since the test does not simulate the actual situation in a reinforced earth wall. Nevertheless, in some methods of a reinforced earth wall design, the friction coefficient obtained from such tests is used as the limiting value.

The advantage of the Direct Shear Apparatus used by the author is that the reinforced samples simulates the reinforced earth wall through the configuration of the reinforcement strips in the shear box (Figure 6C.2). The lateral (x axis) spacing of the strips in the box is analogous to vertical spacing $S_v$ of strips in a
Figure 6C.2  Configuration of reinforcement strips in shear box (plan view)
reinforced earth wall while the longitudinal (z axis) spacing of strips in the box is analogous to the horizontal spacing $s_h$, of strips in a reinforced earth wall. The shearing plane of the Direct Shear Apparatus may be viewed as the failure surface of a reinforced earth wall. By using this system the test shows the global behaviour of reinforced earth when reinforced with different type of reinforcement strip. The system also allows for the field conditions such as group effect and dilatancy to occur during the tests.

6C.3 Experimental Set-up

Three series of direct shear tests were performed. They were tests on:

1) unreinforced sand
2) sand reinforced with smooth strips
3) sand reinforced with ribbed strips

The tests were performed under normal confining pressure of 6.74kPa, 7.86kPa, 8.98kPa, 10.08kPa, 11.16kPa, 12.26kPa, 13.34kPa and 14.44kPa. The low confining pressure allows the strength enhancement to be characterized by an enhanced internal friction angle.

In the second and third series, the dimensions, material and arrangement of the reinforcement strips within the shear box were the same, Figure 6C.3. The only difference was that in the third series the reinforcement strips have ribs. This was achieved by glueing $0.8\text{mm}$
diameter aluminium wire across the width of the 20mm wide by 160mm long aluminium strip. The arrangement of the ribs is shown in Figure 6C.4. This arrangement followed this used by Schllosser and Elias (1978).

It should be noted that since a series of direct shear tests has been performed on unreinforced sand in Experiment A, the results from these tests shall be reproduced in this experiment.

6C.4 Test Results

6C.4.1 Force-Displacement behaviour

The shear resistance versus shear displacement is shown by Figures 6C.5, 6C.6, 6C.7, 6C.8, 6C.9, 6C.10, 6C.11 and 6C.12. Each figure shows the force-displacement behaviour of the samples reinforced with smooth strips and ribbed strips at a certain applied normal pressure. The force-displacement behaviour of unreinforced samples are as shown in Figure 6A.2 in Experiment A.

At applied normal pressure of 6.74kPa, the shear resistance for sample with smooth strips was instantly mobilized and then the rate of mobilized shear resistance decreased after 2mm shear displacement. The resistance dropped at around shear displacement of 7mm and increased again to a peak resistance at 17mm shear displacement. For sample with ribbed reinforcement, highest rate of mobilized shear resistance was between shear displacement of 2mm to 3mm. The rate then tapered off to a peak resistance at 15mm
Figure 6C.3 Arrangement of reinforcement strips in shear box

Ribbed reinforcement strip

\[ d_r = 10\text{mm} \]

Figure 6C.4 Arrangement of ribs

Rib diameter = 0.8mm
Figure 6C.7 Force-displacement behaviour of samples at 8.98kPa

Figure 6C.8 Force-displacement behaviour of samples at 10.08kPa
Applied Normal Pressure of 11.16kPa

Figure 6C.9 Force-displacement behaviour of samples at 11.16kPa

Applied Normal Pressure of 12.26kPa

Figure 6C.10 Force-displacement behaviour of samples at 12.26kPa
Figure 6C.11  Force-displacement behaviour of samples at 13.34kPa

Figure 6C.12  Force-displacement behaviour of samples at 14.44kPa
shear displacement. The sample with ribbed reinforcement have lower peak resistance than sample with smooth reinforcement.

At applied normal pressure of 7.86kPa, the force-displacement behaviour was almost identical for both types of sample. Maximum rate of mobilized shear resistance was at shear displacement of 1mm to 4mm and the peak resistance was at around 18mm shear displacement. The sample with smooth strips had a higher peak shear resistance.

At applied pressure of 8.98kPa, for sample with smooth strips, maximum rate of mobilized shear resistance was between 1mm to 3mm shear displacement after which the rate tapered off to a peak resistance at 17mm shear displacement. As for a sample with ribbed strips, it can be seen that the rate of mobilized shear resistance started to taper off after 3mm shear displacement. The peak shear resistance, which was lower than for a sample with smooth strips, was at shear displacement of 20mm.

At applied pressure of 10.08kPa, the force-displacement behaviour of both type of samples is almost identical with the sample reinforced by ribbed strips having a higher peak shear resistance.

At applied normal pressure of 11.16kPa, the force-displacement behaviour of the both type of samples were similar where the rate of mmobilized shear resistance
was highest at a very small shear displacement and the rate quickly tapered off. The peak shear resistance for a sample with smooth strips was at 15mm shear displacement. The peak resistance for a sample with ribbed strips was higher at 20mm shear displacement.

The force-displacement curve at applied normal pressure of 12.26kPa for both types of sample show that the rate of mobilized shear displacement was highest at a small shear displacement and the rate tapered off to a peak shear resistance. A sample with ribbed strips has a higher peak shear resistance than a sample with smooth strips.

At applied normal pressure of 13.34kPa, the rate of mobilized shear resistance for a sample with smooth strips was high at small shear displacement of 1mm to 2mm. The rate then decreased and remained constant to the peak resistance at 19mm shear displacement. As for a sample with ribbed strips the rate was high up to 2mm shear displacement after which it tapered off to a peak shear resistance at 28mm shear displacement. The peak shear resistance for a sample with ribbed strips was higher than for a sample with smooth strips.

At applied normal pressure of 14.44kPa, the rate of mobilized shear resistance for a sample with smooth strips was highest between shear displacement of 1mm to 2.5mm, after which the rate decreased. At 13mm displacement, the resistance dropped but increased again to a peak shear resistance at 19mm shear displacement. The shear rate for a
sample with ribbed strips was highest from 2.5mm to 4mm shear displacement. It then tapered off to a peak resistance at 21mm shear displacement. The peak shear displacement for a sample with ribbed strips is higher than for a sample with smooth strips.

In general, it can be seen that for both types of sample the rate of mobilized shear resistance was highest between shear displacement of 1mm to 3mm, which is 0.3% to 1.0% of the sample size (300mm), and the peak shear resistance was achieved at around 20mm shear displacement, which is 6.7% of the sample size. At lower applied normal pressure, the peak shear resistance for a sample with smooth strips was higher than for a sample with ribbed strips but at higher applied normal pressures the peak shear resistance for samples with smooth strips was lower than for samples with ribbed strips.

In order to compare the stiffness of the samples, the average tangent moduli were taken between shear displacement of 5mm to 15mm for every applied normal pressure. The tangent moduli are tabulated in Table 6C.1.
It can be generalised that the stiffness of both types of sample increases with the applied confining pressure. This was expected since greater confining pressure results in greater interlocking of the sand particle (Lambe and Whitman, 1979). The tangent modulus of each type of sample, at a given applied normal pressure, were approximately the same thus it can be said that the stiffness of a composite with smooth strips was the same as the stiffness of a composite with ribbed strips.

Experiment A has shown that the tangent modulus of unreinforced sand at an applied confining pressure of 14.44 kPa is 51 N/mm as compared to 70 N/mm for a composite with smooth strips and 72 N/mm for a composite with ribbed strips, thus it can be said that the both types of reinforced material are stiffer than the unreinforced sand.

An additional conclusion can be found by comparing the force-displacement relationships, at applied normal
pressure of 14.44kPa, for a composite with smooth strips (Figure 6C.12) and those for samples reinforced with 20mm wide smooth strips (Figure 6A.5) in Experiment A. In both experiments, the dimension and type of reinforcement were the same however the number of strips were different: in Experiment A the sample was reinforced with nine strips while in the experiment illustrated by Figure 6C.12 the sample was reinforced with six strips. The sample with six strips had a tangent modulus of 70 N/mm and the tangent modulus for sample with nine strip was 57 N/mm; the peak shearing resistance of the sample with nine strips was 1513N at shear displacement of 26mm and the peak shearing resistance of the sample with six strips was 1539N at shear displacement of 19mm. Thus, it may be suggested that increasing the amount of reinforcement does not necessarily result in a stiffer and stronger composite.

6C.4.2 Shear Strength Parameters

The peak shearing resistance under each of the applied vertical pressures was used to obtain the shear strength parameters, c and $\phi_r$, through plots of shear stress versus normal stress. The plots are shown in Appendix T. Regression analyses were performed and the results are tabulated in Table C6.2.
performed and the results are tabulated in Table C6.2

<table>
<thead>
<tr>
<th>Strip type</th>
<th>$c$ (kPa)</th>
<th>$\phi_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td>1.64</td>
<td>$\phi = 41^0$</td>
</tr>
<tr>
<td>smooth</td>
<td>2.01</td>
<td>$46.3^0$</td>
</tr>
<tr>
<td>ribbed</td>
<td>-2.93</td>
<td>$57.1^0$</td>
</tr>
</tbody>
</table>

6C.5 Analysis of Results

The values of the apparent cohesion were disregarded in the analysis. The justification for this is the same as in Experiment A. The experimental results can then be illustrated by Figure 6C.13. The results show that the internal friction angle of the composite with smooth strips was 1.13 times that of the unreinforced sand and the internal friction angle of a composite with ribbed strips was 1.39 times that of unreinforced sand.

The efficiency factor, $e_f$, which has been defined in Experiment A was calculated for each type of sample and the result is tabulated in Table 6C.3. It can be seen that the composite with smooth strips had 20% more strength than unreinforced sand and the composite with ribbed strips was 78% stronger than unreinforced sand. By comparing the efficiency factors, it can be seen that the composite with ribbed strips was 48% stronger than the composite with smooth strips.
Figure 6C.13 Experimental results showing Coulomb failure envelopes
Table 6C.3 Efficiency Factors

<table>
<thead>
<tr>
<th>Strip type</th>
<th>efficiency factor $e_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>unreinforced sand</td>
<td>1.00</td>
</tr>
<tr>
<td>smooth</td>
<td>1.20</td>
</tr>
<tr>
<td>ribbed</td>
<td>1.78</td>
</tr>
</tbody>
</table>

6C.6 Conclusion

The force-displacement curves show that stiffness of composites with smooth and ribbed reinforcement strips are the same and these composites are stiffer than unreinforced sand.

The tests have shown that both types of reinforcement strips are effective in strengthening the sand and that ribbed strips are more efficient. Since in both series of tests, the rigidity of the reinforcement strips were the same, the test results also prove that the strength enhancement of reinforced soil is due to the friction mechanism and not due to the rigidity of the reinforcement.

Previous researchers have shown that ribbed strips have higher adherence property over smooth strips. The work presented in this section quantifies the strength enhancement due to this better adherence property by finding that sand reinforced with ribbed strips is nearly 50% stronger than sand reinforced with smooth strips.
SECTION 6D
EXPERIMENT D

The Effect of Reinforcement in Submerged Soil

6D.1 Aim

The aim of this investigation is to determine the behaviour of reinforced soil submerged in water. Four series of direct shear tests were performed using the Direct Shear Apparatus. They were tests on:

1) unreinforced dry sand
2) reinforced dry sand
3) unreinforced submerged sand
4) reinforced submerged sand

6D.2 Introduction

Reinforced earth has been used as marine structures or as dams. In these kind of structures reinforced earth is subjected to water. John (1986) has instrumented reinforced earth walls in tidal condition, where the wall was reinforced with geotextile (polymer). So far the behaviour of conventional reinforced earth wall (one that is reinforced with metallic strips) when submerged in water or exposed to variation in water table has not been documented.

In designing a reinforced earth wall, the frictional resistance against pull-out for strip at a given depth h measured from the wall crest is given as:
The tensile force that can be mobilized in the reinforcement strip, which determines the strength of the reinforced soil, is limited by the frictional resistance of the reinforcement. Therefore from the Equation 6D.1, there are two parameters that have to be considered in a submerged reinforced earth wall: $\gamma$ and $f$.

In a submerged wall, the bulk unit weight is replaced by a bouyant unit weight, $\gamma'$. The effect of water on the soil-reinforcement coefficient $f$ have not been clearly defined. The French Ministry of Transport (1980) in their recommendation for the design of reinforced earth wall, has specified that the test to obtain the soil-reinforcement coefficient should be conducted using saturated soil and done under rapid shearing, which is probably why marine structures to date are safe.

By evaluating Potyondy's (1961) results on the skin friction of various materials interacting with soil it can be seen that the skin friction obtained when the sand is saturated is less than when the sand is dry (Table 6D.1). It should be noted that these results were obtained using a
Table 6D.1 Direct shear test results of skin friction for sand
(after Potyondy, 1961)

<table>
<thead>
<tr>
<th>Material</th>
<th>Applied normal pressure</th>
<th>47.9 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth steel</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 24°10'</td>
</tr>
<tr>
<td>Rough steel</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 34°00'</td>
</tr>
<tr>
<td>Smooth concrete</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 39°30'</td>
</tr>
<tr>
<td>Rough concrete</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 44°00'</td>
</tr>
<tr>
<td>Wood parallel to grain</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 35°00'</td>
</tr>
<tr>
<td>Wood at right angles to grain</td>
<td>$\phi$ 44°30'</td>
<td>$\psi$ 39°00'</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Applied normal pressure</th>
<th>47.9 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth steel</td>
<td>$\phi$ 39°00'</td>
<td>$\psi$ 24°50'</td>
</tr>
<tr>
<td>Rough steel</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Smooth concrete</td>
<td>$\phi$ 39°00'</td>
<td>$\psi$ 34°40'</td>
</tr>
<tr>
<td>Rough concrete</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Wood parallel to grain</td>
<td>$\phi$ 39°00'</td>
<td>$\psi$ 33°20'</td>
</tr>
<tr>
<td>Wood at right angles to grain</td>
<td>$\phi$ 39°00'</td>
<td>$\psi$ 34°30'</td>
</tr>
</tbody>
</table>

$\phi$ internal friction angle of sand
$\psi$ mobilized friction angle at the sand-material interface
direct shear test where one side of the central shear plane is flushed with the material to be tested. This kind of test does not simulate the actual conditions in the field. Therefore, it was thought that by running a direct shear test using a sample that resemble the reinforced earth wall at failure, the behaviour of submerged reinforced soil could be determined.

The focus of this investigation was on the determination of the strength of reinforced soil submerged in water at a unit cell level. It was a comparative test where the strength of the submerged reinforced sand was compared to the strength of dry reinforced sand. The investigation was designed so that the effectiveness of the reinforcement when it was placed in soil submerged in water can be quantified.

6D.3 Experimental Set-up

The series of direct shear tests were performed using the Direct Shear Apparatus. To allow the sample to be submerged in water and to specify the water table, two holes 20mm in diameter were drilled on two sides of the top half of the internal shear box. These holes were covered with cloth to keep the sand in place. The reinforced sand samples were prepared according to the same procedures used in the previous investigations with the exception that before the upper plate was put in place and the loading system applied, water was poured into the external shear box to submerge the sand sample. Water permeated into the
samples through the crevices at the bottom of the internal shear box and through the holes.

Ideally, the samples should be fully submerged to allow maximum homogeneity of the sample tested. The level of the water table was governed by the depth of the external box which means that 35mm of the top of the sample tested was above the water line. This was however offset by water permeating up to the top of the samples through capillary action. The height of capillary rise was (after Terzaghi and Peck, 1967):

\[
h_{cr} = \frac{C_{cr}}{eD_{10}}
\]

\[
= \frac{0.1}{0.63 \times 0.02}
\]

\[
= 7.9 \text{cm}
\]

where \(C_{cr}\) is an empirical constant ranging between 0.1cm\(^2\) and 0.5cm\(^2\)

\(D_{10}\) is Allen Hazen's effective size

\(e\) is void ratio

Since the conservative value of the height of the capillary rise was found to be 79mm, the top 35mm of the sample was fully saturated (although not submerged) before the samples were sheared (see Figure 6D.1). The capillary phenomenon manifested itself through the top surface of the sample being wet. The amount of water used was 10 litres and the time taken for the samples to be submerged and the
top bit to be fully saturated was 20 minutes. The two phases of the samples: 135mm fully submerged and 35mm fully saturated did create a discontinuity but since the sample was large and only a small percentage of it was not submerged it would be assumed that this would not affect the results. Furthermore, the shearing plane was fully submerged.

The test was an unconsolidated-undrained test. In all four series, the applied normal pressures and the shearing rate were the same. The applied normal pressures were 6.74kPa, 7.96kPa, 8.98kPa, 10.08kPa, 11.16kPa, 12.26kPa, 13.34kPa and 14.44kPa. The shearing rate was 49.7mm/min.

The objective of the test was to find the shear strength parameters and the force-displacement behaviour of the samples. In Series 2 and Series 4, the dimensions, type and the arrangement of the reinforcement strips within the shear box were the same, Figure 6D.2. Since the direct shear test on dry unreinforced sand had been conducted in Experiment A, the results are reproduced in the following section.

6D.4 Test Results

6D.4.1 Force-Displacement behaviour

Figures 6D.3, 6D.4 and 6D.5 show the force-displacement curves for dry reinforced sand, submerged reinforced sand and submerged unreinforced sand
Figure 6D.1 Schematic diagram of sample showing water level

Figure 6D.2 Arrangement of reinforcement strips within shear box
Dry Reinforced Sand at Various Applied Pressures

Figure 6D.3 Force-displacement behaviour of dry reinforced sand

Submerged Reinforced Sand at Various Applied Pressures

Figure 6D.4 Force-displacement behaviour of submerged reinforced sand
Figure 6D.5  Force-displacement behaviour of submerged unreinforced sand
at various applied normal pressures. It can be seen that the force-displacement behaviour for these samples was similar, with the rate of mobilized shear resistance highest after initial shear displacement. The rate then decreased and tapered off to the peak shearing resistance. The peak shear resistance was obtained at shear displacement of 15mm to 20mm (5% and 6.7% of the sample size, respectively).

The force-displacement curves in Figures 6D.3 and 6D.4 show that, at applied normal pressure of 6.74kPa, the dry reinforced sample attained a peak shear resistance at shear displacement of 17mm. The submerged reinforced sand attained peak shear resistance at 16mm displacement. At applied pressure of 8.98kPa, the dry reinforced sand attained peak resistance at shear displacement of 17mm and the submerged reinforced sand attained peak resistance at 15mm displacement. At applied pressure of 14.44kPa, the dry reinforced sand attained peak resistance at shear displacement of 19mm and the submerged reinforced sand attained peak resistance at 20mm displacement. It can be seen that at a specified applied pressure, the shear displacement at peak shear resistance of submerged reinforced sand and dry reinforced were approximately the same.

In order to compare the stiffness of dry reinforced sand and submerged reinforced sand, the average tangent modulus between shear displacement of 5mm to 15mm was derived from the force-displacement curves. The tangent
modulus at various applied pressures is tabulated in Table 6D.2.

Table 6D.2  Tangent Modulus

<table>
<thead>
<tr>
<th>Applied normal pressure (kPa)</th>
<th>Tangent Modulus (N/mm)</th>
<th>Dry sample</th>
<th>Submerged sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.74</td>
<td>24</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>8.98</td>
<td>46</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>14.44</td>
<td>70</td>
<td>68</td>
<td></td>
</tr>
</tbody>
</table>

It can be seen that the stiffness of the dry and submerged reinforced sand were the same. Some small differences in the tabulated values of tangent modulus may be attributed to experimental limitation and accuracy.

6D.4.2 Shear Strength Parameters

The peak shearing resistance at various applied normal pressure was used to plot graphs of shear stress versus normal stress, Appendix U1. The graphs were plotted in terms of total stress. Regression analyses were used to fit the data points and the shear strength parameters obtained are tabulated in Table 6D.3.
### Table 6D.3 Shear Strength Parameters

<table>
<thead>
<tr>
<th>Type of sample</th>
<th>C (kPa)</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unreinforced sand</td>
<td>1.64</td>
<td>( \phi_{\text{dry}} = 41.0^\circ )</td>
</tr>
<tr>
<td>Dry reinforced sand</td>
<td>2.01</td>
<td>( \phi_{\text{r(dry)}} = 46.3^\circ )</td>
</tr>
<tr>
<td>Submerged unreinforced sand</td>
<td>-0.97</td>
<td>( \phi_{\text{submerged}} = 46.0^\circ )</td>
</tr>
<tr>
<td>Submerged reinforced sand</td>
<td>-0.58</td>
<td>( \phi_{\text{r(submerged)}} = 52.5^\circ )</td>
</tr>
</tbody>
</table>

### 6D.5 Analysis of the Results

The apparent cohesion was disregarded in the analyses since the results were to be analysed using the enhanced internal friction angle concept. The justification for this was given in Experiment A. The experimental results can then be illustrated by Figure 6D.6.

The submerged unreinforced sand had an internal friction angle 1.14 times that of dry unreinforced sand and the submerged reinforced sand had an internal friction angle 1.13 times that of dry reinforced sand. The apparently higher internal friction angle for submerged sand could be explained by considering the pore water pressure during shearing.

The tests were performed under undrained conditions, with a high rate of shearing (49.7mm/min), and a medium...
Figure 6D.6 Experimental results showing Coulomb failure envelopes
dense soil sample. Hence, during shearing of the submerged sample, the soil dilated giving rise to negative pore water pressure thus the effective confining pressure increases had occurred. In the test of dry sand, the pore water pressure was zero thus, for a given applied normal pressure (or total pressure), the effective stress in a dry sample was smaller than in a submerged sample. Hence, the shear strength was higher in a submerged sample.

This theory of negative pore water pressure due to a high rate of shearing was proven in an additional direct shear test described in Attachment 6D. The pore water pressure at each applied confining pressures for both submerged unreinforced sand and submerged reinforced sand was backcalculated (see Appendix U2). A statistical test with significant value of 0.05 confirmed that there was no difference in the negative pore water pressure for both types of samples.

Having shown the reason why the internal friction angle of submerged samples could be higher than that of dry samples, we can now proceed to the analysis of the results obtained earlier.

Since the submerged unreinforced sand had a higher internal friction angle than dry unreinforced sand, a higher value of internal friction angle for the submerged composite could be attributed to an apparently higher value for submerged unreinforced sand. Thus, in order to determine the true effectiveness of reinforcement in
submerged sand, the efficiency factor for submerged reinforced sand was taken as:

$$e_r = \frac{\tan \phi_r(\text{submerged})}{\tan \phi(\text{submerged})}$$  \hspace{1cm} (6D.1)

The efficiency factor for dry reinforced sand is:

$$e_r = \frac{\tan \phi_r(\text{dry})}{\tan \phi(\text{dry})}$$  \hspace{1cm} (6D.2)

The efficiency factor of submerged and dry reinforced sand were found to be 1.22 and 1.20, respectively. Since the difference between the two values is very small, it can be concluded that the effectiveness of reinforcement in submerged sand is the same as in dry sand.

6D.6 Conclusion

The tests show that the stiffness of submerged reinforced sand is the same as the stiffness of dry reinforced sand and the shear deformation at peak shear resistance is the same as for dry reinforced sand. Hence it can be concluded that the deformation characteristic of submerged reinforced sand is the same as for dry reinforced sand.

For the amount, type and arrangement of reinforcement tested, the strength enhancement of submerged reinforced soil is the same as the strength enhancement of dry reinforced sand, thus it can be concluded that the
reinforcement is as effective in submerged sand as it is in dry sand.

Under conditions where negative pore pressure arises i.e. under high rate of shearing (49.7mm/min), the apparent $\phi$ in submerged sand was found to be around 14% higher than $\phi$ in dry sand.
Additional Direct Shear Test on Dry Unreinforced Soil and Submerged Unreinforced Soil

Aim

The aim of this additional test is to further investigate the possible role of negative pore water pressure resulting from a high rate of shearing.

Introduction

Previous direct shear tests done using the Direct Shear Apparatus had shown that the internal friction angle of submerged unreinforced sand was 5.8° higher than the internal friction angle of dry unreinforced sand. A similar trend was observed with submerged reinforced sand and dry reinforced sand where the internal friction angle of submerged reinforced sand was 6.2° higher than the internal friction angle of dry reinforced sand. The results are tabulated in Table 6D.3. It should be noted that it was a total stress analysis.

The higher internal friction angle for submerged samples was unexpected and in order to explain the observed behaviour, the following theory was put forward. It was theorized that during previous direct shear tests of submerged samples, the tests were undrained due to the high strain rate: 49.7mm/min. The samples were of medium density and since they dilated during shear, this gave rise to
negative pore water pressure which led to higher effective confining pressure of the sample and thus a higher shear resistance of the sample. Additional direct shear tests were performed to prove that the previous tests were undrained and that the observed behaviour was due to the undrained condition. This was achieved by utilising the fact that the internal friction angle of a submerged sand should be the same as the internal friction angle of dry sand if it was tested under drained condition.

Experimental Set-up

The concept of this additional tests was to perform the direct shear tests at a much slower rate so that drainage of the submerged sample was ensured.

The additional direct shear tests were performed exactly as the previous tests. The only difference was the lower shear rate. In these tests, the strain rate was 12mm/min and the applied confining pressures were 6.74kPa, 7.96kPa, 8.98kPa, 10.08kPa, 11.16kPa, 12.26kPa, 13.34kPa and 14.44kPa. Two series of tests were performed: direct shear tests on submerged unreinforced sand and direct shear tests on dry unreinforced sand.

Test Results

The experimental results are presented in Appendix U3 and the shear strength parameters obtained are tabulated in
Table 6D.4.

Table 6D.4 Shear Strength Parameters

<table>
<thead>
<tr>
<th>Type of sample</th>
<th>c (kPa)</th>
<th>$\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unreinforced sand</td>
<td>0.59</td>
<td>$\phi_{\text{dry}} = 42.8^0$</td>
</tr>
<tr>
<td>Submerged unreinforced sand</td>
<td>1.10</td>
<td>$\phi_{\text{submerged}} = 43.8^0$</td>
</tr>
</tbody>
</table>

In accordance with the other direct shear tests performed, only the internal friction angle in additional tests was analysed. It can be seen that the internal friction angle for dry and submerged sample were almost the same. Statistical analysis with a significance level of 0.05 confirmed that the internal friction angle of submerged and dry sand were the same at $43.3^0$. These results prove that, under low strain rate, there was drainage of the submerged sample thus there was no pore water pressure whereas in previous tests when the shearing rate was high, the test was undrained, and because the sample dilated during shear, this gave rise to negative pore water pressure.

The internal friction angle of dry unreinforced sand ($42.8^0$) obtained in these additional tests is slightly different to the internal friction angle of $41^0$ of dry unreinforced sand obtained in the previous tests. This may be due to the different shearing rate applied or systematic error introduced during the recalibration of the testing.
system. It should be pointed out that the additional tests were performed several months after the previous tests. Therefore, the Direct Shear Apparatus had to be set up all over again.

Analysis

It has been demonstrated that the previous tests, conducted at a high shearing rate had been undrained. The observed behaviour was therefore a result of this undrained condition. These results were then used to backcalculate the pore water pressure in the submerged unreinforced and reinforced samples. This is presented in Appendix U2. It can be seen that, at a given applied confining pressure, the pore water pressure was negative and the pore water pressures in submerged unreinforced and reinforced samples were almost identical. Statistical analysis with a significance level of 0.05 showed that the pore water pressure in an unreinforced sample and a reinforced sample were not significantly different. This shows that the accuracy of the tests performed were good and further supports the explanation of induced pore water pressure.

Conclusion

The additional tests have proven that the higher internal friction angle of a submerged sample obtained in previous tests was due to the fact that the sample was undrained. Considering dilatation of medium dense soil,
development of negative pore water pressure in the sample can be expected. The undrained condition was brought about by the high rate of shearing.
CHAPTER 7
REVIEW OF NUMERICAL ANALYSIS OF REINFORCED EARTH WALLS

7.1 Introduction

This chapter is concerned with the review of finite element analyses of reinforced earth walls. The concept used in the finite element analysis will be presented and several finite element analyses performed by previous researchers will also be included. The main purpose of this chapter is to show the procedure and usage of finite element analysis on reinforced earth walls so that the numerical analysis performed by the author, presented in Chapter 8, can be appreciated within these contexts.

7.2 Finite Element Method in Reinforced Earth

The technique of finite elements can be used to analyse the stress and deformation behaviour of reinforced earth walls in different ways depending on the aims and complexity of the analysis required. A finite element program can be simple or complex depending on what is required of the analysis. A numerical analysis to accurately predict the collapse height of a reinforced earth wall will involve a complex procedure; the actual construction process has to be simulated and the soil-reinforcement interaction must be modelled, taking
into account the yield criterion of the soil and the reinforcement. If stresses and deformations within a stable wall are of interest, however, a simple linear, elastic finite element method which assumes perfect adherence at the soil-reinforcement interface may prove adequate. The two distinct categories of the finite element method used in the analysis of reinforced earth are as follows:

(a) Discrete approach
(b) Composite approach

In the discrete approach the three components of a reinforced earth wall (reinforcement, soil and facing) are modelled separately, while in the composite approach the reinforced zone is treated as homogeneous with composite properties which may be derived from the individual properties of the components. The elastic modulus of the composite material is a function of the moduli of the components; and of the width, thickness and spacing (horizontal and vertical) of the reinforcement strips. The disadvantage of the composite approach is that detailed information such as the edge effect at the facing is not captured. The edge effect (Figure 7.1a) occurs due to the reinforcement having much higher deformation modulus than the soil around it. The edge effect is particularly significant for flexible facing. For rigid facing, the actual deformation at the face of the wall will be as
idealized by the composite approach (Figure 7.1b). The advantage of the discrete approach is that stress concentration due to the presence of reinforcement may be obtained, whereby the stresses and strains at the soil-reinforcement interface can be obtained directly. The disadvantage is the computational cost incurred to model each reinforcing member. Romstad et al. (1976) made a comparison between a discrete approach and a composite approach for a particular reinforced earth wall. The discrete approach used 330 rectangular elements and 253 nodes, while the composite approach used 88 rectangular elements and 108 nodes. The discrete approach required 6.38min of CPU time with a total cost of $43.70, while the composite approach required 2.0min of CPU time with a total cost of $15.13. In this respect the composite analysis is favoured in terms of economy. The disadvantage of composite analysis is that stress concentration is not captured; however Hermann and Al-Yassin (1978) pointed out that, since the proportion of reinforcement is small, stress concentration can be assumed to be relatively unimportant. On the other hand, due to the relatively wide spacing of reinforcement, the edge effect might indeed be relatively important. It should be emphasised again that the edge effect is also a function of the rigidity of the facing element; hence the accuracy of using the composite approach for reinforced earth walls, with relatively rigid facing elements is not impeded.
Figure 7.1  Edge effect (after Hermann and Al-Yassin, 1978)

Note:

- $U_c$ is composite displacement
- $\hat{U}_r$ is edge displacement predicted by composite analysis
- $U_r$ is actual edge displacement
7.2.1 The unit cell concept

The concept of a composite material can be applied to a reinforced soil which has a regular reinforcing pattern. When a small unit of the reinforced soil is isolated, this unit cell completely exhibits the composite characteristic of the material. The unit cell is shown in Figure 7.2. Since the stress-strain response of the unit cell is representative of the response of the composite material, the stiffness matrix for the unit cell can be incorporated into a finite element program to find the global stiffness matrix. The required stress-strain relationship (Romstad et al., 1976) was derived using the concept that the average stresses and strains of the unit cell at local stress state (Figure 7.3a) is equal to the average stresses and strains in the unit cell at composite stress state (Figure 7.3b). The assumptions made were: (a) there is perfect adherence at the soil reinforcement interface; and (b) since the volume of reinforcement is small, the strains of the composite are equal to the strains of the soil.

7.2.2 Reinforcement

In a reinforced earth wall, the reinforcement strips
Figure 7.2 Unit cell (after Romstad et al., 1976)
Figure 7.3 Stress state of a unit cell (after Romstad et al., 1976)
have discrete horizontal spacing; however, due to high computational cost of three dimensional analysis, finite element analysis of reinforced earth walls is usually based on the assumption of plane strain two dimensional deformation. The reinforcement is assumed to extend throughout the breadth of the wall. To justify this idealisation, an equivalent elastic modulus of the reinforcement was proposed by Al-Hussaini and Johnson (1978). Firstly, it is assumed that the major response of the strips is provided by an axial stiffness of the reinforcement strips, \( S_w \), where \( S_w \) is:

\[
S_w = \sum_{j=1}^{N} \frac{A_j E_j}{L_j} = N \frac{A_{w} E_s}{L_s}
\]

where:
- \( N \) is total number of strips in each reinforcing layer
- \( A_j \) is cross-sectional area of a reinforcement strip
- \( E_j \) is modulus of elasticity of a reinforcement strip
- \( L_j \) is length of a reinforcement strip
- \( A_{w} \) is cross-sectional area of reinforcement strip
- \( E_s \) is modulus of elasticity of reinforcement strip
- \( L_s \) is length of reinforcement strip

Secondly, the equivalent stiffness \( S_e \) of the idealised reinforcement sheet which represents each layer of reinforcement strips is taken as being:
where \( A_e \) is equivalent cross-sectional area of sheet
\( E_e \) is equivalent modulus of elasticity of sheet
\( L_e \) is equivalent length of sheet

Since \( S_w \) should be equal to \( S_e \); and \( L \) is the same as \( L_e \), the equivalent modulus of elasticity in a reinforced earth wall with strips of uniform dimensions is:

\[
E_e = N \frac{A_e w E_e}{A_e}
\]  

7.2.3 Facing Element

Once the composite properties of the reinforced soil have been determined, the global model of a reinforced earth wall may be examined. The facing can be modelled by one dimensional elements, as used by Al-Yassin and Hermann (1978). Brown (1980) did not model the facing element because he reasoned that the single element boundary which joins two adjacent reinforcement layers remains linear, and so acts in a similar manner to a rigid facing element. This reasoning will be used later in the numerical analysis.
7.3 A Review of Numerical Analysis of Reinforced Earth Walls

In order to understand and appreciate the use of the Finite Element Method as a tool to study the behaviour of reinforced earth walls, a review of numerical analysis by several investigators is presented. Both the composite and discrete approaches will be included.

Shen et al. (1976) used the composite model developed by Romstad et al. (1976) to study a full-scale reinforced earth wall built on a slope. The numerical results concerning the distribution of vertical and horizontal normal stresses, reinforcement strip forces and shear stresses were presented. The first three were compared with observed behaviour monitored by instrumentation of the wall. The numerical analysis was nonlinear, elastic where the nonlinearity of soil was taken into account by using incremental analysis and the characterization of soil modulus, proposed by Duncan and Chang (1970). These authors formulated the tangent modulus value at any stress condition as:

\[ E_t = (1 - \lambda)^2 E_i \]  \hspace{2cm} (7.4a)

\[ E_i = K_d P_{ae} \left( \frac{\sigma_3}{P_{ae}} \right)^n \]  \hspace{2cm} (7.4b)

\[ \lambda = \frac{R_f(\sigma_1 - \sigma_3)(1 - \sin\phi)}{2c \cos\phi + 2\sigma_3 \sin\phi} \]  \hspace{2cm} (7.4c)
\[ R_r = \frac{(\sigma_1 - \sigma_3)_r}{(\sigma_1 - \sigma_3)_{ult}} \]  \hspace{1cm} (7.4d)

where \( E_i \) is initial modulus

\( K_d \) is modulus number (obtained experimentally)

\( P_{atm} \) is atmospheric pressure

\( \sigma_3 \) is minor principal stress

\( \sigma_1 \) is major principal stress

\( n_i \) is pure number (obtained experimentally)

\( R_r \) is failure ratio

\( (\sigma_1 - \sigma_3)_r \) is the compressive strength or stress difference at failure

\( (\sigma_1 - \sigma_3)_{ult} \) is the asymptotic value of stress difference

\( \phi \) internal friction angle

\( c \) is cohesion

\( K_d \) and \( n_i \) are dimensionless number obtained from series of triaxial tests where the results of these tests are used to plot the graph of initial tangent modulus versus confining pressure on a log-log scale (see Figure 7.4) for examples.

Using this concept, the analysis (by Shen et al., 1976) was performed incrementally where the wall construction was simulated in stages. (Clough and Woodward (1967) have
Figure 7.4  Examples on the determination of parameters $K_d$ and $n$, where $K_d$ is the vertical axis intercept and $n$ is the gradient of the line  (after Duncan and Chang, 1970)
presented in detail the use of incremental analysis to simulate construction process and to take into account the nonlinearity of material.) The wall was 15.2m high and the length of reinforcement strips was uniform, at 13.7m throughout the depth of the wall. The wall had a berm of 3.0m. The properties of the soil and the backfill and the berm were: $\phi = 40^\circ$, $c = 43.1\text{kPa}$, and $\nu = 0.32$ while the properties of the soil in the reinforced zone were: $\phi = 39.5^\circ$, $c = 47.9\text{kPa}$, and $\nu = 0.3$. The reinforcement strips were 3mm thick and 60mm wide. The vertical spacing was 254mm and the horizontal spacing varied at 0.5m, 1.0m and 1.5m. The thickness of the facing element was 3.0mm. In the numerical analysis, the reinforced zone was assumed to be an orthotropic composite material while the backfill was considered to be isotropic. Comparison of the observed results with numerical results was made at three levels of the wall: level A at the base of the wall; level B at 3.0m above the base; and level C at 9.1m above the base.

The numerical results of the vertical normal stress are shown in Figure 7.5 as contours of vertical normal stress. The measured vertical normal stress is also included in the figure (black dots). The numerical results show that there are variations of vertical stress along any horizontal plane and a stress concentration at the toe of the wall. Near the base of the wall, the vertical normal stress is maximum at the facing. The stress near the facing
is higher than the overburden pressure which agree with the observed stress (Figure 7.6). The numerical results, however, did not show the observed stress lower than overburden stress towards the free end of the reinforcement. Both numerical and observed results show that, towards the top of the wall, the distribution of vertical stress is fairly uniform.

The numerical results of the horizontal normal stresses show that, in the backfill zone and towards the top of the wall, the soil is in tension. Shen et al. (1976) pointed out that the tensile stress is a fictitious value, since the soil could not withstand tensile stress. The calculated tensile stress implies that the stress state of the soil may have reached the active condition. This is in agreement with the observed seam cracks at the back of the wall which gave evidence to the tendency of the soil mass to move horizontally and vertically down the slope. There is a stress concentration of horizontal normal stress at the toe of the wall and the numerical analysis agrees well with the observed horizontal stress (Figure 7.7). Shen et al. (1976) reported that calculation of the ratio of the vertical normal stress over horizontal normal stress throughout the wall indicates that the 'at rest' state is prevalent in the wall.

The contours of the shear stress distribution show that there is a stress concentration at the toe of the wall
Figure 7.7  (a) Horizontal normal stress contours  
(b) Horizontal shear stress contours  
(after Shen et al., 1976)  
Note: unit in psf  
(1psf = 47.9N/m²)
Figure 7.5  Vertical normal stress contours (after Shen et al., 1976)
Note: unit in psf (pound per square feet)
(1 psf = 47.9 N/m²)

Figure 7.6  Comparison of theoretical (Finite Element Method) and observed results of vertical normal stress
(after Shen et al., 1976)
(1 ksf = 47.9 kN/m²; 1 ft = 0.305m)
and that the shear stress is very high along the base of the wall. The contours of reinforcement force distribution are shown in Figure 7.8. The computed tensile force near the base of the wall is lower near the facing than towards the free end of the strip. due to the lateral restraint provided by the berm. Measurements show that the strips at the bottom were actually in compression for a significant part of their length. Comparison was made between the numerical and observed results, as illustrated in Figure 7.9. It can be seen that although quantitatively the stresses are in poor agreement, the trend in the stress distribution is quite similar.

Shen et al. (1976) also made a numerical analysis on a simple, hypothetical reinforced earth wall. The finite element representation, along with the boundary condition are shown in Figure 7.10. The properties of the soil in the foundation were: $\phi = 39.5^\circ$, $c = 47.9$ kPa and $\nu = 0.3$. The properties of the soil in the backfill were: $\phi = 33^\circ$, $c = 0$ and $\nu = 0.3$ while the soil in the reinforced zone has $\phi = 39.5^\circ$, $c = 0$ and $\nu = 0.3$. The effect of the $H/L$ ratio on the stress distribution within the wall was studied. The height of the wall was fixed at 25ft (7.6m) and the $H/L$ ratios studied were 1.25 and 2.50. The vertical normal stress, horizontal normal stress and shear stress contours are shown in Figure 7.11 for both wall widths, $L = 10$ ft (3m) and $L = 20$ ft (6m).
Figure 7.8 Contours of force in reinforcement strips
(after Shen et al., 1976)
Note: unit in lb
(1 lb = 4.45 N)

Figure 7.9 Stress in reinforcement strips
(after Shen et al., 1976)
(1 ft = 0.305 m; 1 psf = 47.9 N/m²)
Figure 7.10 Finite element mesh for hypothetical reinforced earth wall (after Shen et al., 1976)

(1 ft = 0.305m)
Figure 7.11 Stress contours for walls with different length of reinforcement (after Shen et al., 1976)

*(1 ft = 0.305 m; 1 psf = 47.9 N/m²)*
A significant observation is the rapid drop in the vertical normal stress at the back of the wall (vertical interface of backfill and reinforced zone) due to difference in vertical stiffness between the backfill and the reinforced zone. This is analogous to the reinforced earth holding up the backfill, or the backfill dragging down the reinforced earth, which results in a vertical stress in the reinforced zone exceeding the overburden pressure by about 10%. The width of the wall appears to have no influence on this effect. It will be shown later by the author that, at some distance from the back of the wall, the vertical pressure distribution within the backfill is uniform as would be expected.

Observations of the horizontal normal stress contours show that on any particular horizontal plane, the horizontal stress is higher in the reinforced zone than in the backfill. Comparison showed that there is a larger drop in computed horizontal stress at the back of the 10ft (3m) wall than that of the 20ft (6m) wall. It can be seen that the back of the wall forms a transition plane thus it can be said that the horizontal stress distribution is influenced by the difference in stiffness of the reinforced earth and the backfill. The horizontal normal stresses within the reinforced zone are close to the 'at rest' condition, while those in the backfill vary from less than
'active' condition near the back of the wall to 'at rest' state at a distance far removed from the reinforced zone. Since it is unlikely that the actual stress at the back of the wall is less than the 'active' condition, Shen et al. (1976) suggested that the assumption of the backfill as an isotropic material might be invalid and that for design purposes, the active state should be used.

The shear stress contours show that there is stress concentration for both wall widths and the shear stress is high at the base of the wall. The shear stress along any particular horizontal plane is higher in the narrower wall. This difference is almost negligible towards the top of the walls. The back of the wall forms a transition plane and the shear stress along this plane is affected by the wall width. Such distribution of shear stress is due to the difference in stiffness of the backfill and the reinforced zone and such a distribution actually restrains the wall from overturning.

The contours of strip forces are shown in Figure 7.12. It can be seen that the effect of the wall width on the peak forces and the distribution is not significant.

It should be pointed out that in their paper, Shen et al. (1976) often stated that the vertical stiffness of the reinforced earth is significantly larger than the lateral...
Figure 7.12 strip force contours in reinforced earth walls (after Shen et al., 1976)

Note: unit in lb

(1 lb = 4.45 N)
stiffness due to the orthotropic nature of the reinforced earth, however no explanation was given. Thus later in Chapter 8, the author will present an analysis of the stiffness of reinforced earth where in fact it was found that the lateral stiffness of soil is significantly increased by the inclusion of reinforcement while the vertical stiffness is marginally improved. This conclusion is opposite to that of Shen et al..

Al-Yassin and Hermann (1979) conducted numerical analysis of full-scale and model reinforced earth walls. An analysis of one full-scale wall using, the composite approach, is presented here to illustrate the process in the finite element analysis once the element stiffness matrix for reinforced earth (Hermann and Al-Yassin, 1978) has been developed. The wall was reinforced with galvanized steel strips with horizontal and vertical spacings of 762mm and 610mm, respectively. The wall was 3.7m high and was constructed in twelve lifts.

The finite element mesh and boundary condition of the WES (Waterways Experiment Station) wall is shown in Figure 7.13. Elements 1 to 18 are continuum element representing existing soil foundation and elements 19 to 60 are composite elements representing reinforced earth. The facing elements are represented by one dimensional elements, 61 to 66. The six layer of the reinforced earth
Figure 7.13 Finite element mesh for WES wall (after Al-Yassin and Hermann, 1979)
wall were placed in separate construction increments. Since the wall was built against an existing vertical soil face, the boundary condition at the back of the wall was modelled by introducing a frictional-cohesion interface between elements 11-60 and an assumed rigid surface. The nonlinearity of soil used as fill and for foundation was taken into account by using Duncan's characterization (Duncan and Chang, 1970). The relevant parameters assumed for the sand fill were: $n_t = 0.5$; $K_d = 580$; $R_f = 0.85$; $\phi = 36^0$; and $c = 0$. The unit weight of sand was $16\text{kN/m}^3$ and a constant bulk modulus of $8.28\text{MPa}$ was chosen to give a Poisson's ratio of approximately $0.3$. Properties of the reinforcement were: elastic modulus of $2.1\times10^8 \text{kN/m}^2$; plastic modulus of $13.7\times10^5 \text{kN/m}^2$; and yield stress of $3.5\times10^5 \text{kN/m}^2$. Since the model of the reinforced earth allows slippage between the reinforcement and the soil, the bond at the soil-reinforcement interface had to be considered. The friction coefficient at the soil-reinforcement interface was $0.32$ and stiffness of fictitious springs, uniformly distributed along the length of the strip was taken as $0.018/\text{mm}$. The friction coefficient was used to calculate the bond stress while the springs allow slippage to occur (Hermann and Al-Yassin, 1978).

Twelve solution increments were used in the analysis: the first one was to initialise the stress state in the
foundation; six increments were used to construct the wall and the final five were used to surcharge the wall. The finite element results of the strip forces is compared with actual observation (Figure 7.14). The analyses were unable to predict failure even though a surcharge of 50% more than the observed failure load was applied. The authors speculated that this may be due to invalid soil characterization for failure conditions and/or that the strength characterization for the connection between the facing element and the reinforcement strip was conservative.

Work by Valliappan et al. (1978) is presented here to compare results of linear and nonlinear numerical analysis of a reinforced earth model wall. The linear analysis was elastic while the nonlinear analysis was elastic-plastic. The authors analysed hypothetical walls using the discrete approach. Continuum elements were used to model the soil and line elements (one dimensional) were used to model the reinforcement. The facing elements that exist in the actual reinforced earth wall, however, were not modelled by the authors. The soil was assumed to be isotropic with limited tensile strength. For the elastic-plastic analysis, Drucker's modified von Mises yield criterion was used. The finite element mesh and the boundary condition of the model is shown in Figure 7.15. The wall was assumed to be reinforced with plastic
Figure 7.14 Tensile stress distribution along strips - WES wall (after Al-Yassin and Hermann, 1979)

($1 \text{ ft} = 0.305 \text{ m}; 1 \text{ ksi} = 6.9 \text{ MPa}$)
reinforcement (polymer material) which extend throughout the width of the wall. Slip at the soil-reinforcement interface was allowed to happen by introducing joint elements as used by Goodman et al. (1968).

For the linear analysis, the total dead load of the wall was applied in a single increment of 1g while for the nonlinear analysis, the total dead load of the wall was applied in increments of 0.25g. This method was used to take into account nonlinearity, however it does not simulate construction process of the wall.

The results of the linear and nonlinear analyses are shown in Figures 7.16 to 7.21. Results on the force and stresses are for the vertical and horizontal sections indicated in Figure 7.15. The deformation of the wall (Figure 7.16) has a similar trend for both linear and nonlinear analysis with the latter predicting slightly larger vertical and horizontal deformations. Vertical stresses along horizontal Section 1-1 (near the top of the wall) and Section 2-2 (near the base) are shown in Figure 7.17. Results of vertical stress along the base for linear and nonlinear analyses coincide. The vertical stress distributions along vertical Sections 1-1 and 2-2 are shown in Figure 7.18. The horizontal stress along horizontal Sections 1-1 and 2-2 is shown in Figure 7.19. The nonlinear analysis predicts higher lateral stress near the base than
Figure 7.15 Finite element mesh for reinforced earth model wall (after Valliappan et al., 1978)
(1 in = 25.4 mm)

Figure 7.16 Deformation of the reinforced earth model wall (after Valliappan et al., 1978)
Figure 7.17 Vertical stress along horizontal sections (after Valliappan et al., 1978)

Note: Horizontal Sections 1 - 1 and 2 - 2 shown in Figure 7.15

(1 in = 25.4mm; 1 psi = 6.9 kN/m²)
Figure 7.18  Vertical stress along vertical sections (after Valliappan et al., 1978)

Note: Vertical Sections 1 - 1 and 2 - 2 shown in Figure 7.15
(1 in = 25.4 mm; 1 psi = 6.9 kN/m²)

Figure 7.19  Horizontal stress along horizontal sections (after Valliappan et al., 1978)
(1 in = 25.4 mm; 1 psi = 6.9 kN/m²)
The linear analysis. The horizontal stress along vertical Sections 1-1 and 2-2 are shown in Figure 7.20. The strip forces at vertical Sections 1-1 and 2-2 is shown in Figure 7.21. As can be seen from the figures, the results from both the linear and nonlinear analyses have similar trends. In general, the difference in quantitative value is small and in most respects, these analyses predict the same results.

A work by Brown (1980) and Brown and Poulos (1984) is included in this review since it includes plastic analysis where this analysis takes into account failure within the soil and failure due to slippage at the soil-reinforcement interface, thus making it capable of predicting the failure of the wall. The soil is represented as an elasto-plastic material obeying a Mohr-Coulomb failure criterion with a non-associated flow rule. Flow rules determine the volume changes during plastic deformation; in associated flow rule, the angle of dilatancy \( \phi \) is equal to \( \phi \) while it is not so in non-associated flow rule (Davis, 1968). The bond at the soil-reinforcement is assumed to be cohesive and/or frictional in nature where the slip behaviour is modelled by the Mohr-Coulomb failure criterion.

The finite element analysis was done on an experimental reinforced earth wall reinforced with rubber strips. The experimental wall was built by Al-Hussaini and
Figure 7.20 Horizontal stress along vertical sections (after Valliappan et al., 1978)
(1 in = 25.4 mm; 1 psi = 6.9 kN/m²)

Figure 7.21 Force in strips at various levels of the wall (after Valliappan et al., 1978)
(1 in = 25.4 mm; 1 lb = 4.45 N)
Perry (1978) with an intended height of 3.66m. The analysis predicted a failure height of 5.0ft (1.52m) while actual failure height was 9.0ft (2.75m). Examination of the collapsed wall revealed that the wall did not fail through the tensile rupture of strips or adherence failure modes, which implies that there was a third failure mechanism. The finite element analysis showed that a plastic wedge had developed at the back of the facing just after the second construction lift was added (Figure 7.22), which resulted in reduced stiffness of the wall. This phenomenon was shown through the large lateral deformation of the facing (Figure 7.23), with the point when the plastic wedge developed marked on the curve. At collapse, the adherence failure of the strips was limited to the vicinity of the wall face and just extending beyond the plastic region which means that there was a considerable amount of reinforcement that remains intact; and since the computed tensile stress of the reinforcement was much less than the ultimate strength of the rubber, the finite element analysis shows that failure mechanism other then tensile and adherence failure had occurred. The analysis indicate that failure occurred due to large deformation of the rubber reinforcement (due to low stiffness) which had reduced the restraint on the plastic wedge. This failure mechanism is in agreement with a conclusion reached by the researchers who tested the actual wall - Al-Hussaini and Perry (1978), where they suggested that a reinforced earth wall may fail due to
Figure 7.22 Development of failure of a reinforced earth wall as it was being constructed (after Brown and Poulos, 1984)

Figure 7.23 Theoretical Wall height - Deflection curve for a reinforced earth wall (after Brown and Poulos, 1984) 

(1 ft = 0.305m, 1 in = 25.4mm)
excessive deformation of the reinforcing strips, even though the stability against tensile and adherence failure is reasonably assured.
CHAPTER 8
NEW NUMERICAL STUDIES FOR REINFORCED EARTH WALLS
(with particular reference to model walls)

8.1 Introduction

Several studies were performed to understand the behaviour of reinforced earth walls with particular reference to the model tests performed as part of this thesis (see Chapters 4 and 5). The composite approach was used and, as can be seen further in the chapter, these studies are different from any other numerical analysis that has been done before. The new approaches used in this chapter were:

1) assumption of isotropy for both reinforced zone (reinforced earth wall) and the unreinforced zone (backfill) and differentiating the reinforced earth wall and the backfill by giving arbitrary relative stiffness between the two,

2) calculation of the anisotropic properties of the reinforced earth wall using a published theory on the elastic properties of reinforced earth,

3) assumption of: i) elastic modulus of soil constant with depth and ii) elastic modulus of soil varying linearly with
depth in performing numerical analysis of reinforced earth model wall

4) quantification of the effect of the backfill thrust on the vertical pressure distribution at the base of a reinforced earth wall by applying lateral forces to the nodes at the vertical boundary of a reinforced earth block, where this boundary is analogous to the back of a reinforced earth wall.

The numerical studies are divided into two parts. Part I consider isotropic behaviour of the reinforced earth wall while Part II consider anisotropic behaviour of reinforced earth wall.

The thrust of this chapter is the numerical analysis of model reinforced earth wall based on the Finite Element Method. The objective is to analyse the vertical pressure distribution within a reinforced earth model wall and, to some extent, compare it to the observed vertical pressure distribution in the experiments presented in Chapters 4 and 5. Analysis of the deformation of the wall, lateral and shear stresses within the wall were also made. The forces in the reinforcement strips were not analysed directly but may be inferred from the lateral stresses, when the reinforcement was assumed to carry the lateral force within its tributary area. The finite element analysis was
performed on reinforced earth model wall at safe working condition where the analysis was limited to the linear elastic behaviour. Such analysis was then extended to full-scale reinforced earth wall.

8.2 Numerical Analysis of a Reinforced Earth Model Wall - Part I (Assumption of isotropic behaviour)

8.2.1 Aims of the analysis

Several numerical analyses were performed to understand the behaviour of a reinforced earth model wall built by the author. The aims of this numerical investigation were:

1) to determine the stresses and deformation within a reinforced earth model wall built on a rigid foundation, considering both the reinforced zone and the unreinforced zone.

2) to investigate the interaction of the reinforced earth wall and the backfill with particular reference to the relative stiffness of the reinforced and the unreinforced zones.

3) to compare the numerical results of the vertical pressure distribution with experimental results.
The analyses were based on the assumption of isotropic properties of both the reinforced and unreinforced zones.

8.2.2 The Finite Element Program

The finite element analysis was performed using a computer software package MICRO-CRISP version (M2) (or CRISPM2). This package was developed by Britto and Gunn (1987) for the use in general soil mechanics problems. The current numerical investigation was designed around the capability of this package; thus an offshoot of this investigation is that it will show how numerical analysis on reinforced earth wall may be performed using available resources. The capabilities of the program, as claimed by the authors, are summarised below (Britto and Gunn, 1987):

(a) Types of analysis:

Undrained, drained and consolidation analyses of two-dimensional plane strain or axisymmetric strain in solid bodies.

(b) Soil models:

Anisotropic elasticity, inhomogeneous elasticity (properties varying with depth), critical state soil models (Cam-clay, modified Cam-clay).

(c) Element types:
Linear strain triangle and cubic strain triangle.

(d) Nonlinear analysis:
Incremental (tangent stiffness) approach. The program assumes small displacements (where external load and internal stresses are assumed to be in equilibrium in relation to the original or undeformed geometry of the finite element mesh); however, there are options for updating nodal coordinates with progress of analysis.

(e) Boundary conditions:
Element sides can be given prescribed incremental values of displacements or excess pore pressure. External loading can be applied as nodal loads or pressure loading on element sides. Excavation or construction can be simulated by removing or adding an element.

(f) Miscellaneous:
Stop-restart facility allows analysis to be continued from a previous run.

Despite the claims concerning the above capabilities, the author found significant limitation concerning some aspects of the program and these were not fully resolved even after direct correspondence with one of the inventors of the Program (Britto - personal communication, 1990).
Using any finite element packages requires the user to fully understand the input requirement and to supply correct information concerning the problem being analysed. The detailed sequence of the input data for CRISPM2 can be found in Britto and Gunn (1987). In general, the data input involves specifying the geometry of the finite element mesh (element numbering and coordinates of the vertex nodes), the type of element (e.g. linear strain triangle), the type of problem (plane strain or axisymmetric), the soil model and its relevant properties, as well as the loading and the boundary conditions of the structure analysed.

8.2.3 The conditions of analysis and finite element mesh of reinforced earth model wall

Since only triangle elements are available, the current finite element investigation is confined to the composite approach where the reinforcement strips are not modelled discretely (i.e. by using one-dimensional elements). The analysis is based on the assumptions of two-dimensional plane strain deformation. The soil models used assume elastic behaviour with either 1) properties (elastic modulus) constant with depth and 2) properties (elastic modulus) varying with depth. Moreover, the elastic modulus was considered to be different in the reinforced and unreinforced zone. The loading is only due to self-weight or dead load and the analysis is linear where
the reinforced earth wall is built to full height in one increment. The justification for conducting a simple linear, elastic analysis is given below:

1) the experimental reinforced earth wall was stable at full height of 300mm. Although the actual safety factor of the reinforced earth wall is not known it is assumed to be high enough to give the wall a linear and elastic response to loading.

2) Results from Clough and Woodward (1967) and Valliappan et al. (1980) show that the trend in stresses for linear and nonlinear analysis is similar.

3) Clough and Woodward (1967) did show that results of vertical displacement from nonlinear analysis of an embankment is very different from linear analysis. However, for the current investigation and considering the capabilities of the program available, elastic analyses with constant or varying modulus were considered to be acceptable.

The finite element analysis is on a reinforced earth model wall similar to the experimental wall described in Chapter 4. The wall is 300mm high and reinforced with reinforcement strips 370mm long. The vertical spacing of the reinforcement is 50mm. The width of the backfill from
the back of the wall (vertical interface between reinforced zone and backfill) to the end of the model box is 630mm. The finite element mesh of the wall is shown in Figure 8.1. Within the reinforced zone, the mesh was arranged such that each lift represents one reinforced layer. The boundary condition at the side is assumed as shown in Figure 8.1. Since the model was built within a rigid tank with a vertical interface at the rear of the soil mass; there is no lateral displacement. The rear vertical surface of the tank was made of glass; it is assumed to be smooth, allowing vertical displacement of soil at the interface. In contrast, the boundary condition at the base of the wall is assumed to be fixed in both directions. The table on which the tank rested acted as the base for the structure; hence there is no vertical displacement along this interface. In addition to friction along the base, the 15mm thick steel base of the temporary support frame forms a berm preventing lateral movement along this basal interface. The roller joint at the vertical back boundary and fixed pin joint along the bottom horizontal boundary are both idealised boundary conditions. It should be appreciated that the accuracy of numerical modelling is dependent on how close the assumed boundary conditions are to the true boundary conditions. In this instance, it is assumed that the two are reasonably close; however this has not been verified.

The configuration is similar to the actual reinforced
**Figure 8.1** Finite Element mesh and boundary condition for reinforced earth model wall
earth wall and since the backfill is modelled as well, numerical results of stresses and deformation will include the influence of the backfill. Henceforth, this numerical analysis will address one of the main topics of this thesis: the actual vertical pressure distribution within the reinforced zone. With the configuration as shown in Figure 8.1, the effect of the backfill thrust cannot be quantified or identified, furthermore the vertical pressure distribution may be influenced by other factors besides the backfill thrust, i.e. the boundary condition. With respect to this another series of numerical investigation was devised and will be presented in another section.

8.2.3.1 Determination of parameters

It should be emphasised that, although this is a linear analysis, it is different from any other types of numerical analysis published previously. The reinforced zone and the backfill were taken as two different materials and were assumed to be isotropic. The parameter that differentiates these two zones is the stiffness. The reinforced zone has a higher stiffness due to the presence of reinforcement and this effect of reinforcement has been discussed in Chapter 2. In this analysis, the difference in stiffness between the reinforced soil (reinforced zone) and unreinforced soil (backfill) is manifested through the value of the elastic moduli of the two materials, which are
required as input data.

For the analysis, the reinforced zone was assumed to be one and a half times stiffer than the backfill. This moderately enhanced value was chosen to reflect the fact that the percentage volume of reinforcement strips in the experimental reinforced earth model wall was small. The stiffening effect of the reinforcement in the experimental reinforced earth model wall should not be overestimated. Moreover, the influence of the facing on the stiffness of the reinforced soil near its vicinity should also be considered. In reality, the facing is stiffer than the sand. Therefore, the elements at the front of the wall (elements 1 to 14) were assumed to be twice as stiff as the elements within unreinforced sand. The single element boundary that joins two adjacent reinforcement layers remains linear; thus it acts in a similar manner to a rigid facing element where, in the actual reinforced earth model wall, the facing elements (see section 4.2.4) can be considered to be relatively rigid. The relative stiffnesses are of arbitrary values (Figure 8.2). As can be seen, this type of analysis represents a new approach yet it involves simplistic concepts. The stiffness modulus of soil is, in fact, a complex issue: one of the factors that influences it is the confining pressure to which the soil is subjected. At very low stresses which are prevalent in the model walls there is little information available.
Figure 8.2  Relative stiffness between zones
concerning actual, or relative, modulus values. Within this context two series of analysis were performed using the soil models available in CRISPM2: 1) assuming the elastic modulus to be constant throughout the depth of the wall and 2) assuming the elastic modulus to vary linearly with depth.

To perform the analysis, the value of the elastic modulus of the backfill, $E_b$, and its Poisson's ratio have to be considered. Due to the small volume of reinforcement, the Poisson's ratio of reinforced sand can be assumed to be similar to that of unreinforced sand. Since the confining pressure within the reinforced earth model wall is low, (300mm high wall and unit weight of sand of 15.66kN/m$^3$) there will be a strong tendency for sand to dilate, thus a relatively high value of 0.35 seems appropriate (see Duncan and Chang, 1970). However it should be remembered that a linear 'elastic' analysis cannot simulate dilatancy, especially if $\nu \leq 0.5$.

In order to determine the appropriate value of the elastic modulus of sand backfill, $E_b$, for the sand in the reinforced earth model wall, a preliminary numerical analysis was performed. It was a trial and error analysis where the objective was to find the value of $E_b$ that would result in a maximum lateral deflection of around 2mm of the facing. In general this was the magnitude of deflection
observed in the experimental walls described in Chapters 4 and 5. The analysis, assuming a constant elastic modulus $E_b$ of 100 kPa, results in the lateral deflection as shown in Figure 7.30a. This value was, therefore, adopted for the constant modulus analysis (i.e. constant with depth but different in each zone). The values of elastic moduli for the different zones are tabulated in Table 8.1.

Table 8.1 Elastic Moduli in Different Zones for 'Constant Modulus Analysis' (modulus in each zone constant with depth)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Elastic Modulus, $E$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill</td>
<td>100</td>
</tr>
<tr>
<td>Reinforced zone</td>
<td>150</td>
</tr>
<tr>
<td>Reinforced zone: elements 1 to 14</td>
<td>200</td>
</tr>
</tbody>
</table>

In the variable modulus analysis (i.e. modulus in each zone varying with depth), the assumption of the relative stiffnesses of the different materials in the finite element mesh was similar to that of constant modulus analysis. The main difference was that the reinforced zone and the backfill zone were modelled by the second option
within the elastic soil model available in CRISPM2 which includes the linear variation of elastic modulus with depth. The Poisson’s ratios remain constant in both options. Since the facing elements used in the experiments were made of metal, there would be no variation of modulus with depth and, as such, the elements near its vicinity were still considered to have a constant elastic modulus.

For the soil model where elastic modulus varies with depth, the elastic modulus at any given depth \( y \) (Figure 8.3) is:

\[
E = E_o + m(y_o - y) \quad (8.1)
\]

where \( E_o \) is elastic modulus at the surface (\( y \) measured from the base)

\( m \) is the gradient of the linear equation

For the numerical analysis of the reinforced earth model wall, in order to define \( E_o \) and \( m \) in equation (8.1), the base of the wall is taken as datum; the elastic modulus at the top of the wall \( (y_o = 0.300) \) is assumed zero and; the level \( y \) to be considered is 0.045m from the base, since on this level the vertical stresses along this horizontal plane was measured in the experiments (Chapters 4 and 5). From equation (8.1), the elastic modulus (in kPa) at 0.045m from the base is:

\[
E_{0.045} = m (0.255) \quad (8.2)
\]
Figure 8.3 Elastic modulus varying with depth
Equation (8.2) was used in the trial and error analysis to find the appropriate value of $E_b$ which gives a maximum value of the horizontal deformation of 2mm for the facing. After several trials, a linear relationship for $E_b$ which gave a value of $E_{0.045} = 150kPa$ was adopted. The form of this relationship is $E_b = 588.24(0.3 - y)$. The corresponding relationship for the reinforced zone should give $E_{0.045}$ equal to 225kPa, in order that the ratio of $E/E_b$ at this level be equal to 1.5.

The elastic modulus for the three different zone in the finite element mesh is tabulated in Table 8.2. It should be emphasised that these values may not be the actual value of the modulus of elasticity within the experimental reinforced earth model wall since in finding these values several assumptions have been made, namely, the arbitrary relative stiffness for the different zones and assumption of isotropy. Moreover, the complete deformation pattern of the model wall was not measured.

Table 8.2 Elastic Moduli in Different Zones for 'Varying Modulus Analysis (modulus in each zones vary linearly with depth)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Elastic Modulus, $E$ at $y$ from base (kPa)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill</td>
<td>588.24($0.3 - y$)</td>
<td>150kPa at 45mm above base</td>
</tr>
<tr>
<td>Reinforced zone</td>
<td>882.35($0.3 - y$)</td>
<td>225kPa at 45mm above base</td>
</tr>
<tr>
<td>Reinforced zone: elements 1 to 14</td>
<td>300</td>
<td>constant modulus at 300kPa</td>
</tr>
</tbody>
</table>
8.3 Numerical Results

8.3.1 Constant Modulus Analysis

The parameters used for this analysis were:
\[ E_b = 100\text{kPa}; \ E_r = 150\text{kPa}; \ E_r = 200\text{kPa} \text{ and } \nu = 0.35 \]

Results of the vertical normal stress contours, horizontal normal stress contours and horizontal shear stress contours are illustrated in Figures 8.4, 8.5 and 8.6, respectively. Throughout this chapter the term 'pressure' is used synonymously with 'stress' and the distribution of stresses along the horizontal plane 45mm above the base is assumed to be representative of the stresses along the base.

The vertical normal stress contours show that there is a stress concentration at the toe of the wall, that the vertical normal stresses along any horizontal plane vary along the width of the wall especially at lower levels of the wall. At the back of the wall (vertical interface between reinforced zone and backfill) there is a distinct zone of transition where, on any horizontal plane, the vertical normal stress drops once entering the backfill zone. At the upper part of the wall and within the
Constant Modulus Analysis with $E_b = 100 \text{kPa}$
backfill, the vertical pressure distribution is fairly uniform. Within the backfill, at a distance from the back of the wall, the vertical pressure distribution is uniform throughout the depth of the structure. Comparison of Figure 8.4 with results from Shen et al. (1976) (Figure 7.4), shows that the distribution of vertical pressure is similar. One can conclude that there is a kind of arching phenomenon where along the base, the vertical normal stress within the backfill is less than within a significant part of the reinforced zone.

An additional analysis was made to investigate the distribution of vertical pressure within a hypothetical earth embankment. The geometry and boundary conditions of the embankment were assumed to be similar to the reinforced earth model wall; however facing or reinforcement were not considered. The results are shown in Figure 8.7. For the vertical normal stress it can be seen that there is a stress concentration at the toe; the vertical pressure distribution near the front boundary is not uniform except for near the top. Thus the stress concentration in reinforced earth model wall is not only due to the presence of facing or reinforcement; the vertical stress distribution close to a boundary within any earth wall must be influenced by the boundary condition. In this case, free surface at the top and at the front and a fixed bottom boundary result in stress concentration at the toe and
Figure 8.7  Vertical stress contours within unreinforced embankment (for comparison with Figure 8.4)  (unit in kPa)
variation near the front vertical plane; elsewhere the vertical stress is predictably uniform along any horizontal plane.

Returning again to the reinforced embankment, Figure 8.5 shows the horizontal normal stress contours. Within the backfill, the horizontal stress along any horizontal plane is constant however, within the reinforced zone, there is stress concentration towards the toe of the wall. A small region at the top of the reinforced zone is in tension which is not significant. More importantly, the tensile force in a reinforcement strip may be inferred from the horizontal compression stresses over its tributary area. From this figure (Fig. 8.5) it is clear that the reinforcement strips at the lower part of the reinforced wall would carry a large force relative to the strips in the upper part of the wall.

Figure 8.6 shows the horizontal shear stress contours and, once again, stress concentration at the toe of the wall is evident; there is large stress variation within the wall and the shear stress within the backfill is small.

The horizontal and vertical deformations of the facing were analysed to see the deformation of the wall. The results are shown in Figure 8.8.
Figure 8.8  Displacements of facing for Constant Modulus Analysis with $E_b = 100\text{kPa}$
8.3.2 Analyses considering Modulus Varying with depth in the reinforced zone and backfill

The parameters used in the analysis were:

\[ E_b = 588.24(0.3 - y) \]
\[ E_r = 882.35(0.3 - y) \]
\[ E_f = 300\text{kPa} \]
\[ \nu = 0.35 \]

The results are presented as stress contours; Figures 8.9, 8.10 and 8.11 are, respectively, for the vertical normal stress contours, horizontal normal stress and horizontal shear stress. As expected, these figures show a stress concentration at the toe of the wall. The vertical normal stress along any horizontal plane is not uniform: the vertical normal stresses near the facing are higher than in the rest of the reinforced zone. There is a distinct transition zone at the back of the wall where the vertical normal stress drops in the region where the reinforced zone meets the backfill zone. Within the backfill, the vertical pressure distribution is uniform throughout the depth of the structure as expected.

The horizontal normal stress contours show that, within the reinforced zone, there is a large variation in stress along any horizontal plane. At the top part of the wall and near the facing, the stresses are low. The
Figure 8.9 Vertical stress contours (Varying Modulus Analysis)

Figure 8.10 Horizontal stress contours (Varying Modulus Analysis)

Figure 8.11 Horizontal shear stress contours (Varying Modulus Analysis) (unit in kPa)
horizontal shear stress contours show that there is large variation in shear stress within the reinforced zone. Within the backfill the shear stresses are very low as expected.

The horizontal and vertical displacements of the facing are shown in Figure 8.12.

8.3.3 Comparison between Constant Modulus and Varying Modulus analyses

In order to compare the two different types of analysis, results of the vertical, horizontal and shear stresses along a particular horizontal plane were selected. The horizontal plane 45mm above the base was chosen since it is the plane instrumented in the experiments and it is close to the base of the structure. For all practical purposes, the stresses on this plane could be regarded as representatives of stresses along the base of the structure. The stresses at several points from the facing were read from the stress contours plots.

Figures 8.13, 8.14 and 8.15 show, respectively the vertical normal stress, horizontal normal stress and horizontal shear stress along the plane. It can be seen that the distribution of both normal components of stress follows similar trend for both analysis. However, shear
Figure 8.12 Displacements of facing for Varying Modulus Analysis with $E_b$ varying from 0 to 176kPa. (This may be compared with results of Constant Modulus Analysis with $E_b = 100kPa$ in Figure 8.8)
note: $L = 370$ mm

Figure 8.13 Vertical normal stress along the base

Figure 8.14 Horizontal normal stress along the base

Figure 8.15 Horizontal shear stress along the base
stress distributions are dissimilar close to the facing.

The horizontal deformation of the facing is shown in Figure 8.16. For both analyses, the maximum displacement is around 2mm. For constant modulus analysis, the maximum displacement occurs approximately at the midheight of the wall whereas for varying modulus analysis, the maximum displacement occur somewhat higher in the upper part of the wall. The vertical displacement of points along the facing is shown in Figure 8.17. The displacements of both types of analysis have similar trend where maximum displacement occurs at the top of the wall and decreases towards the base of the wall. Constant modulus analysis, however, gave larger displacement for any given point. The vertical displacements are obviously downwards movements whereas the horizontal displacements are outward (away from the reinforced zone).

8.3.4 Comparison of numerical results with experimental results

Experimental results of the vertical normal stresses from Series G in the experiment described in Chapter 5 were compared with the numerical results obtained above. In Series G, the width of the wall was 370mm and the vertical pressure along the horizontal plane 45mm from the base of the wall were measured. Nine points were gauged using
Figure 8.16  Horizontal displacement of facing for different types of analysis (these are outward movements).

Figure 8.17  Downward vertical displacement at facing.
pressure cells. The comparison between the results from numerical analysis (varying modulus analysis) and the experimental results is shown in Figure 8.18. Both results show that the vertical normal stress is highest at the facing and there is lowering of stresses in the middle of the reinforced zone. Within the reinforced zone (0 to 370mm from facing) the trend in the vertical pressure distribution is quite similar in parts but there are significant differences too. The common feature between the two is that the vertical pressure distribution is not uniform and the stress is maximum near the facing.

The main discrepancy lies in the fact that the experimental results show significantly lower stresses. Theoretically, the area underneath the experimental curve which gives the total vertical force should be the same as the area underneath the Finite Element curve. However this is obviously not the case in Figure 8.18. An explanation for this may be that arching of vertical normal stress across the reinforcement strips, in the width direction of the wall. This phenomenon was discovered through experiments described in Chapter 5. Moreover, arching across the strip has also been suggested and confirmed by Guilloux et al., 1979 (see Figure 8.19). With respect to Figure 8.19, in the author's reinforced earth wall model tests, the pressure cells were positioned below and in between two reinforcing strips, hence the pressure measured
Figure 8.18 Comparison of vertical normal stresses (at 45mm above base) from Finite Element analysis (varying modulus) with those from experimental results.

Figure 8.19 Arching phenomenon
(after Guilloux et al., 1979) (repeated from Figure 5.18)
was less than overburden.

8.4 Interaction of Reinforced Earth Wall with the Backfill
8.4.1 The aims of the analysis

For a stable reinforced earth wall which is uniformly reinforced, the stiffness of the reinforced block will be governed by the percentage volume of reinforcement. For any chosen density of reinforcement, the condition of the backfill remains the same: unreinforced and hence constant stiffness. Using this concept, parametric analysis of the interaction of the reinforced earth wall with the backfill was made. The parameter varied was the stiffness of the reinforced earth wall. The finite element mesh and boundary conditions are as shown in Figure 8.1. Four values were chosen for the relative stiffness of the reinforced earth and the backfill, (i.e. the value of the ratio $E_r/E_b$) and these were 1.5, 2, 4 and 10. The relative stiffness of the facing and the reinforced earth, $E_f/E_r$, was kept constant at 4/3. In both types of analysis, constant modulus and varying modulus were made.

8.4.2 Constant Modulus parametric analysis (effect of $E_r/E_b$)

Four series of analysis with different values of $E_r/E_b$ were performed. The parameters used in these analysis are
tabulated in Table 8.3. A Poisson's ratio of 0.35 was used for all elements in the finite element mesh. The results are shown in Figures 8.20, 8.21, 8.22 and 8.23 as stress contour plots of vertical normal stress, horizontal normal stress and horizontal shear stress for the different values of relative stiffness of reinforced earth wall and backfill. It can be seen that with increasing stiffness of the reinforced zone, there is a large drop in the vertical stress in going from the reinforced zone to the backfill. Increasing the stiffness of the reinforced zone seems to result in a development of increased shear stresses at the back of the wall.

Table 8.3 Elastic Modulus for 'Constant Modulus Analysis' (no variation with depth)

<table>
<thead>
<tr>
<th>$E_r/E_b$</th>
<th>Backfill</th>
<th>Reinforced zone</th>
<th>Reinforced zone: elements 1 to 14</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>100</td>
<td>150</td>
<td>200</td>
</tr>
<tr>
<td>2.0</td>
<td>100</td>
<td>200</td>
<td>266.7</td>
</tr>
<tr>
<td>4.0</td>
<td>100</td>
<td>400</td>
<td>533.3</td>
</tr>
<tr>
<td>10.0</td>
<td>100</td>
<td>1000</td>
<td>1333.3</td>
</tr>
</tbody>
</table>

To see more clearly the effect of the stiffness of the reinforced earth wall relative to that of the backfill, the distribution of stresses along the horizontal plane 45mm from the base of the structure is examined. The results of the vertical normal stress, horizontal normal stress and horizontal shear stress are shown in Figures 8.24, 8.25 and 8.26, respectively.
Figure 8.20 Stress contours for wall with $E_r / E_b = 1.5$ for Constant Modulus Analysis (unit in kPa)
Figure 8.21 Stress contours for wall with $E_r/E_b = 2$ for Constant Modulus Analysis (unit in kPa)
Figure 8.22 Stress contours for wall with $E_r / E_b = 4$ for Constant Modulus Analysis (unit in kPa)
Figure 8.23 Stress contours for wall with $E_r/E_b = 10$ for Constant Modulus Analysis (unit in kPa)
Constant Modulus Analysis

note: $L = 370\text{mm}$

Figure 8.24  Vertical Normal stress along the base

Figure 8.25  Horizontal normal stress along the base

Figure 8.26  Horizontal shear stress along the base
The vertical pressure distribution curves (Figure 8.24) show that the areas under all the curves are equal. This provides a check on the correctness of the numerical analysis in terms of vertical force equilibrium. It can be seen from the plots that, at low relative stiffness of the wall, the vertical stress is highest near the facing and, with increasing relative stiffness, the vertical stress is more concentrated towards the back of the wall. It can also be seen from the figure that the arching phenomenon is more pronounced with the increase in relative stiffness of the reinforced earth wall.

Figure 8.25 shows that the horizontal normal stress near the facing is low and nearly the same, irrespectively of the relative stiffness $E_r/E_b$. When the wall has low relative stiffness, the stress increases towards the back of the wall and remains nearly constant throughout the backfill. At high relative stiffness, the horizontal stress reaches a maximum near the back of the wall and then drops when entering the backfill.

Figure 8.26 shows that the horizontal shear stress is highest at the facing. At low relative stiffness, the stress decreases almost linearly with the increasing distance from the facing. At high relative stiffness ($E_r/E_b = 10$) the shear stress within the reinforced wall is less than at lower stiffnesses. The stress decreases from
the facing to the middle of the reinforced zone and then increases towards the back of the wall. The shear stress then decreases within the backfill. The results show that with increasing relative stiffness of the reinforced earth wall, the shear stresses are transferred to the backfill.

The horizontal displacement of the facing is shown in Figure 8.27. As expected, the deformation decreases with increasing relative stiffness of the reinforced earth wall. The profile of the displacement is similar for all relative stiffnesses, with maximum displacement at the height of 125mm above the base of the wall.

The vertical displacement of point along the facing is shown in Figure 8.28. The profile of deformation is similar for all relative stiffnesses with highest displacement at the top and decreasing displacement towards the base of the wall. The deformation decreases with increasing relative stiffness of the reinforced earth wall, as expected.

8.4.3 Varying Modulus parametric analysis

(effect of $E_r/E_b$)

The value of the elastic modulus used in the analysis are shown in Table 8.4. For all analyses, the value of
Figure 8.27 Outward horizontal displacement of facing for Constant Modulus Analysis

Figure 8.28 Downward vertical displacement at facing for Constant Modulus Analysis
Poisson's ratio used was 0.35. The contour plots of vertical normal stress, horizontal normal stress and horizontal shear stress are shown in Figures 8.29, 8.30, 8.31, and 8.32, respectively.

Table 8.4 Elastic Modulus for 'Varying Modulus Analysis' (Modulus varying with depth; y measured above base i.e. y = 0.3 at the surface)

<table>
<thead>
<tr>
<th>$E_r/E_b$</th>
<th>Backfill</th>
<th>Reinforced zone</th>
<th>Reinforced zone: elements 1 to 14</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>588.24(0.3 - y)</td>
<td>882.35(0.3 - y)</td>
<td>300</td>
</tr>
<tr>
<td>2.0</td>
<td>588.24(0.3 - y)</td>
<td>1176.47(0.3 - y)</td>
<td>400</td>
</tr>
<tr>
<td>4.0</td>
<td>588.24(0.3 - y)</td>
<td>2352.94(0.3 - y)</td>
<td>800</td>
</tr>
<tr>
<td>10.0</td>
<td>588.24(0.3 - y)</td>
<td>5882.35(0.3 - y)</td>
<td>2000</td>
</tr>
</tbody>
</table>

The variation of the vertical normal stress, horizontal normal stress and horizontal shear stress along a horizontal plane 45mm above the base of the structure are shown in Figures 8.33, 8.34 and 8.35. The vertical pressure distribution shows that, with increasing relative stiffness of the reinforced earth wall, vertical normal stress increases near the back of the wall.

The horizontal normal stress (Figure 8.34) is low near the facing and increases with the distance from the facing. At low relative stiffness, the stress remains fairly
Figure 8.29 Stress contours for wall with $E_r/E_b = 1.5$ for Varying Modulus Analysis (unit in kPa)
Figure 8.30 Stress contours for wall with $E_r/E_b = 2$ for Varying Modulus Analysis (unit in kPa)
Figure 8.31 Stress contours for wall with $E_r/E_b = 4$ for Varying Modulus Analysis (unit in kPa)
Figure 8.32 Stress contours for wall with $E_r / E_b = 10$ for Varying Modulus Analysis (unit in kPa)
Varying Modulus Analysis

note: $L = 370\text{mm}$

Figure 8.33 Vertical normal stress along the base

Figure 8.34 Horizontal normal stress along the base

Figure 8.35 Horizontal shear stress along the base
constant within the backfill. The horizontal stress within the reinforced zone is higher for a wall with high relative stiffness: the stresses are concentrated in the reinforced zone.

The plots of the horizontal shear stress distribution (Figure 8.35) show that the shear stress is a maximum at a distance of 100mm from the facing. The results show that, with increasing relative stiffness of the reinforced earth wall, the shear stress is increasingly transferred to the backfill.

The horizontal displacement of the facing is shown in Figure 8.36. As expected, the deformation decreases with increasing relative stiffness of the wall. The profile of deformation is the same for each analysis and the maximum displacement occurs at the height of 200mm from the base of the wall. The vertical deformation of points along the facing is shown in Figure 8.37. The deformation decreases with increasing relative stiffness. It is largest at the top and decreases towards the base.

For an additional information concerning the deformation, the horizontal and vertical displacements along a horizontal plane were considered. The plane, 200mm above the base, was chosen since at this height the maximum horizontal displacement occurred. The results for
Figure 8.36 Outward horizontal displacement of facing for Varying Modulus Analysis

Figure 8.37 Downward vertical displacement at facing for Varying Modulus Analysis
horizontal displacement are shown in Figure 8.38. It can be seen that, for all relative stiffnesses of the reinforced earth, the displacement is highest at the facing and decreases towards the end of the structure. For a very stiff reinforced earth, the point at the back of the wall has a very small displacement in the opposite direction i.e. towards the backfill. This is analogous to a localised effect such that the stiff material (reinforced earth) is pushing into the soft material (backfill) in a certain location.

Figure 8.39 shows the vertical deformation along the plane. For any given relative stiffness , the vertical displacement along most of the reinforced zone is constant and then there is a sudden increase in displacement on entering the backfill; the displacement within the backfill then remains constant.

8.4.4 Summary of the interaction of reinforced earth wall with backfill

Both constant modulus and varying modulus analyses show that the relative stiffness of the reinforced earth wall with respect to the stiffness of the backfill influences the stress distribution within the structure. With increasing stiffness of the reinforced earth, more vertical and horizontal stress is transferred to it. The
Figure 8.38 Horizontal displacement along horizontal plane, 200mm above the base (Varying Modulus Analysis)

Figure 8.39 Downward vertical displacement along horizontal plane, 200mm above the base (Varying Modulus Analysis)
shear stress within the reinforced earth however decreases with increase in stiffness of the reinforced earth. For a very stiff reinforced earth wall, the maximum vertical stress at the base is near the back of the wall thus the vertical pressure distribution is neither trapezoidal nor uniform. It can be concluded that an arching phenomenon characterises the interaction of the reinforced earth wall and the backfill.

8.5 Numerical Analysis of the Effect of Backfill Thrust on the Vertical Pressure Distribution in Reinforced Earth Model Wall

8.5.1 The aim and procedure of the analysis

The aim of this analysis is to quantify the effect of the backfill thrust on the vertical pressure distribution within the reinforced earth model wall and to compare it with the experimental results. This is done by simulation of the experiment of Series D described in Chapter 4 where backfill is placed as subsequent to a fully built reinforced earth wall. The investigation involves two numerical analysis: one analysis to determine the vertical stresses when there is no backfill and another when there is backfill behind the wall. The effect of the backfill thrust is quantified by comparing the stresses from these two analyses along a particular horizontal plane. In these
analyses, constant elastic modulus assumption was used (i.e. no variation of modulus with depth).

The finite element mesh and boundary condition for the numerical setup are shown in Figure 8.40. To simulate the backfill thrust, a triangular lateral pressure is assumed and the total force $F_x$ calculated from this triangle is distributed to nodes: 9, 10, 27, 28, 45, 46, 63 and 64. The forces $F_1$, $F_2$, $F_3$ etc. are distributed evenly to the nodes by moment consideration (Figure 8.41). For an example on how this was done, consider the two rectangular and triangle areas of the pressure diagram represented by forces $F_6$ and $F_7$. These forces were to be distributed to nodes 28 and 45. The centroid of the rectangular area is midway between nodes 28 and 45 hence each nodes is assigned a half of $F_6$. Since the centroid of the triangle is at a third of its height, from node 45, two third of $F_7$ was assigned to node 45 while a third was assigned to node 28. For completeness, the total force at node 28 was the sum of fractions of forces $F_4$, $F_5$, $F_6$ and $F_7$, while the total force at node 45 was the sum of fractions of forces $F_6$, $F_7$, $F_8$ and $F_9$. The magnitude of the force at each nodes is tabulated in Table 8.5.
Figure 8.40 Finite Element mesh configuration and the assumed lateral pressure distribution on reinforced earth model wall
Figure 8.41 Lateral force distribution to nodes

\[ F_x = \frac{1}{2} K \gamma H^2 \]
Table 8.5  Lateral Force at Nodes (given $\gamma = 15.66 \text{kN/m}^3$)

<table>
<thead>
<tr>
<th>Node</th>
<th>Lateral force (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.67 K</td>
</tr>
<tr>
<td>10</td>
<td>20.0 K</td>
</tr>
<tr>
<td>27</td>
<td>60.0 K</td>
</tr>
<tr>
<td>28</td>
<td>97.5 K</td>
</tr>
<tr>
<td>45</td>
<td>135.0 K</td>
</tr>
<tr>
<td>46</td>
<td>175.0 K</td>
</tr>
<tr>
<td>63</td>
<td>156.25 K</td>
</tr>
<tr>
<td>64</td>
<td>57.08 K</td>
</tr>
</tbody>
</table>

The analyses are based on constant elastic modulus assumptions where the value of $E_r$ is 150kPa and $E_f$ is 200kPa. The value of the Poisson's ratio is 0.35. In order to determine the value of the earth pressure coefficient $K$, preliminary trial and error analysis was performed for different values of $K$. The objective was to obtain a maximum horizontal displacement of approximately 2mm of the facing. After several runs, a value of 0.35 was adopted for $K$. The resulting horizontal deformation of the facing for the analysis of backfill thrust where $K = 0.35$ is shown in Figure 8.42.

8.5.2 Results of numerical analysis

Before the two main analyses were performed, an additional analysis was done on a hypothetical block (no reinforcement or facing), similar in geometry to the reinforced earth wall. The boundary conditions were also
Figure 8.42 Outward horizontal displacement of facing of a reinforced earth wall with lateral thrust
similar i.e. free surfaces at both vertical sides and at the top, and a fixed bottom. The elastic modulus used was 100 kPa and the Poisson's ratio was 0.35. The objective was to see the vertical pressure distribution within the block. As can be seen from the vertical normal stress contour (Figure 8.43) there is a concentration at the toes of the block. It shows that, along the base of the block, the distribution of the vertical pressure is not uniform and there is arching of stress to the sides of the block.

The contour plots of the vertical normal stress of the reinforced earth wall without the backfill thrust is shown in Figure 8.44a and, for reinforced earth wall with lateral thrust, is shown in Figure 8.44b. The vertical pressure distribution along the horizontal plane 45 mm above the base derived from the contour plots for both analysis is shown in Figure 8.45. Even without lateral thrust, the vertical pressure distribution is not uniform. It can be seen that the lateral thrust increases the vertical stress at the facing and reduces the vertical stress at the back of the wall.

The effect of the backfill thrust is analysed by calculating the increase or decrease in the vertical normal stress along the horizontal basal plane relative to stresses which are due to self-weight alone. Comparison is made with the experimental results and the trapezoid theory (Figure 8.46). Results for the trapezoid theory is
Figure 8.43 Vertical normal stress contours in an earth block under its own weight - showing effect of boundary conditions, namely stress concentration at the toes.
Figure 8.44 (a) Vertical normal stress in reinforced earth wall with no backfill thrust (the stiffness of elements along the vertical left boundary was made higher to account for facing elements) (unit in KPa)

Figure 8.44 (b) Vertical normal stress contours in reinforced earth wall with backfill thrust
Figure 8.45 Vertical normal stress along the base showing effect of backfill thrust.

Figure 8.46 Percentage change in vertical normal stress due to backfill thrust (relative to stress due to selfweight alone).
calculated assuming that the vertical pressure distribution is uniform when there is no backfill (Table 8.6). To be consistent with the numerical analysis, the value of the earth pressure coefficient used in the trapezoid theory (see Equations 3.5 and 3.6) was 0.35. It can be seen that the numerical results are close to the trapezoid theory where the backfill thrust (lateral thrust) resulted in highest increase in the normal stress at the facing and highest decrease in the normal stress near the back of the wall. Experimental results, however, show that the effect of the thrust (friction effect excluded) is most marked at two points: (a) highest decrease in normal stress at the back of the wall and (b) highest increase in normal stress within the middle of the reinforced zone. The effect of the thrust then lessens with the distance from this part towards the facing. This is the true behaviour of the reinforced earth mass in a model wall which is not captured either by the numerical analysis or by the conventional analytical method. It is, of course, not known whether this behaviour is typical of actual stresses in full-scale walls.
Table 8.6 Vertical Stress Calculated for Different Assumptions of Vertical Pressure Distribution

<table>
<thead>
<tr>
<th>Distance from facing (mm)</th>
<th>Vertical Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniform</td>
</tr>
<tr>
<td>31</td>
<td>3.99</td>
</tr>
<tr>
<td>94</td>
<td>3.99</td>
</tr>
<tr>
<td>157</td>
<td>3.99</td>
</tr>
<tr>
<td>220</td>
<td>3.99</td>
</tr>
<tr>
<td>283</td>
<td>3.99</td>
</tr>
<tr>
<td>346</td>
<td>3.99</td>
</tr>
</tbody>
</table>

8.5.3 Conclusion

The numerical investigation shows that the vertical pressure distribution at the base of a reinforced earth wall is not uniform even when there is no backfill; the maximum vertical normal stress occurs near the facing. This is in agreement with the experimental result. The non-uniformity is primarily because of boundary conditions.

The effect of the backfill thrust (lateral thrust) is to reduce the vertical pressure at the back of the wall and increase the vertical stress at the facing. The numerical results agree quite well with the trapezoidal theory. It can be concluded that if a more accurate distribution of vertical pressure is needed in the design of a reinforced
earth wall, the analysis performed above may be utilised where the use of a reinforced earth block and lateral forces minimises the number of grid elements. However, this is only valid where the actual construction procedure is similar to that assumed, namely, full construction of reinforced earth wall followed by backfilling. The gradual or incremental construction process is not accounted for here. Moreover, as pointed out above, the experimental behaviour of model wall is different although the conventional approach is confirmed by Finite Element Method approach.

8.6 Numerical Analysis of Reinforced Earth Model Wall – Part II
(Assumption of anisotropic behaviour for reinforced zone only)

8.6.1 The aim of the analysis

The aim of the analysis is to determine the effect of the strip width on stress distribution within a reinforced earth block and the adjoining backfill. It should be noted that the strip width determines the axial stiffness of the reinforcement. The numerical analysis is based on the geometry of the experimental model walls described in Chapter 5. Several series of reinforced earth model walls were built and the varying parameter was the reinforcement strip width. Since the number of reinforcement strips,
their thickness and length were kept constant in each series, the stiffness in each experiment was related to the chosen strip width (see Equation 7.1). The percentage volume of reinforcement per linear metre of the 300mm reinforced earth model wall, and the axial stiffness, $S_w$, of reinforcement per linear metre of reinforcing layer are tabulated in Table 8.7.

The effect of the width of reinforcement strips on the elastic properties of reinforced earth is incorporated into current numerical analysis based on the theory presented by Harrison and Gerrard (1972).

Table 8.7 Volume and Axial Stiffness of Reinforcement
(based on Equation 7.1)

<table>
<thead>
<tr>
<th>Series</th>
<th>Strip width (mm)</th>
<th>Percentage volume</th>
<th>Axial stiffness, $S_w$ (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>0.02%</td>
<td>2.33</td>
</tr>
<tr>
<td>B</td>
<td>13.5</td>
<td>0.027%</td>
<td>3.15</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>0.04%</td>
<td>4.67</td>
</tr>
<tr>
<td>D</td>
<td>25</td>
<td>0.05%</td>
<td>5.83</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>0.064%</td>
<td>7.47</td>
</tr>
</tbody>
</table>

8.6.1.1 Elastic properties of reinforced earth

Harrison and Gerrard (1972) formulated the equations
for elastic properties of reinforced earth based on the formulae develop by Salamon (1968) to find the equivalent elastic properties of a stratified rock mass. In developing the theory for the properties of reinforced earth the following assumptions were made:

(1) reinforced earth is a composite material consisting of alternating layers of soft (soil) and stiff (reinforcement) materials,

(2) the soft layers have elastic modulus $E_1$ and Poisson's ratio $\nu_1$, and the stiff layers have elastic modulus $E_2$ and Poisson's ratio $\nu_2$,

(3) the layers are horizontal and,

(4) each layer is isotropic and there is no relative displacement at the interface between the layers.

The relative thickness of stiffer layers is the ratio of the combined thickness of the stiff layers $S_e$, over a vertical distance $H$ and is given as:

$$ t = S_e / H $$

A constant $K_e$ is defined as:

$$ K_e = \frac{t E_2}{E_1} $$

The elastic properties of the anisotropic reinforced earth are given as:
\[ v_h = \frac{\frac{v_1}{1 - v_1^2} + K_e \frac{v_2}{1 - v_2^2}}{\frac{1}{1 - v_1^2} + K_e \frac{1}{1 - v_2^2}} \]  
(8.5)

\[ v_{hv} = (1 - v_h) \frac{v_1}{1 - v_1} \]  
(8.6)

\[ E_h = (1 - v_h^2) \left( \frac{1}{1 - v_1^2} + \frac{K_e}{1 - v_2^2} \right) E_1 \]  
(8.7)

\[ \frac{1}{E_v} = \frac{1}{E_1} \left( 1 - \frac{2v_1^2}{1 - v_1} \right) + \frac{2v_{hv}^2}{(1 - v_h)E_h} \]  
(8.8)

\[ G_h = \frac{E_h}{2(1 + v_h)} \]  
(8.9)

\[ G_v = \frac{E_v}{2(1 + v_1)} \]  
(8.10)

where \( x, y, z \) are rectangular coordinates (lateral, vertical and longitudinal, respectively)
$E_h$ is modulus of elasticity in x and z directions
$E_v$ is modulus of elasticity in y direction
$G_h$ is shear modulus applying to shear strains occurring in z-x plane
$G_v$ is shear modulus applying to shear strains occurring in x-y plane or z-y plane
$\nu_h$ is Poisson’s ratio - effect of strain in x direction on strain in z direction
$\nu_{hv}$ is Poisson’s ratio - effect of strain in x direction on strain in y direction

8.6.2 Configuration of numerical model and procedure of analysis

The experimental reinforced earth wall described in Chapter 5 is 300mm high and the length of reinforcement strips was uniform at 300mm. Each lift of reinforcement consisted of four reinforcement strips spaced at 200mm centre-to-centre. Numerical analyses were performed based on the experimental reinforced earth model walls in Series A, B, C, D and E. The finite element mesh and the boundary conditions for the reinforced earth model wall are shown in Figure 8.47.

The analyses were based on the assumption of constant elastic modulus and the reinforced zone and the backfill were considered as two different materials. Moreover, the
reinforced zone was regarded as anisotropic while the backfill was assumed to be isotropic. The presence of the facing was not modelled by using a separate element or by allocating a higher modulus to the elements along the face of the wall as had been done in the earlier work. However, since each lift of elements in the reinforced zone represented a layer of reinforcement, the single element boundary at the facing which joins two adjacent reinforcement layers remains linear and so act in a similar manner to a relatively rigid facing element in the actual reinforced earth model wall.

8.6.2.1 Input parameters

The analysis is two-dimensional plane strain and for the soil model, the program CRISPM2 requires the input for $E_h$, $E_v$, $\nu_h$, $\nu_v$, and $G_v$. The program does not require the parameter $G_h$ for the plane strain analysis. For the composite reinforced earth these values are calculated using the formulae presented above. Since both the theory and the numerical analyses assume that the reinforcement is in the form of sheets, the discrete reinforcements strips in Series A, B, C, D, and E had to be converted to an equivalent reinforcement sheet using Equation 7.3. The 0.2mm thick shim steel reinforcement strip has the modulus of elasticity of $70 \times 10^3$MPa and there are four strips per layer of reinforcement within the 0.8m long wall. The
equivalent modulus of elasticity, $E_e$ (Equation 7.3), calculated for the different strip width was tabulated and is presented in Table 8.8.

Table 8.8 Equivalent Elastic Modulus $E_e$

<table>
<thead>
<tr>
<th>Series</th>
<th>Width of strip (mm)</th>
<th>$E_e$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>3500</td>
</tr>
<tr>
<td>B</td>
<td>13.5</td>
<td>4725</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>7000</td>
</tr>
<tr>
<td>D</td>
<td>25</td>
<td>8750</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>11200</td>
</tr>
<tr>
<td>F</td>
<td>no strip</td>
<td>NA</td>
</tr>
</tbody>
</table>

The reinforcement layers have the modulus of elasticity $E_2$ equal to $E_e$ and a Poisson's ratio $\nu_2$ of 0.28. The Poisson's ratio $\nu_1$ of the sand is taken as 0.35 while the value of the modulus of elasticity $E_1$ had to be considered. Initially, a value of 1.0MPa was used since this resulted in the horizontal deformation of the facing close to 2mm (criterion used in the earlier analysis) for wall in Series A (strip width of 10mm). However, it was found from the result of the numerical analysis that the vertical normal stress is way off from that observed experimentally (see Figure 8.48). It can be seen that, at the basal plane, the vertical normal stress near the facing is tensile, in contrast to purely compressive stress measured. Also, the maximum compressive stress (20 kPa) is much larger than that observed (5 kPa) experimentally.
Moreover, the horizontal displacement obtained from the numerical analysis of the series of walls (A, B, C, D and E) show that, generally, the movement is inward which is in contrast to the outward movement observed in the experiments (see Figure 8.49). (The anisotropic properties calculated when $E_1$ is equal to 1.0MPa are presented in Table 8.9).

Table 8.9 Anisotropic Elastic Parameters  
(based on $E_1 = 1.0$MPa)

<table>
<thead>
<tr>
<th>Series</th>
<th>Width of strip (mm)</th>
<th>$K_e$ (MPa)</th>
<th>$E_h$ (MPa)</th>
<th>$E_v$ (MPa)</th>
<th>$\nu_h$</th>
<th>$\nu_{hv}$</th>
<th>$G_h$ (MPa)</th>
<th>$G_v$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>14.0</td>
<td>15.050</td>
<td>1.536</td>
<td>0.28</td>
<td>0.39</td>
<td>0.370</td>
<td>5.879</td>
</tr>
<tr>
<td>B</td>
<td>13.5</td>
<td>18.9</td>
<td>19.950</td>
<td>1.552</td>
<td>0.28</td>
<td>0.39</td>
<td>0.370</td>
<td>7.793</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>28.0</td>
<td>29.050</td>
<td>1.568</td>
<td>0.28</td>
<td>0.39</td>
<td>0.370</td>
<td>11.348</td>
</tr>
<tr>
<td>D</td>
<td>25</td>
<td>35.0</td>
<td>36.050</td>
<td>1.575</td>
<td>0.28</td>
<td>0.39</td>
<td>0.370</td>
<td>14.082</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>44.8</td>
<td>45.850</td>
<td>1.582</td>
<td>0.28</td>
<td>0.39</td>
<td>0.370</td>
<td>17.910</td>
</tr>
<tr>
<td>F</td>
<td>no strip</td>
<td>NA</td>
<td>1.0</td>
<td>1.0</td>
<td>0.35</td>
<td>0.35</td>
<td>0.370</td>
<td>0.370</td>
</tr>
</tbody>
</table>
Figure 8.48 Vertical stress contours for Series A (E_b = 1MPa) (unit in kPa)

Figure 8.49 Horizontal displacement of facing (E_b = 1MPa)
Note: - ve outwards; + ve inwards
Due to the unrealistic behaviour encountered when $E_1 = 1.0$MPa was used, for the actual numerical analysis in this section, an arbitrary value of 6.9MPa was adopted for the modulus of soil. Obviously, with this value the horizontal deformation of the facing will be much less than observed experimentally. This value has been used by Brown (1980) for reinforced earth model wall similar in size to the author's model wall. The anisotropic properties calculated for this value of $E_1$ are presented in Table 8.10. It can be seen that the value of $K_e$ is 2.03 for the least reinforced wall (Series A) and 6.49 for the most reinforced wall (Series E). It will be seen later in this chapter that $K_e$ for a typical (full-scale) reinforced earth falls within this range hence the use of $E_1$ equal to 6.9MPa justify the concept of modelling reinforced earth walls. The use of this value is further justified by the results of the numerical analyses which will be seen later, suffice to say that the results of the magnitude of the vertical normal stress is within that observed experimentally.

The input parameters ($E_v$, $E_h$, $\nu_h$, $\nu_{hv}$, and $G_v$) for the series of analysis are as in Table 8.10, where each numerical analysis is given the same label as in the experiments. Although in the experiment of Series A (where the strip width was smallest at 10mm) the wall failed at a height less than 300mm, (note that this was not the general case), the numerical analysis is made for a 300mm wall for the sake of uniformity with other series.
<table>
<thead>
<tr>
<th>Series</th>
<th>Width of strip (mm)</th>
<th>$K_e$ (MPa)</th>
<th>$E_h$ (MPa)</th>
<th>$E_v$ (MPa)</th>
<th>$\nu_{h}$</th>
<th>$\nu_{hv}$</th>
<th>$G_v$ (MPa)</th>
<th>$G_h$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>2.03</td>
<td>20.986</td>
<td>9.094</td>
<td>0.30</td>
<td>0.38</td>
<td>2.556</td>
<td>8.072</td>
</tr>
<tr>
<td>B</td>
<td>13.5</td>
<td>2.74</td>
<td>25.824</td>
<td>9.409</td>
<td>0.30</td>
<td>0.38</td>
<td>2.556</td>
<td>9.932</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>4.06</td>
<td>35.043</td>
<td>9.813</td>
<td>0.29</td>
<td>0.38</td>
<td>2.556</td>
<td>13.582</td>
</tr>
<tr>
<td>D</td>
<td>25</td>
<td>5.07</td>
<td>41.969</td>
<td>10.001</td>
<td>0.29</td>
<td>0.38</td>
<td>2.556</td>
<td>16.267</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>6.49</td>
<td>51.781</td>
<td>10.188</td>
<td>0.29</td>
<td>0.38</td>
<td>2.556</td>
<td>20.070</td>
</tr>
<tr>
<td>F</td>
<td>no strip</td>
<td>NA</td>
<td>6.9</td>
<td>6.9</td>
<td>0.35</td>
<td>0.35</td>
<td>2.556</td>
<td>2.556</td>
</tr>
</tbody>
</table>

Note: $G_h$ is not required in the input data.

It can be seen from the table that the composite material consisting of soft and stiff layers of isotropic materials has anisotropic properties. Comparison of the
properties of reinforced earth and sand alone show that the effect of reinforcement is orientation dependent where the stiffness of soil in the direction the reinforcement is laid (lateral) is greatly enhanced, up to 650% (Series E). The stiffness enhancement is dependent on the amount of reinforcement. In the direction perpendicular to the reinforcement (vertical), the effect of reinforcement on the stiffness is relatively small, with the largest increase in the stiffness of only 47.8% (Series E). The vertical shear modulus of the composite assumes that of the soft material (sand).

8.6.3 Numerical results

An example of the data input is shown in Figure 8.50 to illustrate the amount of information required to run an analysis. Results from the analysis are used to plot the contours of the vertical normal stress, horizontal normal stress and horizontal shear stress. These are shown in Figures 8.51, 8.52, 8.53, 8.54 and 8.55 for Series A, B, C, D and E respectively.

The vertical normal stress contours for Series A (strip width of 10mm) show that there is stress concentration at the toe of the wall. Along the base, the vertical normal stress is highest at the facing; it decreases and then increases again towards the back of the
WA SERIES E2 second experiment
91 144 3 2 2 0
0 0
0 0 0 0 0 0 0 0 0 0
0 0 0 0
1 0 .3
2 .035 .3
3 .105 .3
4 .175 .3
5 .245 .3
6 .3 .3
7 .3 .275
8 .245 .275
9 .175 .275
10 .105 .275
11 .035 .275
12 0 .275
13 0 .225
14 .035 .225
15 .105 .225
16 .175 .225
17 .245 .225
18 .3 .225
19 .3 .175
20 .245 .175
21 .175 .175
22 .105 .175
23 .035 .175
24 0 .175
25 0 .125
26 .035 .125
27 .105 .125
28 .175 .125
29 .245 .125
30 .3 .125
31 .3 .075
32 .245 .075
33 .175 .075
34 .105 .075
35 .035 .075
36 0 .075
37 0 .025
38 .035 .025
39 .105 .025
40 .175 .025
41 .245 .025
42 .3 .025
43 .3 .0
44 .245 .0
45 .175 .0
46 .105 .0
47 .035 .0
48 0 .0
49 .34 .3
50 .34 .275
51 .34 .225
52 .34 .175
53 .34 .125
54 .34 .075
55 .34 .025
56 .34 0.
57 .385 .3
58 .385 .275
59 .385 .225
60 .385 .175
61 .385 .125
62 .385 .075
63 .385 .025
64 .385 0.
65 .44 .3
66 .44 .25
67 .44 .2
68 .44 .15
69 .44 .1
70 .44 .05
71 .44 0.
72 .515 .3
73 .515 .2
74 .515 .1
75 .515 0.
76 .625 .3
77 .625 .2
78 .625 .1
79 .625 0.
80 .75 .3
81 .75 .2
82 .75 .1
83 .75 0.
84 .875 .3
85 .875 .2
86 .875 .1
87 .875 0.
88 1 .3
89 1 .2
90 1 .1
91 1 .0
0
1 2 1 37 48 47
2 2 1 38 37 47
3 2 1 36 37 38
4 2 1 35 36 38
5 2 1 25 36 35
6 2 1 26 25 35
7 2 1 24 25 26
8 2 1 23 24 26
9 2 1 13 24 23
10 2 1 14 13 23
11 2 1 12 13 14
12 2 1 11 12 14
13 2 1 11 12 11
14 2 1 1 1 1
15 2 1 38 47 46
16 2 1 39 38 46
17 2 1 35 38 39
18 2 1 34 35 39
19 2 1 26 35 34
20 2 1 2 26 34

Figure 8.50 An example of input data for CRISPM2
Figure 8.51 Stress contours for Series A (E_s = 6.9MPa) (unit in kPa)
Figure 8.52 Stress contours for Series B (unit in kPa)
Figure 8.53 Stress contours for Series C (unit in kPa)
Figure 8.54 Stress contours for Series D (unit in kPa)
Figure 8.55 Stress contours for Series E (unit in kPa)
wall. From the back of the wall to the backfill, the vertical stress decreases and then remain constant. At several points, the vertical stress exceeded the overburden pressure \( \gamma h \) and, at some points, the vertical stress is less than overburden. The plot shows a transition zone at the back of the wall. Within the reinforced zone the vertical pressure distribution is not uniform although towards the top of the wall the distribution is fairly uniform. The horizontal normal stress contours show that there is stress concentration at the base of the reinforced earth wall, the horizontal stress is maximum at the base and, at top of the wall, the stress is tensile. Throughout most part of the facing, the horizontal stress is small. The horizontal shear stress contours show that there is stress concentration at the toe; the stress is highest at the toe and, within the top half of the reinforced earth wall, the shear stress acts in clockwise direction while at other part the shear stress acts in anticlockwise direction. Within the backfill the shear stress is very small.

The vertical normal stress contours for Series B (strip width of 13.5mm) show that, for any horizontal plane in the reinforced earth wall, the vertical pressure distribution is not uniform. At some regions within the wall, the vertical normal stress exceeded the overburden pressure e.g. the regions with stress of 5kPa near the base
and around the middle of the wall while, at some points, the stress is less than overburden pressure. The horizontal normal stress contours show that the maximum stress is at the base of the reinforced earth wall. At the top of the wall, the stress is tensile. At most part of the facing, the horizontal normal stress at the facing is small. The horizontal shear stress contours show that the stress is maximum at the toe; within the top half of the wall, the shear stress acts in clockwise direction while within the rest of the wall, the shear acts in anticlockwise direction. The shear stress within the backfill is very small.

The vertical normal stress contours for Series C (strip width of 20mm) show that, within the reinforced earth wall, the vertical pressure distribution on any horizontal plane is not uniform; at some region the stress is larger than overburden while at some region the stress is less than overburden. The vertical pressure distribution on any horizontal plane in the backfill is uniform. The horizontal normal stress contours show that stress is highest along the base of the reinforced earth wall; at the top of the wall, the stress is tensile. The stress at most part of the facing is small. The horizontal shear stress contours show that the stress is highest at the toe; there is concentration of stress at the back of the wall. Within the top half of the wall, the shear stress acts in
The vertical normal stress contours for Series D (strip width of 25mm) show that along the base of the reinforced earth wall, the vertical pressure distribution is uniform; at some points within the wall, the stresses are less than overburden while at some points the stresses are larger than overburden. Within the backfill, the vertical pressure distribution is uniform. The horizontal normal stress contours show that the stress is maximum along the base of the reinforced earth wall; at the top and at the front of the wall, the stress is tensile. The horizontal shear stress contours show that the stress is highest at the toe; there is stress concentration at the back of the wall. Within the top front of the wall, the shear stress acts in clockwise direction. The shear stress within the backfill is small.

The vertical normal stress contours for Series E (strip width of 32mm) show that vertical pressure distribution on any horizontal plane within the reinforced earth wall is not uniform; along the base the vertical stress is highest near the back of the wall. The vertical pressure distribution within the backfill is uniform. The horizontal normal stress contours show that the stress is highest at the base of the reinforced earth wall; at the
top of the wall, the stress is tensile. The horizontal shear stress contours show that the stress is highest at the toe of the wall; at the top front of the wall the shear stress acts in clockwise direction. There is stress concentration at the back of the wall.

The horizontal deformation of the facing for each series is shown in Figure 8.56. The deformations for all series are very small thus comparison between series is qualitative in nature. It can be seen that the maximum horizontal displacement is at around 100mm from the base. The vertical displacement along the facing is shown in Figure 8.57. In Series E there is a slight upward displacement.

8.6.4 Discussion and conclusion

The width of the reinforcement strip controls the value of the relative stiffness $K_e$ which affects the value of the elastic parameters. In the numerical simulation of the reinforced earth model walls the value of $K_e$ ranged from 2.03 to 6.49. The numerical analysis show that magnitude of stresses varies between each series however the trend in stresses distributions is similar. The vertical pressure distribution within any horizontal plane in the reinforced earth wall is not uniform while, within the backfill, the vertical pressure distribution is
Figure 8.56 Outward horizontal displacement of facing

Figure 8.57 Vertical displacement along facing + ve downwards
uniform. At some point within the wall, the stress may exceed overburden while, at another point, the stress may be less than overburden. The horizontal stress is highest at the base of the wall and at the top of the wall, the horizontal stress is tensile. The shear stress is maximum at the toe and there is shear stress concentration at the back of the wall.

An important outcome of this part of work is that it shows that the stress and deformation of a reinforced earth wall may be analysed using a basic finite element program without doing any alteration to the program: this is achieved by deriving the properties of the reinforced earth according to the theory presented by Harrison and Gerard (1972).

8.7 Numerical Analysis of Full-Scale Reinforced Earth Wall and Backfill

8.7.1 The aim of the analysis

The aim of the analysis is to determine the stress and deformation of a full-scale reinforced earth using the composite approach. Two types of analysis, as used in the analysis of reinforced earth model wall, were employed: 1) assuming isotropy and giving arbitrary relative stiffness to the different zones and; 2) using the elastic parameters calculated from theory by Harrison and Gerrard (1972). In
the first type of analysis, only the constant modulus analysis was performed in order to maintain uniformity with the second type of analysis.

8.7.2 Configuration of the wall

The analysis is on a hypothetical full-scale reinforced earth wall whose dimensions were based on the dimension used by Romstad et al. (1976). The configurations of the wall are:

\[
\begin{align*}
H &= 7.5 \text{m}; \\
L &= 6 \text{m}; \\
S_v &= 250 \text{mm}; \\
S_H &= 1.2 \text{m}; \\
w &= 60 \text{mm}; \\
ts &= 3 \text{mm}
\end{align*}
\]

and the properties of its constituents (soil and reinforcement) are:

\[
\begin{align*}
\gamma &= 20 \text{kN/m}^3; \\
\nu_{sd} &= 0.30; \\
E_{sd} &= 35 \text{MPa}; \\
\nu_{st} &= 0.28; \\
E_{st} &= 200 \times 10^3 \text{MPa};
\end{align*}
\]

where 

- \( H \) is height of wall
- \( L \) is length of reinforcement strips
- \( S_v \) is vertical spacing of reinforcement strips
- \( S_H \) is horizontal spacing of reinforcement strips
- \( w \) is width of strip
- \( t_s \) is thickness of strip
- \( \gamma \) is unit weight of sand
- \( E_{sd} \) is elastic modulus of sand (value adopted from Lambe and Whitman, 1979)
- \( \nu_{sd} \) is Poisson's ratio of sand
- \( E_{st} \) is elastic modulus of steel reinforcement
\( \nu_{st} \) is Poisson's ratio of steel

The percentage volume of reinforcement per linear metre of reinforced earth wall is 0.06%. The value of \( K_e \) (as defined by Equation 8.4) is 3.41. The ratio of the length of reinforcement over the height of the wall is 0.8 and the wall is assumed to rest on a rigid foundation. The boundary conditions are similar to those in the reinforced earth model wall. The finite element mesh for the numerical analysis is shown in Figure 8.58.

8.7.3 Input parameters

In the first type of analysis, the reinforced soil were assumed to be one and a half times stiffer than unreinforced soil (backfill). The elements along the facing (elements 1-16) were assumed to be twice stiffer than unreinforced soil to include the effect of the facing elements. The input parameters are as follows:

\[ \gamma = 20 \text{kN/m}^3; \quad E_b = 35 \text{MPa}; \quad E_r = 52.5 \text{MPa}; \quad E_f = 70 \text{MPa}; \quad \nu = 0.30 \]

In the second type of analysis the elastic parameters were calculated following the procedure presented before. The equivalent elastic modulus \( E_e \) is 9960 MPa and the constant \( K_e \) is 3.41. The input parameters for the reinforced zone in this analysis were:
Figure 8.58  Finite Element mesh for full-scale reinforced earth wall
The input parameters for the backfill were:
\[ \gamma = 20\text{kN/m}^3; \quad E_h = 154.80\text{MPa}; \quad E_v = 43.58\text{MPa}; \quad \nu_h = 0.28; \quad \nu_{hv} = 0.31; \quad G_v = 13.46\text{MPa} \]

8.7.4 Results

The contours of the vertical normal stress, horizontal normal stress and horizontal shear stress obtained from Type 1 analysis are shown in Figure 8.59. The vertical normal stress plot shows a stress concentration at the toe of the wall; along any horizontal plane the stress varies from one point to another and there is a distinctive transition zone at the back of the reinforced earth wall. Towards the top of the structure, the vertical pressure distribution is fairly uniform. The horizontal normal stress distribution shows a stress concentration at the toe; the stress is highest near the base. Near the facing the stresses are generally low, as expected, and, at the top part of the reinforced zone there are tensile stresses. The horizontal shear stress plot shows a stress concentration at the toe; at the upper part of the structure there is no shear stress. At the base, the shear stress is maximum close to the facing and decreases towards the backfill.
Figure 8.59 Stress contours in full scale reinforced earth wall: Type 1 Analysis (isotropic reinforced zone and modulus constant with depth) (unit in MPa)
The vertical normal stress, horizontal normal stress and horizontal shear stress contours for Type 2 analysis are shown in Figure 8.60. The vertical normal stress contours show a stress concentration at the toe of the wall and a transition zone at the back of the reinforced earth wall. At the upper part of the structure, the vertical pressure distribution is fairly uniform. The horizontal normal stress plot shows a stress concentration at the toe; the stress is highest at the base and low at most part of the facing. There are tensile stresses at the upper part of the reinforced earth wall. The horizontal shear stress plot shows stress concentration at the base of the reinforced earth wall. In general the stresses act in anticlockwise direction. However, at the top part of the reinforced zone, the stresses act in clockwise direction.

The horizontal displacement at the facing for both types of analysis is shown in Figure 8.61 and the vertical displacement of points along the facing is shown in Figure 8.62. It can be seen that Type 2 analysis predicted larger horizontal displacement than Type 1 analysis. For vertical displacement, Type 1 analysis predicted larger displacement than that predicted by Type 2 analysis.
Figure 8.60 Stress contours in full scale reinforced earth wall: Type 2 Analysis (anisotropic reinforced zone and modulus constant with depth) (unit in MPa)
Figure 8.61 Outward horizontal displacement of facing for full-scale reinforced earth wall

Figure 8.62 Downward vertical displacement at facing for full-scale reinforced earth wall
8.7.5 Discussion and conclusion

Type 2 analysis predicted higher horizontal stress at the base of the reinforced earth wall than Type 1 analysis however, in general, the results from Type 1 analysis has similar trend to that from Type 2 analysis. Type 1 analysis is a simplistic approach where the elastic parameters were of arbitrary values while the parameters for Type 2 analysis were based on a theory, hence this part of numerical analysis highlighted two points:

1) that a simple analysis to determine the stress and deformation of reinforced earth wall may be made assuming isotropy and that reinforced earth is stiffer than unreinforced soil.

2) that the numerical analysis of reinforced earth wall may be made using any commercial finite element package where the required elastic properties may be calculated using the theory presented by Harrison and Gerrard (1972).

The numerical results show that the vertical pressure distribution along any horizontal plane of the full-scale wall is not uniform and there is stress concentration at the toe of the wall. The horizontal normal stress contours show that, at the top of the reinforced zone, the stresses are tensile. The horizontal shear stress contours show that the stress is high along the base of the reinforced zone.
CHAPTER 9
CONCLUSIONS

This chapter consists of two parts:

Part A: Conclusions or Findings
Part B: Implications of the Findings on the Design of Reinforced Earth Walls

In Part A, results obtained from analytical, numerical and experimental investigations are summarised and in Part B the implications of these findings on the design of reinforced earth walls are discussed.

PART A

9.1 Conclusions or Findings

The work presented in this thesis can be grouped into three categories:

Category 1 - physical modelling of reinforced earth walls - simple laboratory models
Category 2 - numerical modelling of reinforced earth walls
Category 3 - investigation concerning the strength of reinforced soil in direct shear tests.

In each category, several investigations were performed.
The results from the physical and numerical modelling of reinforced earth walls concern the overall behaviour of a reinforced earth structure. On the other hand the results from direct shear tests concern the behaviour of reinforced earth at a unit cell level. The findings corresponding to the investigations performed under each of the above categories, are now summarised.

9.1.1 Category 1 - Physical modelling of reinforced earth walls

(a) The investigation concerning the effect of backfill thrust on the vertical pressure distribution along the base of a reinforced earth wall (Chapter 3), involved a reinforced earth model wall with a certain geometry (L/H of 1.23) and resting on a rigid foundation. The overall effect of the backfill thrust was to increase the vertical normal stress over most of the length of the wall, L. This behaviour was due to the combined effects of the horizontal thrust and friction mobilized at the back (vertical interface between reinforced and unreinforced zones) of the wall. Analysis assuming the wall as a rigid body show that the effect of this friction is to increase the vertical normal stress near the back of the wall and reduce the stress near the facing. When the friction effect was isolated, the effect of the lateral (horizontal) thrust is to decrease the vertical normal stress near the back of the
wall and increase the vertical normal stress elsewhere. The effect of the lateral thrust lessens with distance from the back of the wall. The observed results were compared with the theoretical (conventional) vertical pressure distributions: uniform, Meyerhof's and trapezoidal. The reduction in vertical normal stress near the back of the wall (end of the reinforcement strips) conforms qualitatively to the trapezoid theory. However, the maximum increase in vertical normal stress does not occur at the facing as predicted by that theory. In fact, the maximum increase in vertical normal stress is found to be at a point, approximately $0.8L$ (where $L$ is length of reinforcement strip) distance from the facing.

Also in Chapter 3, the moment resulting from a particular vertical pressure distribution was analysed about the toe of the wall. Both the experimental and theoretical vertical pressure distributions were analysed. The analysis of moments about the toe show that firstly, the effect of the backfill thrust (friction effect excluded) is to reduce the overturning resistance about the toe of the wall where the experimental observation is similar to that predicted by the trapezoid theory. Secondly, the resulting moment of the observed vertical pressure distribution for wall with backfill thrust is similar to that obtain from trapezoidal distribution indicating that the trapezoid theory is at least
 qualitatively correct in incorporating the effect of the backfill thrust with respect to the moment about the toe.

(b) In the second series of experiments using a reinforced earth model (Chapter 4), the influence of strip dimension (width) on the vertical pressure at the base of the reinforced earth wall and the backfill (unreinforced zone) was measured. It was found that:

1) the vertical pressure at the base of the structure is not uniform. Underneath the reinforced earth wall, vertical normal stress is highest near the facing and within the middle of the reinforced zone, the vertical normal stresses are low. The vertical pressure distribution is dependent on fill height; for low fill height (H/3 where H is 300mm) the vertical pressure distribution is fairly uniform. It would seem that the stress distribution is influenced by an arching phenomenon (in the direction of strips i.e. dimension L). Moreover, the boundary effects are obviously significant.

2) the vertical pressure distribution at the base of the structure is influenced by the reinforcement strip width. This conclusion were reached by comparing the shape of the pressure diagrams for several series of walls where the varying parameter between the series is the width of the reinforcement strips.
3) for reinforced earth model walls of different size (different L), the shape of the pressure diagram under the same fill height i.e. vertical pressure distribution along the base of the reinforced zone is similar no matter what the length (L) of the reinforcement strips. For the particular experiment, the lengths of reinforcement strip tested were 300mm and 370mm.

An important outcome of this investigation is a confirmation of the proposed arching phenomenon across the reinforcement strips which has been suggested by previous researchers i.e. Guilloux et al. (1979). They proposed that dilatancy occurs in the immediate vicinity of the reinforcement (due to frictional shear stress); however, the overburden suppresses the volumetric expansion giving rise to a locally enhanced vertical normal stress. These researchers have proven through constant volume direct shear tests on sand, that the vertical normal stress is enhanced when volume expansion is prevented, however, this is the first time that the arching across the reinforcement strips in a reinforced earth wall is confirmed in model tests. This investigation shows that the presence of reinforcement induces arching of vertical normal stress across the wall (width direction of wall i.e. dimension B). For a constant spacing of reinforcement strips, the arching is influenced by the reinforcement strip width, i.e., the
magnitude of the vertical normal stress across the wall is affected by the strip width. This arching phenomenon should not be confused with possible arching in the direction of strips (L direction) which appears to influence the distribution of vertical normal stress (at the base) along the length of the wall, i.e., in L direction.

9.1.2 Category 2 - Numerical modelling of reinforced earth walls

Numerical analyses were performed considering the reinforced zone to be a composite material of higher elastic modulus than the unreinforced zone. Results from these numerical analysis of reinforced earth model wall show that the vertical pressure distribution at the base of the structure is not uniform. The vertical normal stress is highest at the toe of the wall where it exceeds the overburden pressure. The horizontal normal stress increases with depth and is, as expected, highest near the base of the structure. The horizontal normal stress within the reinforced earth wall is higher than that within the backfill. The horizontal normal stress near the facing is generally small as expected and, at the top of the wall, the stress is tensile. Horizontal shear stress is high along the base of the wall and at the back of the wall; within the backfill the shear stress is small, as expected. The maximum horizontal displacement of the facing occurs at
about a third of the height of the wall, above the base. The vertical displacement is largest at the top of the wall and decreases towards the base.

Numerical study concerning the interaction of the reinforced earth wall and the backfill was also performed. This study shows that as the relative stiffness between the reinforced earth and the backfill, i.e. $E_r/E_b$, increases, the vertical and horizontal normal stresses within the reinforced earth wall increases while the horizontal shear stress decreases. Concurrently, within the backfill, the vertical and horizontal normal stresses decreases while the horizontal shear stress increases. This shows that the normal stresses are transfered to the reinforced earth wall as it gets stiffer relative to the backfill. As expected, deformation of the facing of the wall decreases with increasing relative stiffness.

Numerical studies concerning the effect of backfill thrust (lateral) were also made. These show that the backfill thrust reduces the vertical normal stress at the back of the wall and increases the vertical normal stress at the facing. The effect of the thrust is nonlinear as opposed to linear decrease and increase in vertical normal stress along a horizontal plane, predicted from trapezoid theory.
The influence of the strip width on the elastic properties of reinforced earth was calculated using the theory proposed by Harrison and Gerrard (1972). It was found that reinforcement placed laterally (along direction L), enhanced the stiffness of the soil in the lateral direction markedly (up to 650% in this case). The vertical stiffness of the soil was also enhanced but not as much (only up to 48%) as the lateral stiffness. The stiffness enhancement is dependent on the amount of reinforcement. The properties thus calculated were used in numerical analyses to investigate the effect of reinforcement strip width on the stress and deformation response of a reinforced earth model wall. It was found that, although not markedly, the distribution of stresses and deformation were different between model walls reinforced with different width of strips. An important point about this approach is that a finite element analysis may be performed without need for alteration to a basic finite element package; the elastic properties required in the input data may be calculated according to the theory presented.

9.1.3 Category 3 - Investigation concerning the strength of reinforced soil in direct shear tests

Direct shear tests on reinforced sand show that the presence of reinforcement increases the deformation modulus
of the sand and increases its internal friction angle. The strength enhancement is affected by the width of the strip and the relationship between the width of strip and the strength enhancement is nonlinear: there is an optimum width where the efficiency is optimum. Two hypotheses, based on the concept that a reinforced sample fails when there is slip at the soil-reinforcement interface, may be proposed from these laboratory tests:

1) It may be suggested that, above the optimum width, there is less grip on the strip perhaps because of slip planes which occur at the soil-reinforcement interface. It is known that the soil-reinforcement friction angle is lower than soil-soil friction angle.

2) Considered in conjunction with the findings that the arching across the reinforced earth model wall is influenced by the strip width, it may be proposed that the magnitude of stress that acts perpendicular to the reinforcement interface which governs the frictional resistance varies with strip width.

The type of reinforcement greatly influenced the strength enhancement of sand. Ribbed strips increased the strength of sand by 78% as compared to 20% with smooth strips. The deformation modulus is not affected by the type of strips.
Investigations were also made concerning the effect of submergence on the behaviour of reinforced sand in the shear box. It shows that the efficiency of reinforcement is not affected by submergence. The tests also show that the deformation modulus is not affected by submergence.

PART B

9.2 Implication of the Findings on the Design of Reinforced Earth Walls

The experimental work, and the numerical analyses described in previous chapters have led to several findings on the behaviour of reinforced earth. These findings concern soil-reinforcement interaction at the unit cell level and at the global level, and can be divided into two categories:

1) those from direct shear tests and
2) those from studies of reinforced earth model walls.

Some of the findings could be pertinent to the design of reinforced earth walls. It would be of interest to consider their implications in basic design considerations.
9.2.1 Implications of results from direct shear tests

Investigation on direct shear tests show that:

1) Reinforced soil has an enhanced internal friction angle $\phi_r$

2) Ribbed strips are more efficient in enhancing the strength of soil

3) The strength enhancement is not effected by submergence

1) Reinforced sand has an enhanced internal friction angle $\phi_r$

This finding is according to expectation and although it applies to a unit cell, it has implications for the design of a reinforced earth wall. Currently, in the design of reinforced earth wall, the internal friction angle $\phi$ of the soil is used to calculate the coefficient of active earth pressure $K_a$. The use of the internal friction angle
ϕ_r of the composite where the reinforced earth wall is treated as a homogenous composite, is proposed. It has also been demonstrated in Chapter 6 that an internal friction angle of composite is higher than the internal friction angle of soil. To see the effect of using internal friction angle of composite rather than the internal friction angle of soil, two design considerations were analysed. These are:

1) the lateral pressure within a reinforced earth wall which determines the tensile stress that will be induced in the reinforcement strips and,

2) the location of the Rankine failure plane.

Consider a hypothetical reinforced earth wall with a height, H, of 7.5m and a length, L, of 6m. The internal friction angle ϕ of sand is assumed to be 35° and the internal friction angle of composite is 42.3° which represents an increase of 21% This was the maximum increase in internal friction angle found in experiment described in Section 6A of Chapter 6. The bulk unit weight of the soil is 20kN/m^3. Since the volume of reinforcement in reinforced earth wall is small, its weight is negligible. The active earth pressure coefficient K_a and the maximum lateral stress in the wall calculated using Equation 9.1 for the two values of friction angle is tabulated in Table 9.1.
\[
\sigma_{H(\text{max})} = K_a \sigma_v
\]

\[
= K_a \gamma H
\]  

(9.1)

Table 9.1  
Maximum Lateral Stress

<table>
<thead>
<tr>
<th>Internal friction angle</th>
<th>(K_a)</th>
<th>[\tan^2(45 - \phi/2)]</th>
<th>(\sigma_{H(\text{max})}) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\phi = 35^0)</td>
<td>0.27</td>
<td></td>
<td>40.5</td>
</tr>
<tr>
<td>(\phi_r = 42.3^0)</td>
<td>0.20</td>
<td></td>
<td>30.0</td>
</tr>
</tbody>
</table>

It can be seen that by using an internal friction angle of composite, the tensile stress in the reinforcement strip which is directly related to the lateral stress (see Equation 3.20) will be reduced; in this case by 26%. Therefore, detailed design will be influenced e.g. selection of reinforcement strip cross-section, length and spacing. Suffice to say that the use of internal friction angle of composite will result in cost saving in term of the proportion of reinforcement to be used in a reinforced earth wall.

As discussed in Chapter 3, the Rankine earth pressure theory is used in some design methods to predict the location of the potential failure plane. This plane is used
to define the boundary between the active and the resistant zones. In some instances, only the length of reinforcement strip within the resistant zone is considered effective for adherence or bond resistance. For the example of wall considered above, it can be seen in Figure 9.1 that the Rankine failure plane based on \( \phi_r \) bounds a smaller failure wedge (implying a larger resisting zone) than the failure plane when \( \phi \) is used. Thus for the same length of reinforcement strip, higher safety factor in term of adherence (see Equation 3.26) will be computed, more significantly at higher levels of the wall. Consequently a shorter length of some strips may be required at higher levels in the Rankine design method.

Having shown that the use of internal friction angle of composite can result in a less conservative and more economical design, we need to consider how, in practice, the value of the internal friction angle of composite may be obtained. The results found in Section 6A of Chapter 6 are relevant in this regard. There was an optimum width of strips in the direct shear box which resulted in a composite with the highest \( \phi_r \). This result cannot be applied directly to a reinforced earth wall since the degree of similitude is not known. For the purpose of illustration, one may consider only the range of increase in the internal friction angle. The experiment showed that the maximum increase in friction angle was 21% while the
Figure 9.1  Rankine failure planes using internal friction angle of soil, $\phi$, and composite internal friction angle, $\phi_r$, in a reinforced earth wall
minimum increase was 4%. The corresponding internal friction angles of composite, for a soil of $\phi = 35^0$, would be in the range of $36.4^0$ to $42.3^0$.

Investigations from direct shear tests also confirmed that
2) Ribbed strips are more efficient in enhancing the strength of soil.

In practice ribbed strips have been used extensively. This is due to their superior performance resulting from better frictional interaction. In this thesis, this fact is further developed where the strength enhancement due to the improved frictional interaction was quantified. It was found that soil reinforced with ribbed strips is nearly 50% stronger than soil reinforced with smooth strips. Once again, the implications for calculating the coefficient of active earth pressure and the location of the Rankine failure plane are obvious. Of course, changes in design procedures must be confirmed by test on full scale walls or tests from more sophisticated laboratory tests such as tests of models in a centrifuge.

From direct shear tests described in Section 6C in Chapter 6, composite (reinforced sand) reinforced with smooth strip has internal friction angle which is 12.9% more than internal friction angle of unreinforced sand
while composite reinforced with ribbed strips has internal friction angle which is 39.3% more than internal friction angle of unreinforced sand. This result will be used in the analysis to calculate the maximum lateral stress as performed before on the hypothetical full-scale reinforced earth wall; the internal friction angle of the sand is assumed to be 35° and the internal friction angle for composite with smooth strips and composite with ribbed strips are 39.5° and 48.8°, respectively. The results are tabulated in Table 9.2.

Table 9.2 Maximum Lateral Stress

<table>
<thead>
<tr>
<th>Friction angle</th>
<th>$K_a$</th>
<th>$\sigma_{H(\text{max})}$ (kPa)</th>
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</thead>
<tbody>
<tr>
<td>$\phi = 35^\circ$</td>
<td>0.27</td>
<td>40.5</td>
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<tr>
<td>$\phi_r(\text{smooth})$</td>
<td>0.22</td>
<td>33.0</td>
</tr>
<tr>
<td>$\phi_r(\text{ribbed})$</td>
<td>0.14</td>
<td>21.0</td>
</tr>
</tbody>
</table>

It can be seen that when internal friction angle of composite is used over internal friction angle of unreinforced soil, for both types of reinforcement strip, the maximum tensile stress will be reduced; the use of ribbed strips will be more efficient since it brings a reduction of maximum tensile stress by 48%. This will have a considerable effect on the design outcome, since the maximum tensile stress will govern the dimensions and the spacing of the strips. Moreover, the reinforced earth wall with ribbed strip has a smaller active wedge, hence, in
terms of factor of safety against adherence failure, the use of ribbed strips is superior to the use of smooth strips. Consequently, a shorter length of reinforcement may be used when the strips are ribbed.

In conclusion, the work presented in this thesis confirms the superior performance expected from ribbed strips and the internal friction angle of composite with ribbed strips is found to be up to 39% larger than the internal friction angle of unreinforced soil.

Investigation from direct shear tests also showed that
3) The strength enhancement is not affected by submergence.

This result may be used to validate or justify design considerations where factor of safety against internal failure (adherence failure) mode are the same for either dry or submerged walls.

9.2.2 Implications of results from model tests and numerical analysis of reinforced earth walls

Experimental and numerical studies of reinforced earth model walls showed that at the base of the wall, the vertical pressure distribution is neither uniform nor trapezoidal. The vertical normal stress is highest at the toe of the wall where it exceeds the overburden stress. This must be taken into account to calculate the maximum
tensile stress in the reinforcement strip and also bearing capacity failure of the foundation soil on which the reinforced earth wall is built on.

Experimental investigation on the backfill thrust show that there is frictional effect at the back of the wall. If the trapezoid theory is to be used, this frictional effect must be taken into account in addition to the horizontal lateral thrust. Moreover this maximum stress might occur at the back of the wall. The maximum stress will influence the design in terms of bearing capacity of the foundation e.g. the selection of the length L to avoid bearing capacity failure.
REFERENCES


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under static and repeated loading'. Proc. 6th. Conf. on Soil Mechanics and Foundation Engineering, Budapest, pp.511-517.


deposits'. Lecture 3, Post-graduate course on Soil Reinforcement - Mechanics and Design, University of Sydney.


Shen, C.K., Romstad, K.M. and Hermann, L.R.


APPENDIX A

TESTS DATA

(Vertical pressure readings)
### Table A.1

<table>
<thead>
<tr>
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<th>Fill height above measuring surface h (mm)</th>
<th>Pressure transducers reading (kPa)</th>
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<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
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<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
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Table A.4  Average Result: TESTS A2 and A3

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<th>Fill height above measuring surface h (mm)</th>
<th>Pressure transducers reading (kPa)</th>
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Table A.5  Result: TEST B1

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<td>PT2</td>
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Table A.6  Result: TEST B2

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<th>Fill height above measuring surface h (mm)</th>
<th>Pressure transducers reading (kPa)</th>
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<td>105</td>
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<td>(before lift)</td>
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### Table A.7

Result: TEST B3

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<th>PT4</th>
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### Table A.8

Result: TEST B4

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### Table A.9

Average Result: TESTS B1, B2, B3 and B4

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**Table A.11**

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<td>5.310</td>
<td>2.813</td>
</tr>
<tr>
<td>300</td>
<td>300 (after lift)</td>
<td>5.487</td>
<td>4.847</td>
<td>3.618</td>
<td>3.298</td>
<td>5.351</td>
<td>3.694</td>
</tr>
</tbody>
</table>

**Table A.12**

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Fill height above measuring surface h (mm)</th>
<th>Pressure transducers reading (kPa) PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>55</td>
<td>1.403</td>
<td>0.994</td>
<td>0.890</td>
<td>0.733</td>
<td>1.307</td>
<td>1.721</td>
</tr>
<tr>
<td>150</td>
<td>105</td>
<td>2.600</td>
<td>1.740</td>
<td>1.220</td>
<td>1.181</td>
<td>1.838</td>
<td>2.183</td>
</tr>
<tr>
<td>200</td>
<td>155</td>
<td>3.465</td>
<td>2.734</td>
<td>1.628</td>
<td>1.506</td>
<td>3.186</td>
<td>2.519</td>
</tr>
<tr>
<td>250</td>
<td>205</td>
<td>4.373</td>
<td>3.728</td>
<td>1.992</td>
<td>1.792</td>
<td>4.248</td>
<td>3.694</td>
</tr>
<tr>
<td>300</td>
<td>255</td>
<td>5.404</td>
<td>5.178</td>
<td>2.846</td>
<td>2.280</td>
<td>5.024</td>
<td>3.862</td>
</tr>
</tbody>
</table>
Vertical pressure readings along the reinforced zone of 300mm as the backfill was being placed.

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Height of backfill beyond partition S (mm)</th>
<th>Pressure transducers reading (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>5.404</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>5.448</td>
</tr>
<tr>
<td>300</td>
<td>150</td>
<td>5.446</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>5.446</td>
</tr>
<tr>
<td>300</td>
<td>250</td>
<td>5.446</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.569</td>
</tr>
<tr>
<td>(before lift)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.569</td>
</tr>
</tbody>
</table>

Table A.14 Average Result: TESTS C1 and C2
(from Tables A.10 and A.12)

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Fill height above measuring surface h (mm)</th>
<th>Pressure transducers reading (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>1.217</td>
</tr>
<tr>
<td>150</td>
<td>105</td>
<td>2.498</td>
</tr>
<tr>
<td>200</td>
<td>155</td>
<td>3.300</td>
</tr>
<tr>
<td>250</td>
<td>205</td>
<td>4.249</td>
</tr>
<tr>
<td>300</td>
<td>255</td>
<td>5.322</td>
</tr>
</tbody>
</table>

Table A.15 Average Result: TEST C1 and C2 (from Tables A.11 and A.13)
Vertical pressure readings along the reinforced zone of 300mm as the backfill was being placed.

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Height of backfill beyond partition S (mm)</th>
<th>Pressure transducers reading (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>5.322</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>5.322</td>
</tr>
<tr>
<td>300</td>
<td>150</td>
<td>5.322</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>5.342</td>
</tr>
<tr>
<td>300</td>
<td>250</td>
<td>5.363</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.424</td>
</tr>
<tr>
<td>(before lift)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.528</td>
</tr>
</tbody>
</table>
Vertical pressure reading as the reinforced zone was being constructed. No backfilling beyond partition.

<table>
<thead>
<tr>
<th>Total fill height ( d ) (mm)</th>
<th>Fill height above measuring surface ( h ) (mm)</th>
<th>Pressure transducers reading ((kPa))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT1</td>
<td>PT2</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>1.030</td>
</tr>
<tr>
<td>150</td>
<td>105</td>
<td>2.310</td>
</tr>
<tr>
<td>200</td>
<td>155</td>
<td>3.424</td>
</tr>
<tr>
<td>250</td>
<td>205</td>
<td>4.414</td>
</tr>
<tr>
<td>300</td>
<td>255</td>
<td>6.064</td>
</tr>
</tbody>
</table>

Table A.17 Result: TEST D1
Vertical pressure readings along the reinforced zone of 300mm as the backfill was being placed.

<table>
<thead>
<tr>
<th>Total fill height ( d ) (mm)</th>
<th>Height of backfill beyond partition ( S ) (mm)</th>
<th>Pressure transducers reading ((kPa))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT1</td>
<td>PT2</td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>6.064</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>5.940</td>
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<tr>
<td>300</td>
<td>150</td>
<td>5.940</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>5.982</td>
</tr>
<tr>
<td>300</td>
<td>250</td>
<td>5.940</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.940</td>
</tr>
</tbody>
</table>

Table A.18 Result: TEST D2
Vertical pressure readings as the reinforced zone was being constructed. No backfilling beyond partition.

<table>
<thead>
<tr>
<th>Total fill height ( d ) (mm)</th>
<th>Fill height above measuring surface ( h ) (mm)</th>
<th>Pressure transducers reading ((kPa))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT1</td>
<td>PT2</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>0.866</td>
</tr>
<tr>
<td>150</td>
<td>105</td>
<td>1.774</td>
</tr>
<tr>
<td>200</td>
<td>155</td>
<td>2.475</td>
</tr>
<tr>
<td>250</td>
<td>205</td>
<td>3.300</td>
</tr>
<tr>
<td>300</td>
<td>255</td>
<td>4.373</td>
</tr>
</tbody>
</table>
Vertical pressure readings along the reinforced zone of 300mm as the backfill was being placed.

### Table A.19

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Height of backfill beyond partition S (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>4.373</td>
<td>3.935</td>
<td>2.683</td>
<td>2.402</td>
<td>4.984</td>
<td>4.366</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>4.208</td>
<td>3.977</td>
<td>2.946</td>
<td>2.606</td>
<td>5.024</td>
<td>4.366</td>
</tr>
<tr>
<td>300</td>
<td>150</td>
<td>4.208</td>
<td>3.977</td>
<td>2.927</td>
<td>2.728</td>
<td>5.147</td>
<td>4.324</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>4.208</td>
<td>3.977</td>
<td>3.049</td>
<td>2.991</td>
<td>5.229</td>
<td>3.946</td>
</tr>
<tr>
<td>300</td>
<td>250</td>
<td>4.208</td>
<td>3.977</td>
<td>3.171</td>
<td>3.013</td>
<td>5.270</td>
<td>3.442</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>4.208</td>
<td>4.060</td>
<td>3.330</td>
<td>3.176</td>
<td>5.351</td>
<td>2.813</td>
</tr>
</tbody>
</table>

### Table A.20

Average Result: TESTS D1 and D2

Vertical pressure readings as the reinforced zone was being constructed. No backfilling beyond partition.

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Fill height above measuring surface h (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>55</td>
<td>0.948</td>
<td>0.725</td>
<td>0.712</td>
<td>0.652</td>
<td>1.022</td>
<td>1.112</td>
</tr>
<tr>
<td>150</td>
<td>105</td>
<td>2.042</td>
<td>1.492</td>
<td>1.199</td>
<td>1.221</td>
<td>2.062</td>
<td>2.099</td>
</tr>
<tr>
<td>200</td>
<td>155</td>
<td>2.950</td>
<td>2.340</td>
<td>1.544</td>
<td>1.608</td>
<td>3.206</td>
<td>2.544</td>
</tr>
<tr>
<td>250</td>
<td>205</td>
<td>3.857</td>
<td>3.231</td>
<td>1.951</td>
<td>1.954</td>
<td>4.125</td>
<td>2.896</td>
</tr>
<tr>
<td>300</td>
<td>255</td>
<td>5.218</td>
<td>4.308</td>
<td>2.480</td>
<td>2.382</td>
<td>5.066</td>
<td>3.736</td>
</tr>
</tbody>
</table>

### Table A.21

Average Result: TESTS D1 and D2

Vertical pressure readings along the reinforced zone of 300mm as the backfill was being placed.

<table>
<thead>
<tr>
<th>Total fill height d (mm)</th>
<th>Height of backfill beyond partition S (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>5.218</td>
<td>4.308</td>
<td>2.480</td>
<td>2.382</td>
<td>5.066</td>
<td>3.736</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>5.074</td>
<td>4.329</td>
<td>2.663</td>
<td>2.688</td>
<td>5.106</td>
<td>3.590</td>
</tr>
<tr>
<td>300</td>
<td>150</td>
<td>5.074</td>
<td>4.329</td>
<td>2.744</td>
<td>2.830</td>
<td>5.208</td>
<td>3.454</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>5.074</td>
<td>4.329</td>
<td>2.866</td>
<td>2.992</td>
<td>5.290</td>
<td>3.212</td>
</tr>
<tr>
<td>300</td>
<td>250</td>
<td>5.074</td>
<td>4.329</td>
<td>2.988</td>
<td>3.135</td>
<td>5.331</td>
<td>2.854</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>5.074</td>
<td>4.391</td>
<td>3.108</td>
<td>3.258</td>
<td>5.392</td>
<td>2.498</td>
</tr>
</tbody>
</table>
APPENDIX B

CORRECTION FOR FRICTION
Table B.1 Total Vertical Force at H = 300mm (therefore $h = 255\text{mm}$)

<table>
<thead>
<tr>
<th>Test</th>
<th>Force before lift</th>
<th>Force after lift</th>
<th>Gauged length $L_g$</th>
<th>Force difference $\Delta p$</th>
<th>Total force based on weight $W$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kN/m)</td>
<td>(kN/m)</td>
<td>(mm)</td>
<td>(kN/m)</td>
<td>(kN/m)</td>
</tr>
<tr>
<td>A1</td>
<td>1.0087</td>
<td>1.2747</td>
<td>350</td>
<td>0.2660</td>
<td>1.3976</td>
</tr>
<tr>
<td>A2</td>
<td>1.4060</td>
<td>1.4808</td>
<td>350</td>
<td>0.0748</td>
<td>1.3976</td>
</tr>
<tr>
<td>A3</td>
<td>1.3167</td>
<td>1.4987</td>
<td>350</td>
<td>0.1820</td>
<td>1.3976</td>
</tr>
<tr>
<td>B1</td>
<td>1.1006</td>
<td>1.2383</td>
<td>315</td>
<td>0.1377</td>
<td>1.2579</td>
</tr>
<tr>
<td>B2</td>
<td>1.1343</td>
<td>1.2420</td>
<td>315</td>
<td>0.1077</td>
<td>1.2579</td>
</tr>
<tr>
<td>B3</td>
<td>1.0622</td>
<td>1.1847</td>
<td>315</td>
<td>0.1225</td>
<td>1.2579</td>
</tr>
<tr>
<td>B4</td>
<td>1.1899</td>
<td>1.3050</td>
<td>315</td>
<td>0.1351</td>
<td>1.2579</td>
</tr>
</tbody>
</table>

* A1 disregarded in any mean calculation

Correction for Friction at the Back of the Wall

Theory

It has been shown in the text (Figure 4.12) that when there is backfill thrust, the vertical stress at a point within the reinforced earth wall are a summation of three components: overburden, lateral thrust effect and friction effect. Before the partition was lifted the stress is only due to overburden. Using these informations, the effect of the lateral thrust can be isolated where:

$$\text{Overburden} + \text{lateral thrust effect} + \text{friction effect} = \text{measured stress after lift}$$

$$\text{lateral thrust effect} = \left( \text{measured stress after lift} \right) - \left( \text{Overburden} + \text{friction effect} \right)$$
(Overburden + friction effect) is defined as the corrected pressure or stress. The stress due to overburden is the vertical pressure measured before lift while the stress due to friction has to be determined. The friction force is known, i.e. $\Delta p$, however the area it acts is unknown. An assumption was made where the friction force was assumed as a concentrated force at PT6. Accordingly, the corrected stress at PT6 can be determined using the fact that the corrected area of the pressure diagram between PT5 and PT6 should be equal to the area between PT5 and PT6 before lift plus $\Delta p$ (see example below).

**Example**

**Test B2**

pressure reading at $d = 300\text{mm}$ (before lift) at:

$\text{PT5} = 3.799\text{kPa}$

$\text{PT6} = 3.023\text{kPa}$

From Table B.1, the frictional force $\Delta p$ is $0.1077\text{kN/m}$

<table>
<thead>
<tr>
<th>PT5</th>
<th>PT6</th>
<th>Pressure diagrams</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.799</td>
<td>3.023</td>
<td>$\Delta p$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Before lift (uncorrected)</td>
</tr>
<tr>
<td>3.799</td>
<td>3.023</td>
<td>$\Delta p$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Before lift + Friction assumed as concentrated force at PT6 (corrected)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Q = 6.440\text{kPa}$</td>
</tr>
</tbody>
</table>

**Concept:** Area 1 + $\Delta p$ = corrected pressure diagram

Area 1 = $0.5(3.799 + 3.023) \times 0.063$

= 0.2149

Area 2 = corrected pressure diagram

$0.2149 + \Delta p = 0.5(3.799 + Q) \times 0.063$

$0.2149 + 0.1077 = 0.5(3.799 + Q) \times 0.063$

$Q = 6.440\text{kPa}$

Therefore corrected pressure reading at PT6 at $300\text{mm}$ before lift is $6.440\text{kPa}$. 
Table B.2 Vertical Pressure at PTB at H = 300mm (h = 255mm)

<table>
<thead>
<tr>
<th>Test</th>
<th>Pressure reading before lift (kPa)</th>
<th>Pressure after correction (kPa)</th>
<th>Pressure reading after lift (kPa)</th>
<th>Percentage change in vertical pressure due to lateral thrust (friction effect excluded)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>2.603</td>
<td>10.203</td>
<td>4.576</td>
<td>-55.2%</td>
</tr>
<tr>
<td>A2</td>
<td>3.778</td>
<td>5.920</td>
<td>4.576</td>
<td>-22.7%</td>
</tr>
<tr>
<td>A3</td>
<td>3.568</td>
<td>8.768</td>
<td>5.458</td>
<td>-37.7%</td>
</tr>
<tr>
<td>mean</td>
<td>3.673</td>
<td>7.344</td>
<td>5.017</td>
<td>-30.2%</td>
</tr>
<tr>
<td>S_n-1</td>
<td>0.148</td>
<td>2.014</td>
<td>0.824</td>
<td>10.7</td>
</tr>
</tbody>
</table>

| B1   | 3.316                            | 7.680                         | 4.954                           | -35.5%                                                                          |
| B2   | 3.023                            | 6.440                         | 4.660                           | -27.6%                                                                          |
| B3   | 2.939                            | 6.829                         | 4.450                           | -34.8%                                                                          |
| B4   | 3.358                            | 7.648                         | 4.996                           | -34.7%                                                                          |
| mean | 3.159                            | 7.149                         | 4.765                           | -33.2%                                                                          |
| S_n-1| 0.209                            | 0.615                         | 0.258                           | 3.7                                                                           |

| C1   | 3.694                            | 9.590                         | 3.694                           | -61.4%                                                                          |
| C2   | 3.862                            | 7.122                         | 3.694                           | -48.1%                                                                          |
| mean | 3.778                            | 8.351                         | 3.694                           | -54.8%                                                                          |
| S_n-1| 0.119                            | 1.738                         | 0.0                             | 9.4                                                                           |

* disregarded in the mean calculation
APPENDIX C

EFFECT OF LATERAL THRUST ON VERTICAL PRESSURE
Table C.1 Percentage Change in Vertical Pressure along the Reinforced Zone due to Lateral Thrust (friction effect excluded)

<table>
<thead>
<tr>
<th>Test</th>
<th>Percentage Change at Gauged Point %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(+) increase , (-) decrease</td>
</tr>
<tr>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>A1</td>
<td>12.2</td>
</tr>
<tr>
<td>A2</td>
<td>0.6</td>
</tr>
<tr>
<td>A3</td>
<td>5.2</td>
</tr>
<tr>
<td>B1</td>
<td>-0.6</td>
</tr>
<tr>
<td>B2</td>
<td>-2.1</td>
</tr>
<tr>
<td>B3</td>
<td>1.4</td>
</tr>
<tr>
<td>B4</td>
<td>7.7</td>
</tr>
<tr>
<td>C1</td>
<td>3.9</td>
</tr>
<tr>
<td>C2</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Table C.2 Mean Percentage Change (results from Table C.1)

| Mean from Test | Percentage change % |
|               | Gauged Point         |
|               | PT1  | PT2  | PT3  | PT4  | PT5  | PT6  |
| A2,A3         | 2.9  | 4.8  | 5.3  | 6.4  | 14.0 | -30.2 |
| B1,B2,B3,B4   | 1.6  | 2.8  | 4.4  | 8.2  | 20.2 | -33.2 |
| C1,C2         | 2.0  | 3.1  | 3.0  | 0.0  | 1.4  | -54.8 |
APPENDIX D

THEORETICAL VERTICAL PRESSURE DISTRIBUTIONS
Vertical Pressure Distribution

Uniform Distribution

\[ \sigma_v = \gamma h \]
\[ = 15.66 \times 0.255 \]
\[ = 3.993 \text{kPa} \]

Meyerhof's Distribution

\[ \sigma_v = \frac{\gamma h L}{L - 2e_m} \]
\[ = \frac{0.266 \times 0.255^2}{6 \times 0.370} \]
\[ = 7.79 \times 10^{-3} \text{m} = 7.8 \text{mm} \]

\[ e_m = \frac{K h^2}{6L} \]
\[ = \frac{0.266 \times 0.255}{6 \times 0.315} \]
\[ = 4.201 \text{kPa} \]
\[ \sigma_a = \gamma h \left[ 1 + K_a \left( \frac{h}{L} \right)^2 \right] \]
\[ = 15.66 \times 0.255 \left[ 1 + 0.266 \left( \frac{0.255}{0.370} \right)^2 \right] \]
\[ = 4.498 \text{kPa} \]

\[ \sigma_b = \gamma h \left[ 1 - K_a \left( \frac{h}{L} \right)^2 \right] \]
\[ = 15.66 \times 0.255 \left[ 1 - 0.266 \left( \frac{0.255}{0.370} \right)^2 \right] \]
\[ = 3.489 \text{kPa} \]

Thus from linearity, vertical pressure at:

PT1 (31.5mm from facing) is 4.412kPa

PT6 (346.5mm from facing) is 3.553kPa

Compare with experimental result:

(average from Bs)

vertical pressure at PT1 is 6.054kPa

vertical pressure at PT6 is 4.765kPa
APPENDIX E1
MANUFACTURER'S SPECIFICATION
(for pressure transducer)

SPECIFICATIONS

Pressure: 0-6, 0-15, 0-25, 0-50,
Ranges: 0-100, 0-200, 0-500,
0-1000, 0-2000, 0-3000,
0-5000, 0-10000, 0-20000 PSIG
0-15, 0-25, 0-50 PSIA

Overload: 2 times rated pressure
without damage. 5 times
rated pressure without
bursting. Transducer undamaged by 3 million
cycles, 0 to full range.
Adapter working pressure, 10,000 PSI max.;
burst pressure, 20,000 PSI.

Material: Stainless Steel

Accuracy: Within 1% over full
range at normal temperature.

Excitation Voltage:

Voltage: 5 V DC or AC RMS.

Output Signal at rated

Pressure: 100 millivolts ± 1%.

Bridge Output: 115 ± 25 ohms.

Resistance: 150 ± 50 ohms.

Operating Temperature: -65° to +200°F.

Shock and Vibration: Undamaged by 50 G.

Weight: Approximately 2 oz. less cable.

SCHEMATIC DIAGRAM

OUTLINE DIMENSIONS

ACCESSORIES AVAILABLE

TYCO has a complete line of accessory equipment for use with these pressure transducers. Ask for information on:
- Power Supplies
- Amplifiers
- Indicators: analog and digital
- Controllers
- Recorders
- Special Systems
- Meters
- Adapters for different thread connections.

PURCHASE INFORMATION

Specify Model No. as below:
AB — (Pressure Range)
Example: AB-500 defines a 500 PSI transducer.
The following adapters are stocked:
AD-1: 1/4" male NPT, cad. plated steel
AD-1 SS: 1/4" male NPT, stainless steel
AD-2: 1/4" tubing, cad. plated steel

Specifications subject to change without notice.

TYCO INSTRUMENT DIVISION | 150 COOLIDGE AVE. WATERTOWN, MASS. 02172 / TEL (617) 926-3400 / TWX (710) 327-6998
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CALIBRATION OF PRESSURE TRANSDUCERS

Note:

- Room temperature 22.5°C
- Water temperature 22.5°C

Thus unit weight of water at this temperature is 9.78 kN/m³

RESULTS

VOLTAGE versus PRESSURE

Regression Analysis

- PT1, \( y = -0.35909 + 2.4239x \) \( R^2 = 1.000 \)
- PT2, \( y = 3.9955 + 2.4276x \) \( R^2 = 1.000 \)
- PT3, \( y = -4.7455 + 2.4545x \) \( R^2 = 1.000 \)
- PT4, \( y = -3.2864 + 2.4536x \) \( R^2 = 1.000 \)
- PT5, \( y = 2.7955 + 2.4481x \) \( R^2 = 1.000 \)
- PT6, \( y = -1.4818 + 2.3840x \) \( R^2 = 1.000 \)
**CALIBRATION FACTORS OF PRESSURE TRANSUDCERS**

<table>
<thead>
<tr>
<th>Pressure Transducer</th>
<th>Calibration Factor (mV/kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>2.424</td>
</tr>
<tr>
<td>PT2</td>
<td>2.428</td>
</tr>
<tr>
<td>PT3</td>
<td>2.454</td>
</tr>
<tr>
<td>PT4</td>
<td>2.454</td>
</tr>
<tr>
<td>PT5</td>
<td>2.448</td>
</tr>
<tr>
<td>PT6</td>
<td>2.384</td>
</tr>
</tbody>
</table>
APPENDIX F

RESULT OF DIRECT SHEAR TEST ON SAND
(using 60mm square shear box)

Regression analysis: \[ y = 0.43111 + 0.71150x \quad R^2 = 0.983 \]

Therefore \( \phi \) is 35.4°
APPENDIX G

TENSILE TEST ON SHIM STEEL


Dimension of specimen

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Specimen A</th>
<th>Specimen B</th>
<th>Specimen C</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>50.00mm</td>
<td>50.00mm</td>
<td>50.00mm</td>
</tr>
<tr>
<td>W</td>
<td>12.90mm</td>
<td>12.46mm</td>
<td>12.86mm</td>
</tr>
<tr>
<td>T</td>
<td>0.20mm</td>
<td>0.20mm</td>
<td>0.20mm</td>
</tr>
<tr>
<td>L</td>
<td>216mm</td>
<td>216mm</td>
<td>217mm</td>
</tr>
<tr>
<td>B</td>
<td>57mm</td>
<td>57mm</td>
<td>57mm</td>
</tr>
<tr>
<td>A</td>
<td>70mm</td>
<td>70mm</td>
<td>70mm</td>
</tr>
<tr>
<td>C</td>
<td>20mm</td>
<td>20mm</td>
<td>20mm</td>
</tr>
</tbody>
</table>
## Test results

<table>
<thead>
<tr>
<th></th>
<th>Specimen A</th>
<th>Specimen B</th>
<th>Specimen C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rupture load (kN)</td>
<td>1.50 kN</td>
<td>1.46 kN</td>
<td>1.47 kN</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>581.40</td>
<td>585.87</td>
<td>571.54</td>
</tr>
<tr>
<td>Young's Modulus of Elasticity (MPa)</td>
<td>64816</td>
<td>70022</td>
<td>70635</td>
</tr>
</tbody>
</table>

* Specimen A ruptured outside gauge length thus its results are not used to calculate average results:

\[
\text{Tensile strength} = \frac{585.87 + 571.54}{2} = 580 \text{ MPa}
\]

\[
\text{Young's Modulus of Elasticity} = \frac{70022 + 70635}{2} = 70329 \text{ MPa}
\]
CALIBRATION OF PRESSURE TRANSDUCERS

Note:

Room temperature $22.5^\circ$ C
Water temperature $22.5^\circ$ C
Thus unit weight of water at this temperature is $9.78 \text{kN/m}^3$

RESULTS

VOLTAGE versus PRESSURE

Regression Analysis

<table>
<thead>
<tr>
<th>PT1</th>
<th>$y = -5.0955 + 2.9341x$</th>
<th>$R^2 = 1.000$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT2</td>
<td>$y = -6.2045 + 2.9991x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT3</td>
<td>$y = -10.032 + 3.0139x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT4</td>
<td>$y = -8.5364 + 2.9666x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT5</td>
<td>$y = 1.7545 + 2.9499x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT6</td>
<td>$y = -7.2318 + 2.8915x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT7</td>
<td>$y = -0.48636 + 3.0213x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT8</td>
<td>$y = 1.0000E-1 + 3.0705x$</td>
<td>$R^2 = 1.000$</td>
</tr>
<tr>
<td>PT9</td>
<td>$y = 1.8455 + 3.0223x$</td>
<td>$R^2 = 1.000$</td>
</tr>
</tbody>
</table>
## CALIBRATION FACTORS OF PRESSURE TRANSUDCERS

<table>
<thead>
<tr>
<th>Pressure Transducer</th>
<th>Calibration Factor (mV/kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>2.934</td>
</tr>
<tr>
<td>PT2</td>
<td>2.999</td>
</tr>
<tr>
<td>PT3</td>
<td>3.014</td>
</tr>
<tr>
<td>PT4</td>
<td>2.967</td>
</tr>
<tr>
<td>PT5</td>
<td>2.950</td>
</tr>
<tr>
<td>PT6</td>
<td>2.891</td>
</tr>
<tr>
<td>PT7</td>
<td>3.021</td>
</tr>
<tr>
<td>PT8</td>
<td>3.070</td>
</tr>
<tr>
<td>PT9</td>
<td>3.022</td>
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</tbody>
</table>
APPENDIX I

TESTS DATA

(Vertical pressure readings)
<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.091</td>
<td>0.567</td>
<td>0.597</td>
<td>0.539</td>
<td>0.576</td>
<td>0.623</td>
<td>0.496</td>
<td>0.423</td>
<td>0.298</td>
</tr>
<tr>
<td>150</td>
<td>2.045</td>
<td>1.000</td>
<td>1.028</td>
<td>1.011</td>
<td>1.186</td>
<td>1.245</td>
<td>1.192</td>
<td>1.092</td>
<td>0.960</td>
</tr>
<tr>
<td>200</td>
<td>3.067</td>
<td>1.400</td>
<td>1.327</td>
<td>1.314</td>
<td>1.661</td>
<td>1.799</td>
<td>1.854</td>
<td>1.629</td>
<td>1.390</td>
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<tr>
<td>250</td>
<td>Not available</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>Not available</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.159</td>
<td>0.567</td>
<td>0.431</td>
<td>0.270</td>
<td>0.610</td>
<td>0.553</td>
<td>0.232</td>
<td>0.130</td>
<td>0.165</td>
</tr>
<tr>
<td>150</td>
<td>2.215</td>
<td>1.200</td>
<td>0.863</td>
<td>0.674</td>
<td>1.254</td>
<td>1.176</td>
<td>0.828</td>
<td>0.651</td>
<td>0.662</td>
</tr>
<tr>
<td>200</td>
<td>3.647</td>
<td>1.834</td>
<td>1.161</td>
<td>0.910</td>
<td>1.797</td>
<td>1.730</td>
<td>1.390</td>
<td>1.075</td>
<td>0.960</td>
</tr>
<tr>
<td>250</td>
<td>4.192</td>
<td>2.267</td>
<td>1.360</td>
<td>1.078</td>
<td>2.102</td>
<td>2.144</td>
<td>1.854</td>
<td>1.368</td>
<td>1.158</td>
</tr>
<tr>
<td>300</td>
<td>Not available</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table I.3**

Average from TESTS A1 and A2

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.125</td>
<td>0.567</td>
<td>0.514</td>
<td>0.404</td>
<td>0.593</td>
<td>0.588</td>
<td>0.364</td>
<td>0.276</td>
<td>0.232</td>
</tr>
<tr>
<td>150</td>
<td>2.130</td>
<td>1.100</td>
<td>0.946</td>
<td>0.842</td>
<td>1.220</td>
<td>1.210</td>
<td>1.010</td>
<td>0.872</td>
<td>0.811</td>
</tr>
<tr>
<td>200</td>
<td>3.357</td>
<td>1.617</td>
<td>1.244</td>
<td>1.112</td>
<td>1.729</td>
<td>1.764</td>
<td>1.622</td>
<td>1.352</td>
<td>1.175</td>
</tr>
<tr>
<td>250</td>
<td>4.192</td>
<td>2.267</td>
<td>1.360</td>
<td>1.078</td>
<td>2.102</td>
<td>2.144</td>
<td>1.854</td>
<td>1.368</td>
<td>1.158</td>
</tr>
<tr>
<td>300</td>
<td>Not available</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Table 1.4

**TEST B1**

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.466</td>
<td>0.967</td>
<td>0.763</td>
<td>0.674</td>
<td>0.881</td>
<td>0.796</td>
<td>0.728</td>
<td>0.651</td>
<td>0.430</td>
</tr>
<tr>
<td>150</td>
<td>2.692</td>
<td>2.001</td>
<td>1.327</td>
<td>1.146</td>
<td>1.559</td>
<td>1.384</td>
<td>1.291</td>
<td>1.173</td>
<td>0.860</td>
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<tr>
<td>200</td>
<td>3.749</td>
<td>3.068</td>
<td>1.792</td>
<td>1.550</td>
<td>2.271</td>
<td>1.937</td>
<td>1.953</td>
<td>1.726</td>
<td>1.224</td>
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<tr>
<td>250</td>
<td>4.669</td>
<td>4.335</td>
<td>2.356</td>
<td>1.955</td>
<td>2.881</td>
<td>2.490</td>
<td>2.615</td>
<td>2.215</td>
<td>1.588</td>
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<tr>
<td>300</td>
<td>5.419</td>
<td>5.402</td>
<td>2.853</td>
<td>2.191</td>
<td>3.288</td>
<td>3.009</td>
<td>3.244</td>
<td>2.704</td>
<td>1.853</td>
</tr>
</tbody>
</table>

## Table 1.5

**TEST B2**

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.159</td>
<td>0.767</td>
<td>0.697</td>
<td>0.809</td>
<td>0.881</td>
<td>0.865</td>
<td>0.563</td>
<td>0.554</td>
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<td>150</td>
<td>2.079</td>
<td>1.334</td>
<td>1.062</td>
<td>1.247</td>
<td>1.729</td>
<td>1.522</td>
<td>1.092</td>
<td>1.010</td>
<td>0.926</td>
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<td>2.034</td>
<td>1.427</td>
<td>1.719</td>
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<td>2.179</td>
<td>1.754</td>
<td>1.466</td>
<td>1.191</td>
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<td>4.056</td>
<td>2.668</td>
<td>1.692</td>
<td>1.921</td>
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<td>2.663</td>
<td>2.284</td>
<td>1.824</td>
<td>1.423</td>
</tr>
</tbody>
</table>

## Table 1.6

**Average from TESTS B1 and B2**

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.312</td>
<td>0.867</td>
<td>0.730</td>
<td>0.742</td>
<td>0.881</td>
<td>0.830</td>
<td>0.646</td>
<td>0.602</td>
<td>0.463</td>
</tr>
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<td>150</td>
<td>2.386</td>
<td>1.668</td>
<td>1.194</td>
<td>1.196</td>
<td>1.644</td>
<td>1.453</td>
<td>1.192</td>
<td>1.092</td>
<td>0.893</td>
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<td>2.551</td>
<td>1.610</td>
<td>1.634</td>
<td>2.424</td>
<td>2.058</td>
<td>1.854</td>
<td>1.596</td>
<td>1.208</td>
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<tr>
<td>250</td>
<td>4.362</td>
<td>3.502</td>
<td>2.024</td>
<td>1.938</td>
<td>3.000</td>
<td>2.576</td>
<td>2.450</td>
<td>2.020</td>
<td>1.506</td>
</tr>
<tr>
<td>300</td>
<td>5.130</td>
<td>4.485</td>
<td>2.505</td>
<td>2.208</td>
<td>3.526</td>
<td>3.130</td>
<td>3.078</td>
<td>2.460</td>
<td>1.754</td>
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### Table I.7 TEST C1

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<th>PT6</th>
<th>PT7</th>
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<th>PT9</th>
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### Table I.9

Average from TESTS C1 and C2

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**TEST D1**

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<th>PT9</th>
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**TEST D2**

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**Average from TESTS D1 and D2**

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<td>1.390</td>
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**TEST E1**

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<td>2.814</td>
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**TESTS E2**

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<th>PT6</th>
<th>PT7</th>
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<th>PT9</th>
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**Average from TESTS E1 and E2**

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<th>PT9</th>
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<th>PT6</th>
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<th>PT8</th>
<th>PT9</th>
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**TEST F2**

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<th>PT6</th>
<th>PT7</th>
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<th>PT9</th>
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### Table I.18

Average from TESTS F1 and F2

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<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
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<tr>
<td>200</td>
<td>3.510</td>
<td>2.351</td>
<td>1.310</td>
<td>1.702</td>
<td>2.271</td>
<td>2.508</td>
<td>2.185</td>
<td>2.152</td>
<td>1.804</td>
</tr>
<tr>
<td>250</td>
<td>4.567</td>
<td>2.968</td>
<td>1.593</td>
<td>1.988</td>
<td>2.780</td>
<td>3.078</td>
<td>2.764</td>
<td>2.710</td>
<td>2.184</td>
</tr>
<tr>
<td>Fill height (mm)</td>
<td>PT1</td>
<td>PT2</td>
<td>PT3</td>
<td>PT4</td>
<td>PT5</td>
<td>PT6</td>
<td>PT7</td>
<td>PT8</td>
<td>PT9</td>
</tr>
<tr>
<td>------------------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td>100</td>
<td>0.579</td>
<td>0.467</td>
<td>0.398</td>
<td>0.303</td>
<td>0.441</td>
<td>0.588</td>
<td>0.132</td>
<td>0.163</td>
<td>0.298</td>
</tr>
<tr>
<td>150</td>
<td>1.806</td>
<td>1.267</td>
<td>0.995</td>
<td>0.775</td>
<td>1.085</td>
<td>1.383</td>
<td>0.794</td>
<td>0.717</td>
<td>0.894</td>
</tr>
<tr>
<td>200</td>
<td>2.692</td>
<td>2.167</td>
<td>1.758</td>
<td>1.247</td>
<td>1.797</td>
<td>2.248</td>
<td>1.555</td>
<td>1.336</td>
<td>1.490</td>
</tr>
<tr>
<td>250</td>
<td>3.340</td>
<td>2.501</td>
<td>1.924</td>
<td>1.382</td>
<td>2.169</td>
<td>2.698</td>
<td>1.985</td>
<td>1.694</td>
<td>1.787</td>
</tr>
<tr>
<td>300</td>
<td>4.226</td>
<td>3.201</td>
<td>2.289</td>
<td>1.618</td>
<td>2.610</td>
<td>3.251</td>
<td>2.581</td>
<td>2.182</td>
<td>2.185</td>
</tr>
</tbody>
</table>

Table I.20

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1.056</td>
<td>0.734</td>
<td>0.597</td>
<td>0.506</td>
<td>0.814</td>
<td>0.830</td>
<td>0.927</td>
<td>0.717</td>
<td>0.463</td>
</tr>
<tr>
<td>150</td>
<td>Reading not taken</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>2.897</td>
<td>2.201</td>
<td>1.261</td>
<td>1.011</td>
<td>1.830</td>
<td>2.006</td>
<td>2.317</td>
<td>1.629</td>
<td>1.257</td>
</tr>
<tr>
<td>250</td>
<td>3.545</td>
<td>2.868</td>
<td>1.592</td>
<td>1.281</td>
<td>2.339</td>
<td>2.560</td>
<td>2.946</td>
<td>2.085</td>
<td>1.555</td>
</tr>
<tr>
<td>300</td>
<td>4.124</td>
<td>3.534</td>
<td>1.858</td>
<td>1.449</td>
<td>2.712</td>
<td>3.009</td>
<td>3.641</td>
<td>2.476</td>
<td>1.754</td>
</tr>
</tbody>
</table>

Table I.21

Average from TESTS G1 and G2

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.818</td>
<td>0.600</td>
<td>0.498</td>
<td>0.404</td>
<td>0.628</td>
<td>0.709</td>
<td>0.530</td>
<td>0.440</td>
<td>0.380</td>
</tr>
<tr>
<td>150</td>
<td>1.840</td>
<td>1.234</td>
<td>1.012</td>
<td>0.876</td>
<td>1.017</td>
<td>1.435</td>
<td>0.910</td>
<td>0.798</td>
<td>0.645</td>
</tr>
<tr>
<td>200</td>
<td>2.794</td>
<td>2.184</td>
<td>1.510</td>
<td>1.129</td>
<td>1.814</td>
<td>2.127</td>
<td>1.936</td>
<td>1.482</td>
<td>1.374</td>
</tr>
<tr>
<td>250</td>
<td>3.442</td>
<td>2.684</td>
<td>1.758</td>
<td>1.332</td>
<td>2.254</td>
<td>2.629</td>
<td>2.466</td>
<td>1.890</td>
<td>1.671</td>
</tr>
<tr>
<td>300</td>
<td>4.175</td>
<td>3.368</td>
<td>2.074</td>
<td>1.534</td>
<td>2.661</td>
<td>3.130</td>
<td>3.111</td>
<td>2.329</td>
<td>1.970</td>
</tr>
</tbody>
</table>
APPENDIX J

CHECK ON PRESSURE TRANSDUCERS

Check on pressure transducers

Analysis:

\[
\text{Measured pressure} = \text{Voltage difference} \times \text{Calibration factor}
\]

<table>
<thead>
<tr>
<th>Pressure transducer</th>
<th>Initial voltage (mV)</th>
<th>Final voltage (mV)</th>
<th>Measured pressure (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>-5.5</td>
<td>9.1</td>
<td>14.6/2.934 = 4.976</td>
</tr>
<tr>
<td>PT2</td>
<td>-7.3</td>
<td>7.5</td>
<td>14.8/2.999 = 4.935</td>
</tr>
<tr>
<td>PT3</td>
<td>-11.1</td>
<td>3.7</td>
<td>14.8/3.014 = 4.910</td>
</tr>
<tr>
<td>PT4</td>
<td>-6.8</td>
<td>7.8</td>
<td>14.6/2.967 = 4.921</td>
</tr>
<tr>
<td>PT5</td>
<td>0.4</td>
<td>15.2</td>
<td>14.8/2.950 = 5.017</td>
</tr>
<tr>
<td>PT6</td>
<td>-9.0</td>
<td>5.2</td>
<td>14.2/2.891 = 4.912</td>
</tr>
<tr>
<td>PT7</td>
<td>-4.4</td>
<td>10.2</td>
<td>14.6/3.021 = 4.833</td>
</tr>
<tr>
<td>PT8</td>
<td>-3.6</td>
<td>11.8</td>
<td>15.4/3.070 = 5.016</td>
</tr>
<tr>
<td>PT9</td>
<td>-4.8</td>
<td>10.3</td>
<td>15.1/3.022 = 4.997</td>
</tr>
</tbody>
</table>
APPENDIX K

VERTICAL PRESSURE DUE TO STRIP LOAD

Note:

Value of strip load surcharge q, for LOAD:

LP  \( q = 1.500 \text{ kPa} \)
SL1  \( q = 1.998 \text{ kPa} \)
SL2  \( q = 2.496 \text{ kPa} \)
SL3  \( q = 2.994 \text{ kPa} \)
SL4  \( q = 4.989 \text{ kPa} \)
SL5  \( q = 6.984 \text{ kPa} \)

Table K.1  TEST C1

<table>
<thead>
<tr>
<th>LOAD</th>
<th>Pressure at gauge point (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PT1</td>
</tr>
<tr>
<td>LP</td>
<td>0.033</td>
</tr>
<tr>
<td>SL1</td>
<td>0.065</td>
</tr>
<tr>
<td>SL2</td>
<td>0.196</td>
</tr>
<tr>
<td>SL3</td>
<td>0.425</td>
</tr>
<tr>
<td>SL4</td>
<td>1.112</td>
</tr>
<tr>
<td>SL5</td>
<td>1.701</td>
</tr>
</tbody>
</table>
### Table K.2

**TEST C2**

<table>
<thead>
<tr>
<th>LOAD</th>
<th>Pressure at gauge point (kPa)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>LP</td>
<td></td>
<td>-0.163</td>
<td>0.067</td>
<td>0.066</td>
<td>0.034</td>
<td>0</td>
<td>0</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL1</td>
<td></td>
<td>-0.065</td>
<td>0.134</td>
<td>0.099</td>
<td>0.068</td>
<td>0</td>
<td>0.034</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL2</td>
<td></td>
<td>0</td>
<td>0.200</td>
<td>0.142</td>
<td>0.068</td>
<td>0.034</td>
<td>0.034</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL3</td>
<td></td>
<td>0.294</td>
<td>0.267</td>
<td>0.142</td>
<td>0.102</td>
<td>0.034</td>
<td>0.069</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL4</td>
<td></td>
<td>0.785</td>
<td>0.967</td>
<td>0.497</td>
<td>0.338</td>
<td>0.169</td>
<td>0.242</td>
<td>0.067</td>
<td>0.032</td>
<td>0.033</td>
</tr>
<tr>
<td>SL5</td>
<td></td>
<td>1.243</td>
<td>1.601</td>
<td>0.730</td>
<td>0.506</td>
<td>0.237</td>
<td>0.380</td>
<td>0.100</td>
<td>0.065</td>
<td>0.033</td>
</tr>
</tbody>
</table>

### Table K.3

**Average from TESTS CI and C2**

<table>
<thead>
<tr>
<th>LOAD</th>
<th>Pressure at gauge point (kPa)</th>
<th>PT1</th>
<th>PT2</th>
<th>PT3</th>
<th>PT4</th>
<th>PT5</th>
<th>PT6</th>
<th>PT7</th>
<th>PT8</th>
<th>PT9</th>
</tr>
</thead>
<tbody>
<tr>
<td>LP</td>
<td></td>
<td>-0.130</td>
<td>0.117</td>
<td>0.083</td>
<td>0.050</td>
<td>0</td>
<td>0.017</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL1</td>
<td></td>
<td>0</td>
<td>0.167</td>
<td>0.116</td>
<td>0.084</td>
<td>0</td>
<td>0.034</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL2</td>
<td></td>
<td>0.196</td>
<td>0.250</td>
<td>0.170</td>
<td>0.118</td>
<td>0.034</td>
<td>0.052</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL3</td>
<td></td>
<td>0.360</td>
<td>0.367</td>
<td>0.187</td>
<td>0.186</td>
<td>0.050</td>
<td>0.086</td>
<td>0.033</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>SL4</td>
<td></td>
<td>0.952</td>
<td>1.250</td>
<td>0.514</td>
<td>0.472</td>
<td>0.220</td>
<td>0.242</td>
<td>0.100</td>
<td>0.052</td>
<td>0.033</td>
</tr>
<tr>
<td>SL5</td>
<td></td>
<td>1.472</td>
<td>2.068</td>
<td>0.746</td>
<td>0.691</td>
<td>0.356</td>
<td>0.380</td>
<td>0.149</td>
<td>0.065</td>
<td>0.033</td>
</tr>
</tbody>
</table>
EFFECT OF SUPPORT SYSTEM ON VERTICAL FORCE

Measured vertical force

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>Measured vertical force (kN/m)</th>
<th>TEST B1</th>
<th>TEST B2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>supported</td>
<td>unsupported</td>
<td>supported</td>
</tr>
<tr>
<td>100</td>
<td>0.4028</td>
<td>0.4486</td>
<td>0.3454</td>
</tr>
<tr>
<td>150</td>
<td>0.7642</td>
<td>0.8160</td>
<td>0.6980</td>
</tr>
<tr>
<td>200</td>
<td>1.0996</td>
<td>1.1748</td>
<td>0.9980</td>
</tr>
<tr>
<td>250</td>
<td>1.4246</td>
<td>1.5382</td>
<td>1.2801</td>
</tr>
<tr>
<td>300</td>
<td>1.7807</td>
<td>1.8429</td>
<td>1.5572</td>
</tr>
</tbody>
</table>

Analysis:

Percentage of vertical force transferred to support system when hollow section is in place is:

\[
\text{Force difference} \times 100\% = \frac{\text{Force when supported} - \text{Force when unsupported}}{\text{Force when supported}}\]

Results

<table>
<thead>
<tr>
<th>Fill height (mm)</th>
<th>Force transferred to support system (%)</th>
<th>TEST B1</th>
<th>TEST B2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>11.4</td>
<td>20.9</td>
<td>16.2</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>6.8</td>
<td>5.3</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>6.8</td>
<td>7.4</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>8.0</td>
<td>3.4</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>3.5</td>
<td>4.9</td>
<td>4.2</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX M  

RESISTANCE OF SHEAR BOX

The resistance of shear box in term of friction angle

Analysis:

Comparison is made between the friction angles obtained for the sample when no correction for shear box resistance is applied and when the correction is applied. The difference is the resistance of the shear box in term of friction angle $\phi_{SB}$.

<table>
<thead>
<tr>
<th>Reinforced sand with strip of</th>
<th>Uncorrected friction angle</th>
<th>Corrected Friction angle</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w = 10$mm</td>
<td>43.8°</td>
<td>43.4°</td>
<td>0.4°</td>
</tr>
<tr>
<td>$w = 15$mm</td>
<td>49.5°</td>
<td>48.8°</td>
<td>0.7°</td>
</tr>
<tr>
<td>$w = 25$mm</td>
<td>48.5°</td>
<td>48.3°</td>
<td>0.2°</td>
</tr>
<tr>
<td>$w = 30$mm</td>
<td>43.0°</td>
<td>42.8°</td>
<td>0.2°</td>
</tr>
<tr>
<td>$w = 40$mm</td>
<td>55.6°</td>
<td>55.0°</td>
<td>0.6°</td>
</tr>
<tr>
<td>$w = 45$mm</td>
<td>50.2°</td>
<td>49.6°</td>
<td>0.6°</td>
</tr>
<tr>
<td>$w = 50$mm</td>
<td>45.4°</td>
<td>44.8°</td>
<td>0.6°</td>
</tr>
</tbody>
</table>

\[
\text{mean} = 0.5^\circ \\
\sigma_{n-1} = 0.2^\circ
\]

Therefore:

$\phi_{SB} = 0.5^\circ$
KEITHLEY/SOFT 500 PROGRAM TO MEASURE VOLTAGE CHANGES

Note:

VA applies to LVDT that measures shear displacement

VB applies to LVDT that measures vertical displacement

VC applies to LVDT that measures shear force

10 DIM VALUE(500,4)
20 VO=0:V1=0:V2=0
30 VA=0:VB=0:VC=0
50 CALL INIT
60 CALL IONAME('"V0",6,0,12,1)
70 CALL IONAME('"V1",6,1,12,1)
80 CALL IONAME('"V2",6,2,12,1)
90 CLS
100 LOCATE 5,5:INPUT "INPUT FILENAME[B:\VOLTAGE\]",F$
110 FILE$="B:\VOLTAGE\"+F$
120 OPEN "O",£3,FILES
130 D$="":T$=""
140 D$=DATE$:T$=TIME$
150 CLS
160 PRINT £3,"DATE: ";D$;" FILE ";F$
170 PRINT £3,"START TIME: ";T$
180 CLS
190 CALL ANREAD('"V0",VA,0)
200 CALL ANREAD('"V1",VB,0)
210 CALL ANREAD('"V2",VC,0)
220 VALUE(1,1)=0:VALUE(1,2)=VA:VALUE(1,3)=VB:VALUE(1,4)=VC
230 CLS
240 LOCATE 5,5:PRINT "PRESS ANY KEY TO CONTINUE"
250 I$=INKEY$:IF I$="" THEN 250
260 ST=TIMER
270 COUNT%=1
280 CALL ANREAD('"V0",VA,0)
290 CALL ANREAD('"V1",VB,0)
300 CALL ANREAD('"V2",VC,0)
310 LOCATE 21,5:PRINT USING "+££££.££££ ";VA,VB,VC
320 T=TIMER
330 DELT=TIMER-ST
340 COUNT%=COUNT%+1
350 VALUE(COUNT%,1)=DELT:VALUE(COUNT%,2)=VA:VALUE(COUNT%,3)=VB:VALUE(COUNT%,4)=VC
360 I$=INKEY$: IF I$="" THEN 280 ELSE 370
370 PRINT£3,COUNT%
380 FOR J=1 TO COUNT%
390 PRINT £3, USING "+££££.££££ ";VALUE(J,1);VALUE(J,2);VALUE(J,3);VALUE(J,4)
400 NEXT J
410 CLOSE £3
420 PRINT "DATA ACQUISITION RUN COMPLETED"
430 END
APPENDIX 0

RELATIVE DENSITY OF SAND IN SHEAR BOX

Relative density, $D_r = \frac{\gamma_{d_{\text{max}}}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d_{\text{max}}}}{\gamma_{d_{\text{max}}} - \gamma_{d_{\text{min}}}} \times 100\%

= \frac{16.44}{15.64} \times \frac{15.64 - 14.56}{16.44 - 14.56} \times 100\%

= 60\%

where

$\gamma_{d_{\text{max}}}$ is dry unit weight of soil in densest condition

$\gamma_{d_{\text{min}}}$ is dry unit weight of soil in loosest condition

$\gamma_d$ is in-place (insitu) dry unit weight
### A DIRECT SHEAR TEST

### AN EXAMPLE OF VOLTAGE MEASUREMENT IN A DIRECT SHEAR TEST

**DATE:** 06-14-1989 **FILE se8O**

**START TIME:** 20:33:22

| +0.0000  | -4.4115 | +2.4701 | -0.3553 |
| +0.4453  | -4.2674 | +2.4676 | -0.2583 |
| +0.7734  | -4.2234 | +2.4652 | -0.1900 |
| +1.0547  | -4.2039 | +2.4676 | -0.1575 |
| +1.3828  | -4.2210 | +2.4823 | -0.1355 |
| +1.6563  | -4.2015 | +2.4676 | -0.1258 |
| +1.9844  | -4.1746 | +2.4676 | -0.1003 |
| +2.3750  | -4.1209 | +2.4579 | -0.0940 |
| +2.6953  | -4.1013 | +2.4676 | -0.0745 |
| +3.0234  | -4.0256 | +2.4701 | -0.0476 |
| +3.4141  | -3.9817 | +2.4676 | -0.0330 |
| +3.7422  | -3.9719 | +2.4676 | -0.0159 |
| +4.0156  | -3.9328 | +2.4701 | -0.0085 |
| +4.3438  | -3.9011 | +2.4652 | -0.0085 |
| +4.7344  | -3.8767 | +2.4701 | -0.0037 |
| +5.0000  | -3.8523 | +2.4676 | -0.0012 |
| +5.3806  | -3.8156 | +2.4676 | +0.0012 |
| +5.7188  | -3.7717 | +2.4652 | +0.0037 |
| +5.9922  | -3.7839 | +2.4701 | +0.0159 |
| +6.3750  | -3.7326 | +2.4676 | +0.0256 |
| +6.6563  | -3.6496 | +2.4774 | +0.0427 |
| +6.9854  | -3.6105 | +2.4652 | +0.0579 |
| +7.2896  | -3.5495 | +2.4676 | +0.0672 |
| +7.6406  | -3.4933 | +2.4652 | +0.0769 |
| +8.0234  | -3.4933 | +2.4701 | +0.0769 |
| +8.4141  | -3.4420 | +2.4676 | +0.0818 |
| +8.7422  | -3.4103 | +2.4676 | +0.0818 |
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| +9.4063  | -3.3663 | +2.4676 | +0.0891 |
| +9.7344  | -3.3248 | +2.4676 | +0.0867 |
| +10.0000 | -3.2735 | +2.4701 | +0.1013 |
| +10.3281 | -3.2369 | +2.4676 | +0.1136 |
| +10.6641 | -3.2027 | +2.4676 | +0.1184 |
| +10.9927 | -3.1885 | +2.4676 | +0.1282 |
| +11.3203 | -3.1612 | +2.4701 | +0.1355 |
| +11.6563 | -3.1270 | +2.4701 | +0.1355 |
| +11.9844 | -3.0952 | +2.4676 | +0.1300 |
| +12.3594 | -3.0391 | +2.4652 | +0.1111 |
| +12.6406 | -3.0342 | +2.4676 | +0.1355 |
| +12.8688 | -2.9951 | +2.4701 | +0.1380 |
| +13.3047 | -2.9585 | +2.4676 | +0.1429 |
| +13.6250 | -2.8952 | +2.4701 | +0.1800 |
| +14.0156 | -2.8315 | +2.4676 | +0.1746 |
| +14.3438 | -2.8046 | +2.4701 | +0.1868 |
| +14.6984 | -2.7607 | +2.4676 | +0.1893 |
| +15.0453 | -2.7534 | +2.4676 | +0.1893 |
| +15.2734 | -2.7143 | +2.4701 | +0.1893 |
| +15.5547 | -2.6947 | +2.4701 | +0.1893 |
| +15.9297 | -2.6410 | +2.4676 | +0.1893 |
| +16.2109 | -2.5971 | +2.4652 | +0.1893 |
| +16.5391 | -2.5897 | +2.4676 | +0.1893 |
| +16.9219 | -2.5531 | +2.4701 | +0.1893 |
| +17.2031 | -2.5043 | +2.4676 | +0.1893 |
| +17.5234 | -2.4896 | +2.4701 | +0.2039 |
| +17.9141 | -2.4432 | +2.4701 | +0.2112 |
| +18.2422 | -2.3944 | +2.4701 | +0.2186 |
| +18.5703 | -2.3724 | +2.4701 | +0.2234 |
| +18.9531 | -2.3358 | +2.4701 | +0.2234 |
| +19.2813 | -2.2845 | +2.4652 | +0.2259 |
| +19.6094 | -2.2552 | +2.4701 | +0.2234 |
| +20.0000 | -2.1941 | +2.4676 | +0.2234 |
| +20.3261 | -2.1648 | +2.4676 | +0.2210 |
FORTRAN PROGRAM TO TRANSFORM VOLTAGE MEASUREMENTS INTO DATA FORM

```fortran
DIMENSION T(500), VA(500), VB(500), VC(500), X(500), Y(500)
DIMENSION XAV(500), YAV(500), YAVC(500)
READ(*,*)
READ(*,*)
READ(*,*)N
DO 3 I = 1, N
READ(*,*) T(I), VA(I), VB(I), VC(I)
X(I) = (VA(I) - VA(1))/0.1507
Y(I) = (VC(I) - VC(1))/0.0004
WRITE(*,*) X(I), Y(I)
CONTINUE
XAV(1) = 0.0
YAV(1) = 0.0
YAVC(1) = 0.0
J = 1
ISTOP = 1
SUMX = 0.0
SUMY = 0.0
J = J + 1
ISTART = ISTOP + 1
ISTOP = ISTART + 4
IF (ISTOP .GE. N) THEN
REM = (N - ISTART) + 1
WRITE(*,*) 'N', N, 'ISTART', ISTART, 'REM', REM, 'ISTOP', ISTOP
DO 8 I = ISTART, N
SUMX = SUMX + X(I)
SUMY = SUMY + Y(I)
CONTINUE
XAV(J) = SUMX/REM
YAV(J) = SUMY/REM
YAVC(J) = YAV(J) - 343.736 - 2.1509*XAV(J)
GOTO 6
END IF
DO 10 I = ISTART, ISTOP
SUMX = SUMX + X(I)
SUMY = SUMY + Y(I)
CONTINUE
XAV(J) = SUMX/REM
YAV(J) = SUMY/REM
YAVC(J) = YAVC(J) - 343.736 - 2.1509*XAV(J)
GOTO 5
30 WRITE(*,*) XAV(K), YAV(K), YAVC(K)
CONTINUE
STOP
END
```
**APPENDIX P3**

**AN EXAMPLE OF EXPERIMENTAL DATA**

<table>
<thead>
<tr>
<th>Shear Displacement (mm)</th>
<th>Shear Force (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.00000000</td>
<td>.00000000</td>
</tr>
<tr>
<td>.34573656</td>
<td>106.77500</td>
</tr>
<tr>
<td>.59176368</td>
<td>305.22500</td>
</tr>
<tr>
<td>1.1030040</td>
<td>455.37500</td>
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<tr>
<td>1.6093024</td>
<td>547.55000</td>
</tr>
<tr>
<td>2.2671513</td>
<td>619.00000</td>
</tr>
<tr>
<td>3.0573644</td>
<td>662.35000</td>
</tr>
<tr>
<td>3.8194768</td>
<td>697.75000</td>
</tr>
<tr>
<td>4.5576550</td>
<td>756.37500</td>
</tr>
<tr>
<td>5.3953488</td>
<td>781.97500</td>
</tr>
<tr>
<td>6.2968023</td>
<td>810.67500</td>
</tr>
<tr>
<td>7.1156008</td>
<td>879.67500</td>
</tr>
<tr>
<td>8.0763564</td>
<td>916.89999</td>
</tr>
<tr>
<td>8.9801357</td>
<td>957.19999</td>
</tr>
<tr>
<td>9.9456395</td>
<td>1015.85000</td>
</tr>
<tr>
<td>10.598934</td>
<td>1029.2750</td>
</tr>
<tr>
<td>11.438760</td>
<td>1056.7250</td>
</tr>
<tr>
<td>12.397093</td>
<td>1115.3500</td>
</tr>
<tr>
<td>13.459593</td>
<td>1125.1000</td>
</tr>
<tr>
<td>14.354070</td>
<td>1169.0500</td>
</tr>
<tr>
<td>15.544283</td>
<td>1212.4250</td>
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<tr>
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<tr>
<td>17.960368</td>
<td>1284.4250</td>
</tr>
<tr>
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<td>20.468604</td>
<td>1344.8750</td>
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<tr>
<td>21.500290</td>
<td>1354.0500</td>
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<tr>
<td>22.683333</td>
<td>1392.5250</td>
</tr>
<tr>
<td>24.072383</td>
<td>1407.1500</td>
</tr>
<tr>
<td>25.333623</td>
<td>1439.5250</td>
</tr>
<tr>
<td>26.351162</td>
<td>1430.9500</td>
</tr>
</tbody>
</table>
APPENDIX Q

CALIBRATION OF LVDTs
Calibration chart for LVDT that measures shear displacement

\[
y = -2.4060 + 0.15073x \quad R^2 = 1.000
\]

Calibration Factor = 0.1507 V/mm
Calibration chart for LVDT that measures vertical displacement

\[ y = -1.0340 + 0.90304x \]

\[ R^2 = 1.000 \]

Calibration Factor = 0.9030 V/mm
Calibration chart for LVDT that measures shear force

\[ y = -0.30050 + 4.3914 \times 10^{-4} x \quad R^2 = 0.999 \]

Calibration Factor = 0.0004 V/N
APPENDIX R

PLOTS OF SHEAR STRESS VERSUS NORMAL STRESS

(related to Section 6A of Chapter 6)

note: equation below plot is the result of regression analysis and the label underneath identifies the sample
Unreinforced Sand

y = 1.6436 + 0.87035x  \ R^2 = 0.946

Strip Width 10mm

y = 1.7600 + 1.1068x  \ R^2 = 0.935
\[ y = 1.0377 + 1.1775x \quad R^2 = 0.965 \]

Strip Width 15mm

\[ y = 2.4750 + 1.0349x \quad R^2 = 0.877 \]

Strip Width 20mm
Normal Stress (kPa)

\[ y = 3.9882 + 0.92583x \quad R^2 = 0.926 \]

Strip Width 30mm

Normal Stress (kPa)

\[ y = 1.3073 + 1.0557x \quad R^2 = 0.896 \]

Strip Width 60mm
APPENDIX S

PLOTS OF SHEAR STRESS VERSUS NORMAL STRESS

(related to Section 6B of Chapter 6)
Normal Stress (kPa)

Shear Stress (kPa)

\[ y = -0.25381 + 1.3454x \quad R^2 = 0.943 \]

Strip Width 35mm

\[ y = -0.40801 + 1.4280x \quad R^2 = 0.933 \]

Strip Width 40mm
Shear Stress (kPa)

Normal Stress (kPa)

\[ y = 2.2266 + 1.1771x \quad R^2 = 0.963 \]

Strip Width 45mm

Shear Stress (kPa)

Normal Stress (kPa)

\[ y = 2.5115 + 0.99373x \quad R^2 = 0.922 \]

Strip Width 50mm
APPENDIX T

PLOTS OF SHEAR STRESS VERSUS NORMAL STRESS

(related to Section 6C of Chapter 6)
Shear Stress (kPa)

Normal Stress (kPa)

\[ y = 2.0139 + 1.0450x \quad R^2 = 0.949 \]

Smooth Strip

Shear Stress (kPa)

Normal Stress (kPa)

\[ y = -2.9260 + 1.5467x \quad R^2 = 0.954 \]

Ribbed Strip
APPENDIX U1

PLOTS OF SHEAR STRESS VERSUS NORMAL STRESS

(related to Section 6D of Chapter 6)
Dry reinforced sand

\[ y = 2.0139 + 1.0450x \quad R^2 = 0.949 \]
Normal Stress (kPa)

\[ y = -0.97040 + 1.0671x \quad R^2 = 0.790 \]
Submerged Unreinforced Sand

Submerged Reinforced Sand

\[ y = -0.57930 + 1.3053x \quad R^2 = 0.925 \]
Analysis:

At a specified applied normal pressure \( \sigma \), the pore water pressure \( u \), in the submerged sample is:

\[
u = \sigma - \frac{\tau}{\tan \phi_{(dry)}}
\]

where:
- \( \tau \) is shear strength obtained in the test for the submerged sample at applied normal pressure \( \sigma \).
- \( \phi_{(dry)} \) is internal friction angle of dry sample.

Sample Calculation

Reinforced submerged sample at \( \sigma = 14.44 \text{kPa} \)

relevant information:
- shear strength obtained from test = 18.84kPa
- \( \tan \phi_{(dry)} \) obtained from direct shear test of dry reinforced sample is = 1.0450

Thus the pore water pressure is:

\[
u = \frac{14.44 - 18.84}{1.0450} = -3.60 \text{ kPa}
\]
PORE WATER PRESSURE FOR SUBMERGED REINFORCED AND SUBMERGED UNREINFORCED SAMPLES AT VARIOUS APPLIED CONFINING PRESSURES

<table>
<thead>
<tr>
<th>Applied Normal Pressure $\sigma$ (kPa)</th>
<th>Pore Water Pressure $u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reinforced sand</td>
</tr>
<tr>
<td>6.74</td>
<td>-1.68</td>
</tr>
<tr>
<td>7.86</td>
<td>-1.96</td>
</tr>
<tr>
<td>8.98</td>
<td>-2.24</td>
</tr>
<tr>
<td>10.08</td>
<td>-2.51</td>
</tr>
<tr>
<td>11.16</td>
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</tr>
<tr>
<td>12.26</td>
<td>-3.03</td>
</tr>
<tr>
<td>13.34</td>
<td>-3.32</td>
</tr>
<tr>
<td>14.44</td>
<td>-3.60</td>
</tr>
</tbody>
</table>
APPENDIX U3

RESULTS OF ADDITIONAL TESTS ON SUBMERGED SAMPLES (tested under low shearing rate)
Dry Unreinforced Sand

\[ y = 0.59142 + 0.92598x \quad R^2 = 0.937 \]

Submerged Unreinforced Sand

\[ y = 1.0953 + 0.95896x \quad R^2 = 0.957 \]
ERRATA

1) Include these in the "References":


2) page 8-7: change "Valliappan et al. (1980)" to "Valliappan et al. (1978)"

3) page 8-14: change "Figure 7.30a" to "Figure 8.8a"

4) page 2-7: change "were" - 6th line from bottom to "are"

5) page 2-18: change "affective" - 2nd line from bottom to "effective"
6) page 2-34: change "was" - top line to "were"

7) page 2-45: change \( \sigma_{10} \) in Eq. 2.15 to \( \sigma_1 \)

8) page 3-39: the LHS of Eq. 3.21 is:

\[
T_{max} =
\]

9) page 4-41: change "4.075kPa" - 5th line from top to "3.946kPa"

10) page 6.74: change "avery" - 2nd line from bottom to "every"

11) page R-6: change "thery" - 2nd line from top to "theory"