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STRENGTH AND DEFORMATION BEHAVIOUR OF GEOGRID REINFORCED SOILS

by

YIJUN ZHAO

B. S. (Hons), Northeast University of Technology, 1982

M. S. Northeast University of Technology, 1986

A thesis Submitted to the
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in partial fulfilment of the requirement
for the degree of
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in
Civil Engineering*

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OF GEOGRID REINFORCED SOILS

submitted by

Yijun Zhao, B.S., M.S.

in partial fulfillment of the requirements
for the degree of Doctor of Philosophy

Chair, Department of Civil and Environmental Engineering

Thesis Supervisor

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June, 1993

ABSTRACT

A literature review has revealed that there is a substantial lack of data on shear strength and deformation behaviour of reinforced soil from direct shear tests. Furthermore, most of the existing experimental results were obtained from small test facilities and scale or close physical boundaries of the equipment seem to influence the test results. In order to address these limitations and to develop a data base as well as to establish some theoretical framework based on the test results from large scale test apparatuses, this research was initiated. Two large apparatuses were designed and built. An extensive experimental program was conducted. In the first stage of the experimental program, 64 preliminary pullout tests in a small test rig were carried out to investigate the properties of Conwed Stratagrids and to select a proper geogrid for further investigation. The second stage of the test program consisted of over 100 large direct shear and 15 large pullout tests. The parameters investigated were: soil properties, geogrid properties, orientation of the geogrid to the shear plane and normal stresses. A theoretical analysis was conducted to model the load transfer in direct shear and in pullout between soil and reinforcement. It was shown that the predictions from the proposed models correlated well with experimental results. For the interpretation of test results a flow rule analysis was adopted. It was shown that the flow rule analysis yielded a better explanation of and correlation to the experimental data as compared to a conventional analysis. It is necessary to point out, however, that a reinforced soil system depends on the strength and deformation mobilized in each component and on the strain compatibility (or slip) at the interface. The tensile strain mobilized in the soil, in the direction of the reinforcement, is the source of the tensile strains and forces developed in the reinforcement. The orientation of the reinforcement and the interface friction play an important role in the process.

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STRENGTH AND DEFORMATION BEHAVIOUR OF GEOGRID REINFORCED SOILS

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CHAPTER 1

INTRODUCTION

1.1. GENERAL

The concept of reinforcing soil is old. The basic principles are used in nature by some animals and birds in the construction of their habitat. Use of these techniques by man is known to have existed in the 5th and 4th millenniums B. C. (Jones 1985). The modern form of reinforced soil was introduced by Henri Vidal in France in the 1960's. The term "reinforced earth" was first used by him to describe the earth structures reinforced with flat steel strips (Fig. 1.1) and it is also a registered trademark of his company (i. e. Terre Armée). Two distinct stages can be identified in the recent history of reinforced soil. Initially, walls were built from metallic strips, granular fill and facing panels demonstrating the economic and technical benefits of reinforced soil. The concept worked well in practice and attracted wide commercial and academic interest. This interest coupled with the developments of modern polymer materials then led to the diverse range of reinforcements and application for reinforced soil that we see today (Jewell 1990).

A review of literature indicates that although successful and extensive practical application of reinforced earth, as well as research on these topics has been conducted, a complete understanding of the fundamental mechanism has not been established. This is especially true for soils reinforced with geogrids. In order to establish a rational and theoretical framework to interpret the behaviour of earth systems reinforced by geogrids

a thorough experimental study and theoretical (or numerical) analysis should be conducted to investigate the influence of the soil type, properties, the reinforcement type, dimensions and layout of the reinforcement on the overall behaviour of reinforced soil systems.

Unit cell tests, model simulation and performance monitoring of proto-type structures are three stages in the study of the behaviour of any soil structure. A direct analogy between the direct shear test, pullout test and the behaviour of a reinforced earth structure exists (Fig. 1-2). Unit cell tests, i.e. pullout and modified direct shear tests play an important role in the study of the behaviour of reinforced earth. Furthermore these tests can be carried out in a carefully controlled and repeatable fashion to investigate the influence of many factors and to separate the combined effect by changing one variable at a time. It was decided that both pullout and direct shear tests should be used in this dissertation in order to obtain fundamental parameters and to gain a better understanding of the problem.

Due to scale effects and boundary effects, test results from small facilities can hardly be extrapolated to the behaviour of a prototype structure. Only performance monitoring of actual reinforced structures will overcome the problem of scale effects but it is associated with other problems such as high cost and lack of control of parameters. However, "an experiment using a large scale apparatus under controlled conditions can act as a compromise between the two alternatives" (Shang 1989). In order to minimize the scale effect, two large testing facilities were designed or modified. "The main advantages of a large scale test are the use of site specific soils and reinforcement

materials (minimizing the need for scaling) and the reduction of influences caused by boundary conditions" (Palmeira et al. 1987).

Modelling the load transfer mechanism between soil and reinforcement using stress and strain states during the shear deformation or pullout process is extremely complex, especially when dealing with reinforcement elements, like geogrids. A sophisticated three dimensional finite element method could be one solution for simulating this phenomenon but the mathematical formulation and calculation algorithm is very complicated. A more practical approach is to use simple limit equilibrium models verified by experimental results, especially by the test data from a large experimental apparatus. It seems that the limit equilibrium analysis can be used effectively as a first order of approximation. This approach was adopted in this thesis to investigate the load transfer mechanism in direct shear and in pullout tests . These form the main theoretical framework of the analysis.

1.2. OBJECTIVES

The main objectives of this research are:

1. To study the shear strength and deformation behaviour of unreinforced granular soils, including the mobilized shear strength parameters of granular soils in direct shear and the deformation behaviour of natural granular soils under shearing. The soil deformation behaviour can be identified as (a) soil dilatancy (plastic volume expansion or contraction) and (b) deformation pattern in the shear zone. This study also aimed to investigate the relation between mobilized shear resistance and soil dilatancy, i. e. flow rule of the natural granular soil.

2. To investigate the shear strength and deformation behaviour of geogrid reinforced soil. Of particular interests are:

(1) the mobilized shear strength parameters of reinforced granular soils, which can be affected by the reinforcement properties and the reinforcement orientation with respect to shear plane;

(2) the deformation behaviour of soil reinforced with geogrids considering the following aspects: (a) soil dilatancy, (b) deformation pattern of the soil in the vicinity of the reinforcement, (c) displacement and force distribution in the reinforcement, and (d) the strain compatibility between soil and reinforcement;

(3) the relationship between shear strength and deformation characteristics of reinforced soil.

3. To study the interaction mechanism between soil and reinforcement in a pullout test (anchorage capacity) from (a) the pullout resistance of a geogrid, (b) the displacement and strain distribution in the geogrid, (c) the mobilized tensile force and its distribution in the geogrid and (d) the embedded geogrid length.

4. To investigate other effects on shear strength and deformation behaviour of soil reinforced with geogrids such as (a) soil density and soil compaction, (b) the relationship between shear strength increase in reinforced soil and soil density and (c) the repeatability of test results.

In order to achieve the objectives of this thesis, this investigation involved the following main tasks:

1. Design of Test Facilities

Based on an extensive literature review of current available test procedures and facilities to evaluate the effect of reinforcement on the mechanical behaviour of soils, a large test facility was built, and instrumented to meet the requirements of the study. The equipment included a large direct shear box (1000x1000x940 mm) and a pullout tank (1072x720x1000 mm).

2. Extensive Experimental Program

Using the large shear box and the modified pullout tank an extensive experimental program was conducted to evaluate the effect of two geogrids on the strength and deformation behaviour of coarse sand and crushed stone aggregate.

3. Development of Data Analysis Procedure and Data Base

Data analysis and interpretation schemes were developed for the extensive experimental programs.

4. Theoretical Analysis

An analytical analysis was carried out to investigate the changes of mechanical properties such as strength and deformation behaviour of soil, as well as the failure mechanism, when reinforcement is introduced into the soil in order to relate these changes to the reinforcement properties and orientation.

1.3. OUTLINE OF THESIS

The first part consists of the introduction and literature review. The background, necessity of this research, the objectives and scope of this study are discussed in Chapter 1, then a brief literature review on the direct shear and pullout tests are summarized in

Chapter 2. Chapter 3 is the theoretical modelling of the direct shear and pullout tests. The main concerns are the dilatancy behaviour of the granular soil and the relationship of mobilized tensile strain in the geogrid and the increase of shear strength and pullout resistance in reinforced soil system. Chapter 4 of this thesis describes all the details of the direct shear box apparatus and pullout test apparatus (a small test rig and large test rig), test setup, test preparation and test procedures. One of the features of this experimental research was to use a large direct shear apparatus and a large pullout test facility. Chapter 4 also describes the design consideration of these two apparatuses. The properties of tested materials, properties of soils and properties of the geogrids are presented in Chapter 5. The shear strength properties of the tested soils was part of the experimental results. All the test results from direct shear tests and pullout tests are presented in Chapter 6 and Chapter 7, respectively. The theoretical analysis and discussion of the test results of direct shear and pullout tests are given in Chapters 8 and 9, respectively. The last chapter is Chapter 10 which summarizes the present work. Finally the cited references are listed.

Fig. 1.1 Reinforced earth structure

Fig. 1.2. Analogy of direct shear test and pullout with R/F soil structure
(after Palmeira and Milligan 1989)

CHAPTER 2

LITERATURE REVIEW

2.1. INTRODUCTION

Two kinds of so called "unit cell tests", the pullout test and the modified direct shear test, have been widely used to investigate soil reinforcement interaction properties. Studies on reinforced soil using the triaxial apparatus, in which specimens of soil were reinforced with "disks" and tested in triaxial compression, were reported by Yang (1972) and Holtz et al. (1982). The mechanism of interaction between soil undergoing axisymmetrical compression with "disk" reinforcement and that for soil undergoing plane strain deformation with mesh or sheet reinforcement is likely to be different. Furthermore, the reinforced soil which is commonly used in a structure deforms under overall conditions of plane strain, such as a retaining wall or an embankment (Fig. 1.2). Therefore, the results from studies on reinforced soil in the triaxial apparatus are likely to be relevant only to a limited extent for such a problem. This study is mainly concerned with tests under quasi plane strain conditions, such as in pullout and in direct shear tests.

In practice two different types of materials have been used in reinforced earth structures, namely "inextensible" and "extensible" reinforcements (Schlosser et al. 1985; Giroud 1986; Schlosser and Delage 1987). The theoretical concepts were introduced by McGown et al. (1978) and were refined by Bonaparte and Schmertmann (1988). They showed that steel reinforcement meets the criterion for inextensible reinforcement, whereas geotextiles and geogrids belong to the second category. Studies on the use of inextensible reinforcement, such as metal strips and sheets, (Schlosser and Elias 1978;

Forsyth 1978; Elias 1979; Schlosser et al. 1983) have shown that the effect of corrosion on long term durability of metal inclusions limited their application and led to the use of polymer extensible material, which marked the second historical stage of reinforced earth (Jewell 1990).

A wide variety of polymer reinforcements are now available for use in reinforced earth structures (Bonaparte et al. 1987). They have different mechanical and geometric properties and these will determine the interactive behaviour between soil and reinforcement. Consequently they will influence the testing methodology and interpretation of test results. In selecting a reinforcement for a reinforced earth structure, the following properties and behaviour of reinforcement should be considered:

- (1) Stress-strain relationship and creep behaviour of the reinforcement,
- (2) Pullout resistance of reinforcement,
- (3) Shearing behaviour of the reinforcement, especially the results from a sliding test,
- (4) Orientation of reinforcement to shear plane and its effect on shear behaviour.

2.2. PULLOUT RESISTANCE OF REINFORCED SOIL

One of the major factors that control the performance of a reinforced soil structure is the interaction between the soil and the reinforcement. Bonding between the two can be accomplished by (1) lateral friction, (2) interlocking, (3) passive earth pressure on transverse elements, (4) the effect of restrained dilatancy on normal stress at the interface and (5) a combination of the above.

The mechanism of soil and reinforcement interaction has been discussed by many investigators based on experimental results and theoretical analysis (Ingold 1982, 1983, 1985; Juran et al. 1988,1989). The pullout test has been commonly accepted as an effective method to evaluate the pullout resistance or bond resistance of an embedded reinforcement. In a pullout test, the embedded end of the specimen is unattached while the front end is clamped to a pullout mechanism. The test is used to obtain a pullout force - displacement relationship of the specimen as it is pulled through the soil. There is no standard test or test apparatus. Box dimensions and testing procedures differ depending on the research institute. A summary was given by Farrag (1990) in which most of the available equipment and test parameters were reviewed. Figures 2.1 to 2.4 shows some typical pullout test equipment (Farrag 1990).

Most of the pullout tests were carried out in small scale apparatuses (Ingold 1982, 1983; Juran 1986; Koerner 1987). The size of the tank governs the boundary conditions. The boundary conditions, in turn, have an influence on the test results. Because of the small size the test results from small test apparatus are limited probably to provide information for the interpretation of the load transfer mechanism and field performance of the reinforced soil. The advantage of using a large apparatus is that coarse granular soils compacted to field specifications and representative specimens of reinforcement can be used. A large tank will also minimize influences from boundary conditions and the results could be extrapolated to field performance of the reinforced soil. Therefore, the data from large pullout tests are of particular significance in the assessment of a proposed hypothesis and for design parameters.

Several large pullout tests were carried out in the past few years (Department of Civil Engineering of California 1984; Mowafy 1986; Koerner 1987; Palmeira 1988, 1989; Shang 1989). A large pullout test facility was built at the University of California in 1984 (Department of Civil Engineering University of California 1984). The test apparatus had overall dimensions of 1370×900×510 mm. The maximum overburden load which could be simulated was about 15 m of fill. The pullout rate was in the order of 4 mm/min and was controlled manually. The soil used in that study was medium to coarse sand and the reinforcement was Tensar SR-2, a uniaxial geogrid. All the geogrids had the same width of 0.61 m, but were of different lengths in order to evaluate the anchorage resistance. Although the report made several interesting observations and it concluded that the normal pressure increase, caused by dilation at the reinforcement interface, was effective in increasing the pullout resistance of the geogrid. However, the proper mechanism of soil-grid interaction was not discussed and no theoretical analysis was carried out.

Mowafy (1986) conducted a study of the pullout resistance of mesh type reinforcement at Carleton University. Steel meshes, plastic geogrids and glass grid were tested. The pullout box was 1.02×0.23×0.38 m. The vertical load was applied by lead and steel shot. The pullout load was applied by an electric motor through a gear box. The rate of pullout was kept constant at 1 mm/min. The horizontal movement of the mesh was monitored by a LVDT and the pullout force was measured by a load cell. Data was recorded with a x-y plotter. Mowafy divided the mechanism of interaction between soil and reinforcement into two components: the primary mechanism and the secondary

mechanism. The primary mechanism is concerned with mechanical properties of the mesh, interlock between the mesh and the soil as well as the confinement provided by the mesh; the secondary mechanism is concerned with interface friction between the mesh and the soil. For tested Tensar geogrids (SS1 and SS2 and ARI) 70% of the total maximum pullout force was due to interlock and confinement while the remainder was due to friction only. The interlock and confinement resistance components depend on the mesh opening and soil particle size. In order to achieve optimum effect it was recommended that the soil particle be equal to $1/4$ to $1/3$ of the smallest mesh aperture dimension. It was also indicated that the dry density has a significant effect on the value of the interlock resistance of the mesh reinforced soil. The research provided some very interesting ideas but the problems that remained were the relative small size of the test apparatus and the very low and narrow range of the surcharge.

In 1988 Costalonga (1988) carried out pullout tests at the University of Alberta with cohesive soil. The pullout box had a dimension of $1060 \times 360 \times 200$ mm. The soil used was a well graded silty clay. Three geogrids were employed: Tensar SR2, Signode /ITW TNX5001 and ParaGrid 50S. The embedded geogrids had dimensions of 350 mm wide and 1000 mm long. The maximum vertical surcharge applied was 100 kPa and the pullout rate of the test was 12 mm/min. The tests were conducted under consolidated-undrained conditions. The pullout force, the progressive horizontal displacement, the passive force at the front face of the soil sample and vertical displacement during tests were monitored. The test results showed that in all tests, the failure occurred by pullout rather than by rupture of the geogrids. Also, the pullout resistance developed at a faster rate, in term of

displacement, for geogrids that had a high tensile modulus. It was concluded that the maximum pullout force was mobilized when the whole geogrid specimen started to move. It was found that an average of 11% of the maximum pullout force was transmitted to the soil by passive soil resistance. The magnitude of the vertical displacement was very small, about 2% of the soil height. The drawbacks of this research were the very high rate of pullout and narrowness of the test tank.

Palmeira et al. (1988,1989) described a large instrumented direct shear and pullout test box (1100×1100×1100 mm), built at Oxford University. The pullout load was supplied by an hydraulic jack and the vertical load was applied by a rubber bag filled with water and connected to an air-water interface system to keep the pressure constant throughout the test. The vertical pressure was measured by a mercury manometer and the pullout load by a load cell within the jack. Pressure cells located flush on the inside walls of the box allowed for the measurement of the total stress distribution on the sides of the box. The pullout displacement at the pulling end of the reinforcement was obtained with a LVDT. Steel cable of 3 mm in diameter fixed to the transverse members of polymer grids were used to measure the displacement of the grid. Total volume variations of the compacted soil could be obtained by the measurement of water volume expelled from the rubber bag on top of the sample. Load cells were embedded in the soil below and above the reinforcement to monitor the stress distribution around the reinforcement. His work was comprehensive and important although some ideas might be questionable, for example the measurement of stress distribution around the reinforcement and the method of calculating the strain distribution in the soil for reinforced soil during direct shear.

Shang (1989) conducted a series of large scale (1550×900×1220 mm) pullout tests at Carleton University. Coarse sand and crushed stone aggregate were used as backfill materials. Tensar UX 1600 geogrid was selected as the reinforcement. Piano wires and strain gauges were used to monitor the stretch (tensile strain) in the reinforcement during the pullout test. It was found that the strain gauges failed at a very low surcharge (5 kPa). The reasons could be either stripping of the gauges or excessive strain developed in the strain gauges. Piano wire was successfully used to monitor the displacement of the reinforcement and therefore to calculate average strains. The problems remaining in her work were that the theoretical analysis was inadequate and that the number of tests was limited to draw general conclusions.

Based on the experimental investigation a number of analytical investigations have been conducted to model the interaction between soil and reinforcement (Jewell 1984, 1990; Juran and Chen 1989; Palmeira 1989, 1990). Jewell (1984, 1990) developed some expressions to calculate the pullout force in the pullout test which takes the skin friction and bearing resistance on the transverse member of the reinforcement into account.

$$P_p = 2W_r L_r \sigma_n' f_b \tan \phi_m' \quad (2.1)$$

where P_p = pullout force, W_r = width of the reinforcement, L_r = length of the reinforcement, σ_n' = effective normal stress, f_b = bond coefficient, ϕ_m' = normalized internal friction angle.

The general expression for the bond coefficient can be written as:

$$f_b = \alpha_s \left(\frac{\tan \delta}{\tan \phi'_m} \right) + \left(\frac{\alpha_b B}{S} \right) \left(\frac{\sigma'_b}{\sigma'_n} \right) \frac{1}{2 \tan \phi'_m} \quad (2.2)$$

where α_s = fraction of the reinforcement plane area which is solid (i. e. plane surface area), α_b = fraction of each bearing surface area, B = height of bearing member, S = the distance between two bearing members. The expressions gave a good indication of the bond strength of reinforced soil. But there are two problems in this analysis. First, the model was based on the analogy of anchors being pulled out from the soil as shown in Fig. 2.5. This model may not be applicable to the geogrid reinforced soil system. Secondly, the determination of bearing stress, σ'_b , as he admitted, is difficult. He suggested using a punching failure mechanism of a strip foundation to estimate it.

Further theoretical analysis of pullout mechanism studied soil-reinforcement interaction. Several models were proposed to simulate the load transfer mechanism (Salomone et al. 1978; Costalonga 1988; Juran & Chen 1989). Salomone (1978) used an analogy between soil-pile system and soil-reinforcement system to study the load transfer mechanism in a reinforced soil (Fig. 2.6). An analytical model to simulate a soil-reinforcement system was proposed by Casalunga (1988) (Fig. 2.7).

$$\rho(x) \frac{\partial^2 u}{\partial t^2} - k \frac{\partial^2 u}{\partial x^2} = T_s(u, \frac{\partial u}{\partial x}, \frac{\partial u}{\partial t}, N, \delta) \quad (2.3)$$

where $\rho(x)$ = the average density of a mass at position x on reinforcement, u = the displacement of a point caused by the applied force, t = time, k = a spring force in a rheological model defined as a spring parameter times a displacement, $\partial u / \partial x$ = strain of

the reinforcement, $\partial u/\partial t$ = deformation rate, N = the normal load which confines the reinforcement, δ = the angle of friction or shearing resistance between the soil and reinforcement. The model gave some degree of agreement with the experimental data. But the model itself had several problems. First, there were too many parameters in the model that were unknown functions. Sufficient data were not available which could be used to evaluate all these parameters. Secondly, this model considered the skin friction only which is not sufficient for the geogrid reinforced soil system.

Costalonga (1988) also used the same technique as Salomone to study the interaction mechanism of soil-geogrid system. The model she used is shown in Fig. 2.7. The model needed the shear strength of the soil, the tensile and geometric properties of the reinforcement and the pullout test results as input data. The tensile force along the reinforcement could be predicted as shown in Fig. 2.8. The model was simple and useful. The only problem was it needed to monitor the strain or deformation in the reinforcement which is still difficult.

Juran and Chen (1989) conducted a fairly comprehensive study on the geotextile pullout mechanism. A load transfer model was proposed as shown in Fig. 2.9. In this model two interface soil models were utilized: an elastic - perfectly plastic and elasto-plastic strain hardening interface model. Fig. 2.10 shows a comparison of theoretical prediction and experimental data. Before reaching 16 mm of front end displacement, the prediction of the model matched the test data closely. After reaching 16 mm there was some discrepancy and a sudden drop in the model prediction.

Some disadvantages and limitations revealed from previous research can be

summarized as follows:

1. The use of a large apparatus minimizes or eliminates boundary effects and the need for scaling of the materials involved in the experiment. It is possible, therefore, to use representative specimens of reinforcement (Palmeira et al. 1988, 1989) and coarse grained backfill materials compacted to field specifications, i. e. maximum dry density and optimum moisture content. The large size of the test rig also allows the use of instruments buried in the soil, without significantly influencing the test results. The large test rig, therefore, appears to overcome most of the limitations associated with smaller boxes. Test preparation is necessarily time consuming and labour intensive in a large test setup.

2. While consistent soil preparation and compaction procedures are very important factors none of the methods used were ideal. Several compaction procedures have been used in the laboratory studies, such as electric jack hammers (Johnson 1985), standard Proctor hammers (Saxena and Budiman 1985), hand tamping devices (Elias 1979; Shang 1989) and mechanical tamping (Anderson and Nielsen 1984). Jewell (1980,1989) used a hopper device for placement of dry granular soil, but he could not achieve a high soil density.

3. Side friction effects can cause a reduction of normal load applied at the reinforcement level. Silicon, grease and rubber membranes were used to lubricate the side walls of a test tank to reduce wall friction (Jewell 1980).

4. Some passive earth pressure is developed in front of the transverse grid members as the reinforcement is pulled through the soil. Where the geogrid exits the

pullout tank soil particles may move into the slot and could get wedged. In order to prevent this, Mowafy (1986) used a sponge like material next to the slot. Christopher (1976) incorporated a sleeve protruding into the box to transfer the load point far behind the rigid front wall. Johnson (1985) removed the front wall in order to prevent interaction between the soil and reinforcement at the front panel.

5. It is difficult to monitor the deformation in the geogrid during pullout. The main objective in a pullout test is to monitor the displacement at the front end of the reinforcement, the pullout rate and the pullout force. It is also important to monitor the strain developed in the reinforcement. X-ray photographs were used to make visible the internal deformations (Jewell 1980; Dyer 1985). However, the maximum penetration depth was in the order of 15 cm. Piano wires were attached to nodes of the grid connected to LVDT's used by Shang (1989) to measure the displacement of the embedded geogrid. Juran and Chen (1989) also used the same technique in their tests, but the accuracy of the technique is still unknown.

6. The theoretical analysis is not adequate. In the theoretical analysis of pullout tests, skin friction and bearing resistance should be considered in a geogrid reinforced soil system. Although elastic perfectly plastic interface models reveal some interesting information, a more rational elasto-plastic strain hardening model is required. Furthermore the plastic deformation in the geogrid also should be considered in the modelling.

2.3. SHEAR STRENGTH OF REINFORCED SOIL

Direct shear tests are commonly used to determine the shear strength parameters

of granular soils. Due to their simplicity and consistency of results, direct shear tests remain popular in practice and are preferred over more sophisticated tests. The direct shear test, in a modified version, can also be used to derive parameters for reinforced composites. In a reinforced soil system additional shear strength is developed due to the presence of the reinforcing element. In a direct shear test, the horizontal displacement required to mobilized the shear stresses are measured together with the vertical displacement of the soil. Figures 2.11 through 2.14 show some schematic set-ups for direct shear tests.

In contrast to research on pullout tests on reinforced soil, very few investigations on direct shear tests are available, especially for large shear tests. The size of shear boxes used to determine shear strength of composites varied greatly. They ranged from a conventional (60×60×32 mm) shear box to 330×330×330 mm (Degautte and Mathie 1986; Richards and Scotts 1985; Williams and Houliham 1987). Again a small shear box cannot give reliable parameters which would be indicative of the shear behaviour of reinforced earth structures. Jewell (1980) proposed the following criterion for minimum box dimension, $L/D_{50} \geq 50$, where L is the length of the box and D_{50} is the average particle size of the soil. Jewell (1980) conducted a comprehensive study on reinforced Leighton Buzzard sand and carried out preliminary research on reinforced clay. The radiography technique was used to provide data on the deformation and strain within the reinforced soil. Based on his experimental study he proposed a theoretical model to express the increase in shearing resistance in the direct shear test due to the existence of reinforcement (Fig. 2.15)

$$\tau_{EXT} = \frac{P_R}{A_s} (\cos\theta \tan\phi + \sin\theta) \quad (2.4)$$

$$P_R = \sigma_\theta A_r \tan\delta \quad (2.5)$$

$$\sigma_\theta = \sigma_{yy} \frac{1 + \sin\phi_{ps} \sin(\phi_{ps} + 2\theta)}{\cos^2\phi_{ps}} \quad (2.6)$$

where τ_{EXT} = extra shearing resistance due to the reinforcement, θ = inclination angle of the reinforcement from the normal to the shear plane, P_R = reinforcement force, A_s = area of the shear plane, ϕ = direct shear angle of friction, σ_θ = normal stress on a plane at an angle θ , A_r = surface area of the reinforcement bonding with soil, δ = angle of surface friction between soil and reinforcement, σ_{yy} = applied normal stress and ϕ_{ps} = plane strain angle of friction.

Although predictions from his model match the general trend of experimental data (Fig. 2.16), there are two very important limitations in his work. The small size of the shear box (254x152x152 mm) limited the extrapolation of his data to field performance or for any design consideration. The test results can only be used as a quantitative reference. Another limitation lay in his theoretical model. He only considered the effect of the initial reinforcement orientation angle. He did not consider the change of this angle during the continuing shearing which is an important parameter in the modelling and will be shown in Chapter 3 of this thesis.

Gray et al. (1983,1986) have conducted a series of direct shear tests on reinforced soil. These tests were run on a dry sand with different types of fibres. Test results showed that (i) fibre reinforcement increased the peak shear strength and limited post peak

reduction in shear resistance. (ii) lengthening the fibre reinforcements increased the shear strength of the fibre-sand composite but only up to a point. A limiting or asymptotic shear strength was soon reached beyond which any further increase in fibre length had no effect. A simple continuous fibre reinforced soil model based on force equilibrium was developed (Fig. 2.17), which predicted the influence of various sand-fibre parameters through shear strength increase. The shear strength was (i) directly proportional to concentration or area ratio of fibres; (ii) greatest for initial fibre orientation of 60° with respect to the shear surface; and (iii) approximately the same for a reinforced sand tested in a loose and dense state, respectively.

The model gave an expression for shear strength increase for fibre in sand as

perpendicular fibres:
$$\Delta S_R = t_R [\sin\theta + \cos\theta \tan\phi] \quad (2.7)$$

Inclined fibres:
$$\Delta S_R = t_R [\sin(90-\psi) + \cos(90-\psi) \tan\phi] \quad (2.8)$$

in which

$$\psi = \tan^{-1} \left[\frac{1}{k + (\tan i)^{-1}} \right] \quad (2.9)$$

and S_R = shear strength increase from fibre reinforcement, t_R = mobilized tensile strength of fibres per unit area of soil, ϕ = angle of internal friction of sand, θ = angle of shear distortion, i = initial orientation angle of fibre with respect to the shear surface, k = shear distortion ratio ($k = x/z$, z = thickness of shear zone, and x = horizontal shear displacement). Although the model's predictions gave a reasonable agreement with the experimental data, two problems still cannot be solved by the model. First, the calculation needs the information of the width of shear zone. This has to be assumed.

Gray and Ohashi (1983) had to use their test data to back calculate a value for this thickness. However, the boundaries of the shear zone were less distinct than suggested by the model. Second, the model suggested that the increase in soil strength is directly proportional to the reinforcement area ratio for 0%-1.7%. Results from Jewell (1980) indicated that there is a limit to the strength gained by increasing the reinforcement area ratio. Apparently, at some point the presence of reinforcement produces a mode of deformation which is no longer adequately described by the model.

Juran and Chen (1988) proposed a load transfer model (Fig. 2.18) to simulate a direct shear test on reinforced soil samples. Three assumptions were made: (1) the reinforcement is assumed to be elastic perfectly plastic material (elastic modulus E and tensile strength R_t); (2) the soil is assumed to be an elasto - plastic, strain hardening material with non - associated flow rule characterized by five soil parameters, G/σ_y , ϕ_s , ϕ_{cv} , μ_1 and μ_2 , where G is the initial shear modulus, σ_y is the applied normal stress, ϕ_s is the peak friction angle, ϕ_{cv} is the critical state friction angle, μ_1 is a contractancy modulus and μ_2 is a dilatancy modulus; (3) a perfect adherence is assumed to prevail at the soil-reinforcement interface.

The model was used incrementally to calculate for each shear strain level γ_{xy} , the state of stress and strain in the reinforcement and in the sand.

Strain in the soil is

$$d\epsilon_p - \tan v \left[1 - \frac{\cos(\lambda_i - v) \sin \lambda_i}{\sin v} \right] d\gamma_{xy} \quad (2.10)$$

where v is the dilatancy angle calculated from

$$\tan v = \frac{d\epsilon_v}{d\gamma_{xy}} - \frac{1}{\mu_s} \left[\tan \phi_{cv} - \frac{\tau_{xy}}{\sigma_y} \right] \quad (2.11)$$

where $\mu_s = \mu_1$ or μ_2 .

Strain increment in inclusion ($d\epsilon_R$) is equal to the extension (or compression) strain increment in the soil in its direction ($d\epsilon_R = d\epsilon_p$).

Tensile stress increment in the inclusion is given by

$$d\sigma_R = E d\epsilon_p \quad \text{with } \sigma_R < R_T \quad (2.12)$$

Equivalent state of stress in soil is given by

$$\tau_{xy} = \tau_{xy}^{app} - \Delta\tau_{xy} \quad \text{and} \quad \sigma_y = \sigma_y^{app} + \Delta\sigma_y \quad (2.13)$$

with $\tau_{xy} = \sigma_y g(\gamma_{xy})$

$\Delta\tau = \sigma_R (e/S_v) \sin \lambda_i \cos \lambda_i$ is the equivalent decrease of shear stress in the soil, $\Delta\tau = \sigma_R (e/S_v) \cos^2 \lambda_i$ is the equivalent increase of the normal stress in the soil, $g(\gamma_{xy})$ is the assumed strain hardening function, τ_{xy}^{app} and σ_y^{app} are, respectively, the shear and normal stress applied on the reinforced soil sample and e and S_v are, respectively, the thickness of the reinforcement and their spacings.

The model's predictions compared very well with Jewell's test results but because Jewell used a very small box (254×152×152 mm), without comparison with the results from an adequate sized shear box, the applicability of this model is still in question. Furthermore, the model assumed that a perfect adherence exists at the soil-reinforcement

interface. According to Shewbridge and Sitar's (1989) research, the reinforcement and soil were displaced relative to one another along the length of reinforcement. Otherwise, the computed elongation strain in the reinforcement would far exceed the rupture strains of the reinforcement. Thus, the reinforcements would have failed in tension at the centre of the shear zone, which was not observed in any of the tests. Therefore, perfect adherence between soil and reinforcement cannot be assumed.

Bauer et al. (1985; 1987; 1988(a); 1988(b); 1990(a); 1990(b); 1993a; 1993b;) have also conducted a systematic study on reinforced soil. In addition to a series of large pullout tests (Bauer and Mowafy 1985; Mowafy 1986; Shang 1989), a series of direct shear tests were carried out in order to study the shearing behaviour of reinforced composites. Based on considerable experimental evidence, a load transfer mechanism model (Fig. 2.19) was proposed using the force-equilibrium method (Bauer 1989, 1990). This model is applicable mainly to results from direct shear tests. The model is simple and the predictions agreed well with test results. Again the tests were carried out in a small size box (60×60×32 mm). Boundary effects should be the main concern for the validity of the parameters in the prediction of the behaviour of prototype structures.

In all the above numerical models the assumption of the width of the shear zone affects both the shear resistance and the tension mobilized in the reinforcement. Thus the width of the shear zone is of major importance in the prediction of additional strength of the reinforced soil using these models.

Shewbridge and Sitar (1989,1990) used a fairly large direct shear box ((761×402×400 mm) (Fig. 2.14) to observe the deformation pattern in a shear test. The

test results showed that the deformation of a reinforced soil sample is not confined to a discrete region undergoing linear simple shear as suggested in Gray and Ohashi's model (1983, Fig 2.17), but instead the deformation pattern is curvilinear and symmetric about the centre of the shear zone. Shewbridge and Sitar (1989) showed that the deformed shape of the reinforcing fibres can be described by an exponential decay function:

$$X_1 = B - Be^{-b|X_2|} \quad (2.14)$$

where X_1 is the axis parallel to the direction of shear, X_2 is the axis perpendicular to the direction of shear with the origin at the centre of the shear zone, B is equal to one - half of the externally applied shear displacement, and b is a deformation decay constant. Figure 2.20 is a schematic diagram showing the geometry of the deformed reinforcement described by equation 2.14.

Conclusions were made based on experimental evidence and theoretical analyses and they stated that the width and shape of the shear zone in a reinforced soil composite is altered by the reinforcement concentration, stiffness and reinforcement - soil bond. Shear zones tend to be wider in reinforcement - soil composites than in a natural soil, and the width of the shear zone increases with increasing stiffness of the composite due to increased reinforcement concentration, stiffness or soil bond strength. Reinforcement concentration and type affect the measured strength of the reinforced soil composite, but a linear relationship between reinforcement concentration and increased strength was not found. The model suggested by Shewbridge and Sitar is a useful approach to assess the deformation pattern in direct shear. But the decay constant, which determines the thickness of the shear zone and gives the indication of the distribution and magnitude of

shear strain within the zone, is dependent on the concentration of the reinforcement which in turn depends on the size of the shear box or sheared soil volume. According to Gray and Oshahi's measurement the width of shear zone was 85 mm in their small box. In order to check the validity of the model the shear zone width was measured in this investigation and it was found out that the width of shear zone was about 140mm depending on the orientation of the reinforcement. Thus, the effect of amount and orientation of reinforcement with regard to soil volume must be taken into account in order to give a better interpretation of test results.

The limitation of the previous research regarding direct shear tests can be summarized as follows.

1. The use of a small test apparatus, which cannot accommodate a large quantity of soil and a representative reinforcement specimen is suspect. The sieved down soil in such an apparatus is generally placed in a dry condition. Because of scale the influence of boundary conditions, such as the friction on the walls and the depth to length of reinforcement ratio, as well as the ratio of particle size to minimum reinforcement dimension are not evaluated. It has been shown (Palmeira 1987) that all these effects do have some influence on the test results.

2. Although tests have been carried out on various geogrid reinforced soils under shear, the most common geogrids used were Tensar products. More direct shear test data will be needed for soil reinforced with flexible geogrids, such as Conwed Stratagrids, Mirafi grids and other similar products.

3. The displacement or strain mobilized in the soil and reinforcement were not

monitored in previous investigations. An X-ray technique was used to measure movement of embedded lead shot in order to obtain information of soil deformation and strain but the penetration depth is very limited (150 mm) (Jewell 1980; Dyer 1983).

4. The theoretical analysis of interaction mechanism between soil and reinforcement is still not adequate. Improved theoretical modelling of the composite in direct shear is still needed in order to incorporate the characteristics of the geogrid. The existing models do not consider the mechanical and geometric properties of geogrids (Gray and Ohashi 1983; Jewell and Wroth 1987; Juran et al. 1981, 1982; Bauer 1988, 1990, 1993 and Shewbridges 1989). Therefore, further research is essential.

Fig. 2.1. Pullout Testing Device (after Rowe 1985)

Fig. 2.2. Pullout Testing Decice (after Holtz 1977)

Fig. 2.3. Pullout Testing Device (after Christopher 1976)

Fig. 2.4. Pullout Testing Decice (after Juran 1989)

Fig. 2.5. Model of Bearing Resistance in Grid Reinforcement (after Jewell 1974)

Fig. 2.6. Analogy Between Pile-Soil and Soil-Reinforcement System (after Salomone 1978)

Fig. 2.7. Model of Interaction between Soil and Geogrid (after Costalonga 1988)

Fig. 2.8. Progressive Displacement along the Geogrid (after Costalonga 1988)

Fig. 2.9. Modelling Load Transfer Between Soil and Extensible Inclusion
(after Juran 1989)

Fig. 2.10. Numerical Simulation of Pullout Test on Woven Polyester Strips
(after Juran 1989)

Fig. 2.11. Direct Shear Apparatus (after Rowe 1985)

2.12. Direct Shear Apparatus (after Palmeira and Milligan 1989)

Fig. 2.13. Direct Shear Apparatus (after Siel 1987)

Fig. 2.14. Direct Shear Apparatus (after Shewbridge 1989)

Fig. 2.15. Model of Reinforced Sand in Direct Shear Test (after Jewell 1987)

Fig. 2.16. Comparison of Test Results with Model's Prediction (after Jewell 1987)

Fig. 2.17. Model of Soil Reinforced with Flexible, Elastic Reinforcement in Direct Shear Test (after Gray 1983)

Fig. 2.18. Model of Reinforced Soil in Direct Shear (after Juran 1989)

S : APPLIED SHEAR FORCE

N : APPLIED NORMAL FORCE

F : SHEAR RESISTANCE OF SOIL

T_R : MOBILIZED TENSION (COMPRESSION) IN REINFORCEMENT

NOTE: $T_R \sin \theta$ IS POSITIVE WHEN ACTING IN SAME SENSE AS F

Fig. 2.19. Model of Reinforced Soil in Direct Shear (after Bauer 1990)

Fig. 2.20. Schematic Geometry of Deformed Reinforcement in Coordinate Space (after Shewbridge 1989)

CHAPTER 3

THEORETICAL CONSIDERATION OF REINFORCED SOIL

3.1 INTRODUCTION

There are four potential failure mechanisms in a reinforced earth structure as shown in Fig. 1.2. (1) Tensile failure of the reinforcement (which is beyond the scope of this investigation), 2) sliding along the soil - reinforcement interface, 3) shearing of the soil - reinforcement composite material along a failure surface, and 4) pullout of reinforcement behind the failure surface if the anchorage length is insufficient or the anchorage capacity is exceeded. In order to have a better understanding of the behaviour of a reinforced earth structure it is justified to choose direct shear tests (special case is the sliding test) and pullout tests to simulate the load transfer mechanisms in the laboratory.

The direct shear and pullout apparatus test soil specimens under conditions of plane strain. Thus, it is reasonable to use this test to study the behaviour of long geotechnical structures (such as retaining walls and embankments) reinforced with geogrid under the condition of plane strain. It will be helpful to examine some of the basic definitions and conditions in plane strain before discussing in detail direct shear and pullout tests.

3.2. PLANE STRAIN CONDITIONS

Plane strain conditions are prevalent in any long geotechnical structure - embankments, retaining walls or strip footings, in which the length is at least five times

its cross sectional width.

For such structures, the displacement and strain along the long axis are zero

(Fig. 3.1):

$$\epsilon_{yy} = 0 \quad (3.1)$$

where ϵ_{yy} is the normal strain along the length of the structure (y-direction).

Under such conditions of plane strain, the increment of work per unit volume when an increment of strain occurs under certain imposed stresses is

$$\delta w = \sigma_{xx}' \delta \epsilon_{xx} + \sigma_{yy}' \delta \epsilon_{yy} + \sigma_{zz}' \delta \epsilon_{zz} + \tau_{yz} \delta \gamma_{yz} + \tau_{xz} \delta \gamma_{xz} + \tau_{xy} \delta \gamma_{xy} \quad (3.2)$$

$$\delta w = \sigma_{xx}' \delta \epsilon_{xx} + \sigma_{zz}' \delta \epsilon_{zz} + \tau_{xz} \delta \gamma_{xz} \quad (3.3)$$

since $\delta \epsilon_{yy} = 0$, from symmetry

$$\tau_{xy} = \tau_{yz} = 0$$

and $\delta \gamma_{xy} = \delta \gamma_{yz} = 0$

where δw is the increment of work;

σ_{xx}' , σ_{yy}' , σ_{zz}' are effective normal stresses;

τ_{yz} , τ_{xz} , τ_{xy} are shear stresses;

$\delta \epsilon_{xx}$, $\delta \epsilon_{yy}$, $\delta \epsilon_{zz}$ are normal strain increment;

$\delta \gamma_{yz}$, $\delta \gamma_{xz}$, $\delta \gamma_{xy}$ are shear strain increment.

If we assume that the axes of strain increment coincides with the principal axes of stress, the expression for the work increment can be written in terms of principal effective stress σ_1' , σ_3' and the corresponding principal strain increments $\delta \epsilon_1$, and $\delta \epsilon_3$

$$\delta w = \sigma_1' \delta \epsilon_1 + \sigma_3' \delta \epsilon_3 \quad (3.4)$$

where σ_1' , σ_3' are effective principal stresses;

$\delta\epsilon_1, \delta\epsilon_3$ are principal normal strains;

It is convenient to define some parameters to separate the strain and stress into volumetric and shear parts. Three pairs of parameters may be defined (Fig. 3.1)

(1) The volumetric strain increment

$$\delta\epsilon_v = \delta\epsilon_1 + \delta\epsilon_3 \quad (3.5)$$

where $\delta\epsilon_v$ is the volumetric strain.

The corresponding stress is the mean effective stress in the plane of shearing

$$s' = (\sigma_1' + \sigma_3')/2 \quad (3.6)$$

where s' is the mean effective stress.

(2) The shear strain increment (diameter of the Mohr's circle of strain increment)

$$\delta\gamma = \delta\epsilon_1 - \delta\epsilon_3 \quad (3.7)$$

where $\delta\gamma$ is the shear strain increment.

The corresponding maximum shear stress in the plane of shearing

$$t = (\sigma_1' - \sigma_3')/2 \quad (3.8)$$

where t is the maximum shear stress.

(3) The angle of dilation

$$\sin\psi = -(\delta\epsilon_v)/\delta\gamma = -(\delta\epsilon_1 + \delta\epsilon_3)/(\delta\epsilon_1 - \delta\epsilon_3) \quad (3.9)$$

where ψ the angle of dilation.

The mobilized angle of friction

$$\sin\phi = t/s = (\sigma_1' - \sigma_3')/(\sigma_1' + \sigma_3') \quad (3.10)$$

where ϕ is the angle of friction.

3.3. DEFINITION OF PARAMETERS

In order to interpret the experimental data from direct shear and pullout test, some basic parameters have to be defined.

3.3.1. Stresses

Fig. 3.2 shows a typical direct shear box in the laboratory. In a typical direct shear test, a vertical surcharge load P is applied to the centre of the upper boundary, a horizontal shear force S is exerted to the top and bottom of the shear box. Consequently, a predetermined failure mechanism is induced along a narrow horizontal region in the mid-height of a specimen, i. e. shear plane. In the shear plane the horizontal shear stress acting on a soil element increases from its initial K_0 state to failure under the constant normal stress. Failure is usually assumed to occur when a drastic decrease of shear stress in the shear stress versus shear displacement curve in unreinforced soil is observed.

In order to analyze the direct shear test, it is usually assumed that the sample experiences uniform stress and strain in the shear plane. The internal measured strain field by Jewell (1980) confirmed this assumption. The standard approach to interpret the test results is to draw a Mohr circle of stress at failure. The mobilized friction angle, ϕ_d , is the angle between the tangent line of the Mohr circle through the origin and the horizontal line, which is referred to as direct shear friction angle (Eqn. 3.13). The stress path of a soil element on a horizontal shear plane is represented by a vector AB , as shown in Fig. 3.3.

The stresses in direct shear test can be defined as:

$$\text{shear stress: } \tau_{yx} = S/A_s \quad (3.11)$$

$$\text{Normal stress: } \sigma_{yy} = P/A_s \quad (3.12)$$

where A_s = the area of shear plane.

Therefore:

$$\phi_{ds} = \tan^{-1} \left(\frac{\tau_{yx}}{\sigma_{yy}} \right) \quad (3.13)$$

This approach is known as the maximum obliquity criterion. It has been shown that the horizontal shear plane is the middle of shear box as being a region of zero extension (Jewell 1980; Dyer 1985). If this is true, the Mohr circle of strain increments at failure is similar to the one presented in Fig. 3.3 (b).

3.3.2. Strain increments

The shear zone, which undergoes uniform deformation is usually not known. The relative magnitude of the strain increment $\delta\epsilon_{xx}$, and $\delta\gamma_{xy}$ can be calculated with embedded markers in the soil assuming the orientation of principal axes of strain coincides with the direction of the principal axes of stress, since it is known that the horizontal plane is a region of zero extension. Therefore, $\delta\epsilon_{yy} = 0$, i. e. zero linear strain increment.

Jewell (1980) pointed out that the presence of reinforcement in the soil will cause a non-uniform state of stress and strain increment within the soil sample. The overall stress state τ_{yx} , and σ_{yy} , measured from boundary conditions, may not be representative of the conditions within the sample. But due to the simplicity of this measurement and

data analysis the test results in this thesis for both natural and reinforced soil samples are presented in the same manner, although in a number of special tests the internal strain in the soil and reinforcement were measured.

3.4. INTERPRETATION OF DIRECT SHEAR TESTS

3.4.1. Development of Strains

It is important to distinguish between elastic and plastic strains in order to describe the behaviour of granular soil under shear.

$$\delta\epsilon_{ij} = (\delta\epsilon^{ij})_e + (\delta\epsilon^{ij})_p \quad (3.14)$$

where $\delta\epsilon_{ij}$ is the strain increment developed in the soil

$(\delta\epsilon^{ij})_e$ is the elastic strain increment, and

$(\delta\epsilon^{ij})_p$ is the plastic strain increment.

Elastic strain is caused by a change of stress, and has a magnitude determined by the stress change and two elastic constants, the shear modulus G and Poisson's ratio ν . Plastic strain depends on the current stress ratio, and is caused by particle reorientation. Plastic distortion is described by an angle of dilation ψ . This angle defines the incremental rate of change in plastic volume strain $d\nu^p$ with respect to the plastic shear strain $d\gamma^p$:

$$\sin \psi = - d\nu^p/d\gamma^p \quad (3.15)$$

The relation between mobilised shearing resistance and dilation angle is well established and it is described by a flow rule as discussed in the next section. Stroud (1971) conducted a series of simple shear tests to study the strain or deformation behaviour of

sand under shear. He used elastic parameters determined from triaxial tests and concluded that the elastic strain is negligibly small compared to the plastic strain under constant vertical stress. This may also be true for the direct shear test (Jewell, 1980). Therefore, it can be said that the soil dilatancy behaviour in direct shear is an indication of the plastic behaviour of the soil.

3.4.2. Soil Dilatancy

It is a well established fact that loose sands contract and dense sands expand when they are sheared. The significance of the change in volume associated with distortion of granular materials was discussed by Reynolds (1985, 1986). He coined the term "dilatancy", but he noted that "dilatancy has long been known to those who buy and sell corn". Reynolds performed many experiments with granular materials and was able to describe the phenomena that occurred due to the dilatancy of materials, but he did not make any measurements of the strength of material with various densities of packing. Casagrande (1936), however, did describe the stress and strain curves which were expected when dense and loose sand were sheared in direct shear (Fig. 3.4). During the shearing test on dense sand, "the shearing stress reached a maximum S_D (point B on the curve) and if the deformation is continued, the shearing stress drops again to a small value, at which value it remains constant for all further displacement. During this drop in shearing stress, the sand continues to expand (curve EG), finally reaching a critical void ratio at which continuous deformation is possible at constant shearing stress S_L ". When a loose sample of sand is subjected to a shearing test under constant normal pressure, however, "the shearing stress simply increases until it reaches the shearing

strength S_L , and if the displacement is continued beyond this point the resistance remains unchanged. Obviously, the volume of the sand in this state must correspond to the critical void ratio which we had finally reached when performing a test on the same material in the dense state". Therefore, he concluded that "every cohesionless soil has a certain critical void ratio, in which it can undergo any amount of deformation or actual flow without volume change".

3.4.3. Flow Rule of Coarse Granular Soils

The relationship between plastic strain increment ratio and stress ratio is known as a flow rule governing the mode or mechanism of plastic deformation or flow of soil. Granular soil flows plastically in a well defined fashion. The plastic strain increment ratio depends directly on the current stress ratio in the soil, and is largely independent of stress level or density (Jewell 1980; Stroud 1971). The dilatancy behaviour of a granular soil in a direct shear box can be simulated by the interlocking saw blades shown in Fig. 3.5. Because the soil is expanding (dilating) as it is sheared, it is supposed that sliding within the soil takes place, not on horizontal planes, but on planes inclined at an angle of dilation ψ on these planes. If the angle of friction on the inclined planes is ϕ_{cs}' , then the overall mobilized angle of friction on horizontal plane is ϕ_{α}' ,

$$\phi_{\alpha}' = \phi_{cs}' + \psi \quad (3.16)$$

where ψ is angle of dilation;

ϕ_{α}' is the overall mobilized friction angle on a horizontal plane; and

ϕ_{cs}' is the friction angle on the plane of ψ with the horizontal plane, which is a soil constant.

Figure 3.5 also shows an imaginary saw-tooth sliding surface in a direct shear test on a soil sample. The four quantities that can be measured from boundary conditions of the test apparatus are: normal surcharge load P , shear force S , vertical displacement y and horizontal displacement x . The work done by the applied loads P and S on the soil sample during increment displacement δy and δx is

$$\delta w = P\delta y + S\delta x \quad (3.17)$$

where δw is the work done by the forces on the soil sample, $S\delta x$ represents the work done in shearing the sample and $P\delta y$ is the work done due to the soil volume change as it is sheared (Wood 1990).

Following Taylor (1948), assuming that all of the net work input into the soil is dissipated in friction (elastic strain or displacement is negligible) and that this friction dissipation is controlled by the normal load P and a friction constant μ . Therefore a simple stress-dilating relationship can be obtained

$$\delta w = \mu P\delta x \quad (3.18)$$

substituting Eq. 3.18 into Eq. 3.17 and rearranging gives

$$S/P + \delta y/\delta x = \mu \quad (3.19)$$

S/P is the externally overall mobilized friction on horizontal plane

$$S/P = \tan \phi_{\alpha}' \quad (3.20)$$

The second term describes the dilatancy of the soil sample

$$\delta y/\delta x = - \tan \psi \quad (3.21)$$

The relationship of Eqn. (3.19) can be tested on the data from a direct shear test on coarse sand and crushed limestone aggregate used in this investigation as plotted in

Figs. 5.2, 5.4, 5.6, and 5.7. The slope of the graphs (Figs. 5.6 and 5.7) of horizontal displacement (x) versus vertical displacement (y) is used to generate values of $\delta y/\delta x$, while S/P values can be obtained from Figs. 5.2 and 5.4. The sum of $S/p + \delta y/\delta x$ is plotted against horizontal displacement as shown in Figs. 3.6 and 3.7. It can be observed that except in the early stage of the test, a fairly straight line and constant μ values were obtained for both coarse sand and crushed limestone aggregate. This means that the plastic deformation behaviour of granular soil used in this study can be expressed by equation 3.20, i. e. the flow rule.

3.4.4. Failure Criterion of Coarse Granular Soils

Although thermodynamics offer precise definitions for the stability of a system, it has been difficult to apply it to a soil system, especially to a soil sample in a direct shear test, because how should one express the thermodynamic quantities in terms of soil properties under shear is a problem. "A stable system may be defined as one in which work is being transferred from an external agency into the system; a unstable system, in contrast, is one in which the transfer of energy is in other direction" (Jewell 1980). Therefore, in a direct shear test, the point of failure may be defined as the maximum shearing stress sustained by the sample, which means that under this constant shear stress the soil sample experienced a run-away strain or deformation. This is shown by a drastic decrease in shear force and by a nearly horizontal leveling off of shear force in a shear displacement versus shear force diagram.

3.5. LOAD TRANSFER MODEL FOR REINFORCED SOIL IN DIRECT SHEAR

3.5.1. Introduction

Traditionally reinforced earth structures were reinforced with steel strips. The use of polymer reinforcement in reinforced earth systems has demonstrated economic and technical advantages over the traditional technique. The characteristics of polymer material, such as adequate tensile strength, compatible bending stiffness and low modulus, make its function quite different from steel strip reinforcement. Since the geosynthetic material does not possess much flexural stiffness any increase in shear resistance is attributed to the additional tensile strain mobilized in the reinforcement due to the mobilization of tensile strain in the soil. Therefore, the efficiency of the reinforcement or the increase of shear resistance of the soil due to reinforcement is highly dependent on the direction of tensile strain in the soil. It is known that the shear plane is the plane of zero extension (Jewell 1980; Palmeira 1989), thereby this efficiency is dependent on the orientation of the geosynthetic with respect to the shear plane. The maximum reinforcing efficiency will be reached when the orientation of the reinforcement coincides with the inclination of the minor principal strain(tensile strain) of the soil.

The use of geogrid reinforcement can increase the bond between soil and reinforcement due to the bearing mechanism of the transverse member of the geogrid and the interlock of soil particles within the geogrid apertures. Thus, it will enhance to mobilize the tensile strain in the geogrid. However, the interaction mechanism between the geogrid and soil and the mobilization of tensile strain in geogrid reinforced soil are not yet well understood and are difficult to measure. This phenomenon needs further exploration.

The direct shear test is a suitable mean to study the interaction between soil and reinforcement because it simulates the shear mechanism along a potential failure plane (Fig. 3.8). The present study is aimed to investigate the load transfer mechanism between the geogrid and soil by direct shear and to model this mechanism by connecting the soil deformation (soil dilatancy) with the shear strength increase in order to achieve a better understanding of the fundamental behaviour between geogrid and soil.

3.5.2. Static Model for Reinforced Soil in Direct Shear

Modelling load transfer between soil and reinforcement by the state of stress and strain during the shear deformation is extremely complex. It seems impossible to account for the various interactions between soil particles and a mesh like geogrid (Bauer and Mowafy 1988). Sophisticated three dimensional finite element methods could be one of the solutions but the mathematical formulation and calculation algorithm would be very complicated. A more practical approach would be to use a simple limit equilibrium model verified by experimental results, especially by data from large scale tests, which can represent the field behaviour of reinforced soil. Four models have been documented in the literature (Bauer et al. 1992; Jewell and Wroth 1987; Palmeira 1986 and Gray and Ohashi 1981). All these models are similar to each other and they are all based on limiting equilibrium. The model presented in this paper is a variation of Palmeira's and Bauer's models.

1. Model Assumptions

The assumption for this model are: 1) there is a definite shear zone developed in a direct shear test. The thickness of the zone is the order of 20 to 25 percent of the height

of the test sample (Bauer 1992), 2) the soil adjacent to the horizontal shear zone behaves like rigid blocks in which the reinforcement is firmly embedded, 3) the reinforcement is sufficiently long and is prevented from being pulled out, 4) the reinforcement is flexible (no bending stiffness) and any increase of shear strength of the composite is due to the mobilized tensile strain (or extension) of the reinforcement, 5) the dilatancy (or contraction) of the soil and the interaction between soil and reinforcement only happen in the shear zone, and 6) perfect bonding exists between soil and reinforcement outside of the shear zone.

2. Formulation

a) Force Equilibrium

A simple static model of a reinforced composite in direct shear is shown in Fig. 3.9 (a). The reinforcement is inclined at an initial angle of θ to the normal of the shear plane. This inclination angle will increase with the shear displacement Δx and the vertical displacement Δy (Fig. 3.9 (b)). This increase is denoted by $\Delta\theta$. The tensile force in the reinforcement, acting across the central plane, may be resolved into normal and tangential components.

We may define:

$$\sigma_{yy} = \frac{P+W}{A_s}, \quad \tau_{xy} = \frac{S}{A_s} \quad (3.22)$$

$$\sigma = \frac{N}{A_s}, \quad \tau = \frac{F}{A_s} \quad (3.23)$$

where P = applied surcharge,

W = weight of soil above shear plane,

S = applied shear force,

A_s = cross sectional area of shear box,

N = mobilized normal force on shear plane, and

F = mobilized shear force on shear plane.

The force equilibrium can be written as

$$N = P + W + P_r \cos (\theta + \Delta\theta) \text{ and} \quad (3.24)$$

$$F = S - P_r \sin (\theta + \Delta\theta) \quad (3.25)$$

where

P_r = tensile force developed in the reinforcement.

From Equations (3.22), (3.23), (3.24) and (3.25) the shear stress and normal stress on the shear plane can be expressed as

$$\tau = \tau_{xy} - \sigma_r \sin (\theta + \Delta\theta) \quad (3.26)$$

$$\sigma = \sigma_{yy} + \sigma_r \cos (\theta + \Delta\theta) \quad (3.27)$$

where

$$\sigma_r = P_r/A_s$$

= tensile stress mobilized in the reinforcement.

According to Mohr- Coulomb's theory

$$\tau = \sigma \tan \phi \quad (3.28)$$

where

ϕ = the angle of internal friction of the soil

Substituting equations (3.26) and (3.27) into (3.28) we obtain:

$$\begin{aligned}\tau &= \sigma_r[\sin(\theta + \Delta\theta) + \cos(\theta + \Delta\theta)\tan\phi] + \sigma_{yy} \tan\phi \\ &= C_r + \sigma_{yy} \tan\phi\end{aligned}\quad (3.29)$$

where

$$C_r = \sigma_r[\sin(\theta + \Delta\theta) + \cos(\theta + \Delta\theta)\tan\phi] \quad (3.30)$$

= "apparent cohesion" (Schlosser and Vidal 1969) of the reinforced soil.

b) Tensile Strain in the Reinforcement

An idealized deformation mode of a reinforced soil specimen is shown in Fig. 3.9(b). The central zone of the specimen is the shear zone, whereas the upper solid block undergoes both horizontal and vertical displacements denoted, respectively, by Δx and Δy . Considering the segment OA of the reinforcement, the rotation of OA during shear can be expressed as (Juran et al. 1988)

$$\tan(\theta + \Delta\theta) = \frac{\tan\theta + \frac{\Delta x}{L\cos\theta}}{1 + \frac{\Delta y}{L\cos\theta}} = \frac{L\sin\theta + \Delta x}{L\cos\theta + \Delta y} \quad (3.31)$$

where L is the initial length of segment OA. The reinforcement is elongated by the amount of ΔL . From the geometry of Fig.3.9 (b) the elongation of the reinforcement is

$$\Delta L = [(L \sin\theta + \Delta x)^2 + (L \cos\theta + \Delta y)^2]^{1/2} - L \quad (3.32)$$

The average tensile strain in the reinforcement, therefore, is

$$\epsilon_r = \frac{\Delta L}{L} = \left[\left(\sin\theta + \frac{\Delta x}{L} \right)^2 + \left(\cos\theta + \frac{\Delta y}{L} \right)^2 \right]^{1/2} - 1 \quad (3.33)$$

and

$$\tan\psi = \Delta y / \Delta x \quad (3.34)$$

where

ψ = the soil dilatancy angle

c) Constitutive Relationship of Reinforcement

(1) Linear Elasticity

A tensile force versus tensile strain curve for the polyester geogrid is shown in Fig. 3.10. (Razaqpur 1990). The constitutive relationship can be approximated by

$$\sigma_r = E_r \epsilon_r \quad (3.35)$$

where

E_r = the modulus of elasticity of the geogrid.

(2) Non-linear Elastic-Plastic Model

A more realistic model to account for the non-linearity of the stress-strain relationship of the geogrid can be obtained by a hyperbolic elastic plastic strain hardening model as follows

$$\sigma_r = \frac{a \epsilon_r}{b + \epsilon_r} \quad (3.36)$$

where $a=2.22$, $b=1.66$

are constants and were obtained from non-linear curve fitting.

3. Model Prediction

The proposed model can be used to predict the shear strength increase, to study the effect of original geogrid orientation on the shear strength increase and to evaluate the effect of soil dilatancy on the shear strength increase.

(1) Shear Strength Increase of Reinforced Soil

As stated above the presence of a geogrid in the soil can contribute to the overall

shear strength increase due to the "apparent cohesion", C_r , as described by Equation (3.30) and C_r is a function of the original geogrid orientation and the increase of the orientation angle. The orientation angle increase in turn is a function of soil dilatancy (Eqns. 3.30, and 3.31). Fig. 3.11 shows an example of the model prediction based on the tests result of unreinforced coarse sand used in this investigation. The results show clearly that a shear strength increase can be obtained for soil reinforced with geogrids orientated at 0, 30 and 45°.

(2) Effect of Geogrid Orientation

Fig. 3.12 shows a force equilibrium diagram for a reinforcement in a direct shear test. The equilibrium condition can be expressed as

$$P_r \cos \Delta\theta = 2w \int_0^l \sigma_\theta \cos \theta \tan \delta dz \quad (3.37)$$

where $\tan \delta$ = friction component between geogrid and soil, which can be obtained from a sliding test

σ_θ = interaction force between geogrid and soil

z = depth of soil (from top down)

w = width of the geogrid.

Assuming that σ_θ follows the following distribution and K_0 conditions prevail

$$\sigma_\theta = (\gamma z + q_0) K_0 \quad (3.38)$$

where γ = the unit weight of soil

q_0 = normal surcharge load

K_0 = coefficient of earth pressure at rest.

Substituting (3.38) into (3.37) and integrating

$$P_r \cos \Delta\theta = 2wK_0(\gamma^2/2 + q_0 l) \cos \theta \tan \delta \quad (3.39)$$

Noticed that $\sigma_r = P_r/A_r$ and substituting equation (3.39) into (3.30), the following expression can be obtained

$$C_r = K \frac{\cos \theta \tan \delta}{\cos \Delta\theta} [\sin(\theta + \Delta\theta) + \cos(\theta + \Delta\theta) \tan \phi] \quad (3.40)$$

where $K = 2wK_0(\gamma^2/2 + q_0 l)/A_r$ is a constant for a given soil and test conditions.

Fig. 3.13 shows a relationship of C_r versus θ at different $\Delta\theta$ based on Equation (3.40). It is shown clearly that the shear strength increase of a reinforced soil in shear is greatly affected by the geogrid orientation. Maximum shear strength increases were obtained when the geogrid was orientated between 30 to 40 degree with respect to the normal to the shear plane depending on the soil friction angle. The remaining test results and further comparisons to the model prediction, will be presented in Chapters 6 and 8, respectively.

(3) Soil Dilatancy Effect

As stated before, dense soil will expand in volume when sheared. This volume change is defined by an angle of dilation (Section 3.4). Because of soil dilatancy (or volume expansion) tensile forces are mobilized in the reinforcement through the interaction between soil and reinforcement. The effect of soil dilatancy on the shear strength increase in direct shear can be examined by combining Eqns. (3.30), (3.31) and (3.33). Figure 3.14 presents the relationship between the soil dilatancy angle and shear

strength increase. The figure shows that an approximate linear increase of shear strength can be expected with the mobilization of the soil dilatancy angle to 30° . Beyond this value the shear stress increase become accentuated.

In summary, the proposed static model considers the essential features of reinforced soil in direct shear and the model will be used to predict the experimental results. A detailed comparison and analysis of the results will be presented in Chapter 8 of this thesis.

3.6. LOAD TRANSFER MODEL IN PULLOUT

3.6.1. Introduction

Among the failure mechanisms for a reinforced soil structure (e. g. embankment or retaining wall), pullout and tensile failures are the most common types. In order to accomplish a safe design the reinforcement must be safe against these failures. Thus assumptions for both the magnitude and distribution of tensile force and shear stress along the interface are essential. Inextensible reinforcements (rigid metallic strips and steel bars) move as a rigid body during the pullout process. The tensile force in the reinforcement is assumed distributed linearly or uniformly once the reinforcing element starts to move. The mobilized interface shear stress also has a linear or constant distribution (Schlosser and Vidal 1969; Lee et al 1973). For extensible reinforcement, the extensibility significantly affects the load transfer mechanism, and a nonuniform distribution of tensile force is mobilized in the reinforcement, which results in a decreasing shear displacement along its length. The interface shear stress is, therefore, not uniformly mobilized over the

length of the reinforcement (McGown 1978). Since the extensibility depends upon the mechanical and geometrical properties of the reinforcement scale effects in pullout test must also be considered.

The review of literature revealed that the previous models of load transfer mechanism in pullout used simplifying assumptions (Chapter 2). In these models either a constant shear stress distribution along the interface is assumed or only friction forces between reinforcement and soil is considered. The latter case is only suitable to sheet reinforcement (Juran and Chen 1988). In order to achieve a better understanding of the failure mechanism and interpretation of pullout test data for geogrid reinforced soil, a new load transfer model should be developed in order to consider the special features of geogrids such as bearing loads carried by the transverse members and the interlocking capacity of these grids.

3.6.2. Load Transfer Model

The analytical model presented here is based on the measurement of nodal displacement of the geogrid in pullout; knowing the properties of the soil; and tensile and geometric properties of the geogrid. The model can be used to predict the tensile force in the reinforcement and shear stress mobilized along the geogrid and soil interface.

1. Assumptions

A geogrid is discretized into a series of elements connected by springs (Fig. 3.15). The shear stress is assumed to be constant along each element. The tension within each element decreases from one end to the other. Consequently, the strain along each element also decreases as do the displacements from the beginning of each element. The surface

friction of the geogrid and the soil to soil friction in the aperture as well as bearing resistance on the transverse member of the geogrid are taken into account. The result is shown as a decrease in mobilized shear resistance along the geogrid from the free end (pulled end) to the embedded end.

2. Equilibrium Condition

Referring to Fig. 3.15, the equilibrium of a element in the geogrid can be written as

$$[F(x)-dF(x)] + 2qdx - F(x) = 0 \quad (3.41)$$

$$q = dF(x)/2dx \quad (3.42)$$

$$\tau(x) = dF(x)/2wdx \quad (3.43)$$

where $F(x)$ = the mobilized tensile force in the geogrid at any point (x) ,

q = shear flow at that point (x) ,

$\tau(x)$ = interface shear stress at point (x) ,

w = width of the geogrid.

3. Interface Model

The interface shear stress - displacement relationship of the soil - geogrid interface can be assumed by an elastoplastic strain hardening model (which can be obtained from a sliding test as described in Chapter 6). The model can be expressed as follows (Fig. 3.16):

$$\tau (x) = \frac{Ay}{B+Y} \quad (3.44)$$

where y = the shear displacement at point (x) ,

A, B are constants which can be obtained from curve fitting from the results of the sliding test.

4. Reinforcement Model

An elastic behaviour of the reinforcement is assumed,

$$\epsilon(x) = dy/dx = F(x)/EA \quad (3.45)$$

where $\epsilon(x)$ = strain developed in the reinforcement at point (x),

E = elastic modulus of the geogrid,

A_c = cross section area of the geogrid,

y = displacement of the geogrid and

x = coordinate along the geogrid.

The geogrids used in pullout were the same geogrids as used in direct shear, i.e.

Conwed 5033. Its tensile behaviour was shown in Fig. 3.9.

5. Formulation

Differentiating Equation (3.45)

$$d^2y/d^2x = dF(x)/dx * EA_c \quad (3.46)$$

and substituting Equations (3.43), (3.44) into (3.46)

$$\frac{d^2y}{d^2x} = \frac{2w}{EA_c} \tau(x) = \frac{2w}{EA_c} \frac{Ay}{B+y} \quad (3.47)$$

Let $\alpha = 2wA/EA_c$

$$\therefore \frac{d^2y}{d^2x} = \frac{\alpha y}{B+y} \quad (3.48)$$

Integration of equation (3.48) is done as follows

$$\frac{dy}{dx} = p, \quad \frac{d^2y}{dx^2} = \frac{d}{dx} \left(\frac{dy}{dx} \right) = \frac{dp}{dx}$$

$$\frac{dp}{dx} = \frac{dp}{dy} \frac{dy}{dx} = p \frac{dp}{dy}$$

$$\frac{d^2y}{dx^2} = p \frac{dp}{dy} = \frac{\alpha y}{B+y}$$

$$\int p dp = \int \frac{\alpha y}{B+y} dy$$

$$\frac{1}{2} p^2 = \alpha [y - B \ln(B+y)] + c_1$$

$$\frac{dy}{dx} = p = \sqrt{2\alpha [y - B \ln(B+y)] + c_1}$$

$$x = \int \frac{dy}{\sqrt{2\alpha [y - B \ln(B+y)] + c_1}} + c_2 \quad (3.49)$$

$$F(x) = EA_c \frac{dy}{dx} = \sqrt{2\alpha [y - B \ln(B+y)] + c_1} \quad (3.50)$$

6. Boundary Conditions

1. when $x=l$, $F(l) = 0$

From Equation (3.50)

$$0 = EA_c \sqrt{2\alpha[y - B \ln(B+y)]} + c_1$$

$$c_1 = -2\alpha[y_1 - B \ln(B+y_1)]$$

$$c_1 = \lim_{y_1 \rightarrow 0} [-2\alpha[y_1 - B \ln(B+y_1)]] = -2\alpha B \ln B$$

$$F = \lim_{y_1 \rightarrow 0} [EA_c \sqrt{2\alpha[y - B \ln(B+y)]} - 2\alpha B \ln B] \quad (3.51)$$

2. when $x=0$, $y(0)=y_0$

From Equation (3.49)

$$c_2 = - \int_0^{y_0} \frac{dy}{\sqrt{2\alpha[y - B \ln(B+y)]} - 2\alpha B \ln B} = 0 \quad \text{when } y_0 = 0$$

$$\therefore x = \int_0^y \frac{dy}{\sqrt{2\alpha[y - B \ln(B+y)]} - 2\alpha B \ln B} \quad (3.52)$$

$$F_0 = \lim_{y_1 \rightarrow 0} F(0) = EA_c \sqrt{2\alpha[y_0 - B \ln(B+y_0)]} - 2\alpha B \ln B \quad (3.53)$$

where y_0 = front end displacement of the geogrid

3.6.3. Numerical Simulations

Equation (3.53) can be used to predict the pullout force with front end displacement relationship in pullout test. Figure 3.17 shows a relationship of pullout force F_0 versus front end displacement y_0 as predicted by the model together with an actual test result for comparison. A further detailed comparisons between the model's prediction with test results will be presented in Chapter 9 of this thesis.

Fig. 3.1. Stress and Strain Variables for Plane Strain Condition
(after Wood 1990)

Fig. 3.2. Schematic Diagram of Standard Direct Shear Box

Fig. 3.3. Mohr Circle of Stress and Strain
(after Palmeira 1987)

Fig.3.4. Effect of Shearing on the Volume of Dense and Loose Sands (after Casagrande, 1936)

Fig. 3.5. Analogy between Direct Shear Box and Interlocking Saw Blades

(after Wood 1990)

A resolution test chart featuring 12 sets of patterns. Each set consists of five vertical lines of equal height and five horizontal lines of equal width, arranged in a cross-hatch pattern. To the right of each pattern is a numerical value. The values are: 1.0, 1.1, 1.25, 1.4, 1.6, 1.8, 2.0, 2.2, 2.5, 2.8, 3.2, 3.6, 4.0. The patterns and numbers are arranged in a grid-like fashion, with the numbers increasing from top-left to bottom-right.

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Fig. 3.6. Flow Rule of Coarse Sand

Fig. 3.7. Flow Rule of Crushed Limestone Aggregate

Fig. 3.8. Analogy between Earth Structure and Direct Shear Test

Fig. 3.9. Reinforced Soil Model in Direct Shear

Fig. 3.10. Force Strain Relationship for Conwed 5033

Fig. 3.11. Shear Strength Increase Predicted by the Proposed Model

Fig. 3.12. Equilibrium of Reinforcement in Direct Shear

Fig. 3.13. Effect of the Original Orientation of the Reinforcement

Fig. 3.14. Effect of Soil Dilatancy

Fig. 3. 15. Model of Load Transfer in Pullout Test

Fig. 3. 16. Simulation of Interface Model

Fig. 3.17. Numerical Simulation of Pullout Force from Proposed Model

CHAPTER 4

TEST APPARATUS AND EXPERIMENTAL PROCEDURES

4.1. INTRODUCTION

Direct shear and pullout tests were chosen in this investigation based on the main objectives of this study and on the analysis of previous research (Chapter 2) as well as on the theoretical considerations of reinforced soil (Chapter 3). It has been shown in experimental studies that the scale of the test apparatus has a significant effect on the results and consequently on the extrapolation of the test data for design parameters (Palmeira 1989). In order to minimize the scale effects and to obtain a useful data base for reinforced granular soils a large shear apparatus (1000x1000x940 mm) was built and a large pullout box (1032x960x1020 mm) was modified. Both devices were automated (computer controlled) and equipped with a data acquisition system.

4.2. DIRECT SHEAR TEST APPARATUS

One of the early soil testing apparatus was a direct shear box (Fig.4.1). It was developed by Collin (1846) because it was observed that in many landslides in clay failure occurred on a thin well-defined failure surface. In a shear box the failure in a soil specimen occurs in a similar fashion namely on a thin and well-defined surface. The soil is loaded by a normal load to simulate the surcharge effect in the field. The horizontal displacement is monitored along with the shear force. A typical shear force versus horizontal displacement relationship for dense sand is given in Fig. 4.2. This figure shows three results from this study.

The direct shear test remains popular in soil testing technique due to its many advantages but it also has several shortcomings as mentioned by Potts (1986). The popularity of the direct shear test is due to the fact that it is inexpensive , fast, and simple to carry out. It is a rapid test in free draining soils to measure short term shear strength parameters. For saturated soils of low permeability the test will yield parameters in terms of total stress. The apparatus is also available in various sizes and it can be modified to accommodate soils with reinforcement elements. Another advantage of the apparatus is that it can be used to study the soil-reinforcement interface behaviour since it has a well defined shear surface.

There are some disadvantages for the direct shear apparatus. Jewell (1980) stated that a detailed stress-strain behaviour for unreinforced and reinforced soils cannot be determined directly from the boundary measurement of force and displacement, because the direction of the principal stress planes rotate as the shear strain is increased.

The other disadvantage of the direct shear test (Ingold 1982; Richard and Scott 1985) is due to the problem of stress concentration at the test specimen boundaries, which leads to a non- uniform stress distribution within reinforced soil specimens. The development of passive and active earth pressure zones inside the specimen may cause the loading plate to tilt, which results in non- uniform vertical stress distribution at the shear surface. Furthermore, the shear displacement is limited to the travel length of the direct shear box. All these disadvantages can be overcome or minimized by increasing the size of the shear box.

4.2.1. Design of a Large Direct Shear Apparatus

As mentioned before the size of the test apparatus is likely to affect the boundary condition in this test which in turn has an effect on the test results. The proper interpretation of the load transfer mechanism and design parameters are dependent on reliable test results. Therefore, a proper size of the shear apparatus plays an important role in the experimental study of the reinforced soil.

1. Factors affecting the design of the direct shear box

It is desirable to have a shear apparatus as large as possible in order to minimize the boundary effects. There are several arguments which can be made in favour of a large test apparatus:

- 1). It can simulate field conditions by accommodating coarse granular soil compacted to specific conditions.
- 2). A large shear box can accommodate a large volume of soil and representative reinforcement specimens. This arrangement can minimize the interference between the soil and reinforcement, as well as the boundary effects.
- 3). Boundary effects are minimized.
- 4). The effect of embedded instruments on the test results are minimized.
- 5). A large number of instruments, external and internal can be employed to study the various parameters.
- 6). Control and data acquisition systems can be automated.

On the other hand, some disadvantages also exist:

- 1). Cost of equipment is high.

2). Cost to perform a test itself is high. These costs will include labour, time to prepare the test and the time to run the test, as well as the data analysis.

3). A large apparatus limits the surcharge load and stress level.

These advantages and disadvantages were carefully weighted against the objectives of this research. A shear box of dimensions approximately 1000x1000x1000 mm was chosen. This dimension was selected on the following considerations. The most common vertical space between two adjacent layers of the reinforcement in a reinforced soil structure, such as retaining walls and slopes is about 1000 mm. Therefore, the selected dimension (1000x1000x1000 mm) of the shear box will represent a unit cell of one reinforcement with a half metre of soil below and above. Furthermore, this dimension would eliminate the boundary effect and interference between the geogrid and the shear box wall (the soil volume is over 250 times of the volume of the geogrid and the geogrid could be placed away from the influence of the box wall). This size was also used by a previous research (Palmeria 1989). The size of the shear box also fulfils the requirement stipulated by Jewell (1980) where the minimum box dimension should be at least 50 times the particle size. In order to compare the test results of this study with published results in the literature these dimensions would be appropriate. Another advantage of this size box is that a representative geogrid sample could be used in the investigation. Each test required one cubic metre of soil to be placed under controlled conditions. The maximum width of a geogrid that could be placed in the box is 1000 mm, i. e. ten to twenty ribs for most geogrids presently on the market.

2. Mechanical details of the direct shear box

The shear box was made of welded steel channels and its mechanical design details are given in Appendix A. The shear box basically consists of top and bottom halves. Each half of the box was made from four panels bolted together at the corners. The panels consisted of two C-channels welded together at the flanges. The bottom part of the box was anchored to the concrete floor. The top half was connected to an MTS hydraulic actuator (Fig. 4.3). The shear force was supplied by the hydraulic actuator which moved the top portion of the box horizontally along the shear plane.

In order to minimize wall friction the inside of the box was lined with smooth polished stainless steel sheets.

4.2.2. Test Setup

The direct shear apparatus consisted of the following components (a) shear box, (b) shear force application, (c) surcharge load application, (d) instrumentation, and (e) data acquisition system. A schematic diagram of the test setup is shown in Fig. 4.3. While Fig. 4.4 depicts the test setup.

The shear box was the major component of the test system. The soil and reinforcement were placed in the box and sheared horizontally along the shear plane. Figure 4.5 gives a clear view of the empty shear box before a test.

The shearing force application consisted of an MTS function generator, a servo-hydraulic controller, an actuator assembly and reaction frame. The hydraulic actuator was bolted to the reaction frame which in turn was anchored to the concrete floor. The actuator was equipped with a load cell and an internal Linear-Variable-Displacement-

Transformer (LVDT). The shear force was measured by the load cell which had a capacity of $100 \pm 1 \text{ kN}$ and the shaft movement (stroke) was monitored by the LVDT with an accuracy of $\pm 0.8 \text{ mm}$. The total shear displacement was 76 mm (i. e. 3 inches).

The surcharge load application consisted of a dead-load system of a maximum of nine concrete blocks having different weights. Three surcharge intensities were used in this test program were: 23, 28 and 35 kPa (Fig. 4.6). These surcharge intensities are quite low in terms of simulating the overburden pressure that a soil is subjected to in the field. The maximum surcharge of 35 kPa is only equivalent to a depth of 1.5 m overburden. Using an air bag system seemed to be a good choice to give a higher surcharge, but the problem was that this system could not accommodate the movement of the shear box. A air bag with a moving frame at the same shear rate as the box would have been difficult to realize and expensive. Under the given condition the maximum surcharge load that could be reached with a stack of three blocks was 35 kPa .

The instrumentation for this investigation included both load and displacement measurements. The shear force was monitored by the built-in load cell of the actuator (Fig. 4.3). The stroke displacement of the actuator was measured by an internal LVDT which was used as a reference. Horizontal displacements of the top half of the shear box were monitored by two LVDTs located at the front panel of the box. In addition four LVDTs were used to measure the vertical displacement of the soil specimen (Fig. 4.7).

The data acquisition and controlling system consisted of a PC, a Taurus 9836 instrument controller, a HP 3497A data acquisition/controller unit and a HP 7046B X-Y plotter. The system was controlled by the HP 3094A data acquisition/controller unit run

by the HP 9836 controller. The output data were displayed on the computer screen for visual observation. The horizontal displacement versus shear force was plotted by the X-Y plotter (Fig. 4.8).

The system was set to control the shearing rate and to collect data at pre-programmed intervals. In this research the time interval for data collection was two seconds. During a test the data were stored on the PC's hard disc and later transferred to a floppy disc for further analysis.

4.3. TEST PREPARATION AND PROCEDURE IN SHEAR TESTING

The shear box was assembled and an operational check was carried out before placement of any soil was done. The moist granular soil was weighted and deposited in layers into the lower half of the box. Each layer was about 150 mm high. After placement of a layer it was compacted with an electric impact hammer having an attached flat rectangle plate (Fig. 4.9). The density and moisture content were checked with a nuclear density meter and were verified by taking samples from the compacted soil.

All geogrid specimens were cut to the same width of about 975 mm and to different lengths depending on the specified mesh inclination. For geogrids inclined at zero degree with respect to the normal of the shear plane, the geogrid was anchored to the floor and was also held in position by a clamping system before the soil was placed (Fig. 4.10). For ninety degree inclination, that is in a sliding test, the mesh was kept in place by clamping it to the stationary bottom half of the shear box (Fig. 4.11). For other inclinations, such as forty five and thirty degrees, two sets of plywood frames were built

in order to keep the geogrid in place at the specified inclination during soil placement (Fig. 4.12).

The soil surface was carefully levelled as it reached the top of the shear box. The geogrid was then folded along the top surface before the surcharge was applied by means of a stack of concrete blocks (i.e., dead load system). The soil was allowed to consolidate under the specified surcharge load for several hours before the shearing force was applied. The granular fill material underwent little settlement.

The last phase of the test preparation was to install and connect the LVDTs. The shearing force actuator was switched on as soon as the operational check was completed. The rate of shear force application was 1 mm/min. This rate is commonly accepted for shear tests of granular soils (Shang 1989). A test was terminated automatically by the HP 9836 controller when the horizontal displacement of the shear box reached 76 mm. This displacement was found sufficient in all tests to mobilize post peak strength behaviour (i.e., residual strength of the unreinforced and reinforced sand).

4.4 PULLOUT TEST APPARATUS

Two pullout test apparatuses were used in this investigation. A small rig (1040x230x350 mm) was used in a preliminary study to select the suitable geogrid for further study. A large tank (1550x900x1220 mm) was modified and automated to investigate the pullout behaviour of the selected geogrids in detail.

4.4.1. Small Pullout Apparatus

The small pullout apparatus consisted of the following components, (a) steel tank,

(b) pullout mechanism, (c) surcharge load, and (d) data acquisition and control system.

The steel tank was made of steel C-channels and steel plates. The side walls consisted of two 390x84 mm C-channels. The base and the end walls were made of 10 mm thick steel plates (Fig. 4.13). The interior surface of the walls were lined with smooth sheets of polished stainless steel in order to minimize side friction. A horizontal slot of 10 mm height was cut in one end at the mid-height of the tank to allow the geogrid to be pulled through.

The geogrid, fed through the slot of the tank, was gripped by a clamping assembly, which in turn was fastened to the pulling mechanism. The pullout mechanism was applied through a mechanical gear box and piston assembly (Fig. 4.14). The gear box was driven by a variable speed electric motor. A constant pullout rate of 1 mm/min. was used in this investigation based on previous experience (Mowafy 1986).

The surcharge load was supplied by lead and steel shot. The shot was contained in cloth bags weighing 135 kN each.

The data acquisition system consisted of a load cell to monitor the pullout force, a LVDT to monitor the free end displacement of the geogrid and a x-y plotter to record the pullout force versus geogrid displacement response.

Figure 4.15 shows a schematic diagram of the setup of the small pullout apparatus set-up.

4.4.2. Large Pullout Apparatus

The large pullout apparatus consisted of similar components as the small rig and included the following items, (a) large concrete tank , (b) pullout mechanism, (c) vertical

surcharge load system , and (d) data acquisition and control system.

The pullout tank was made of reinforced concrete panels. The four walls were bolted together. The tank had inside dimensions of 1550 mm long, 900 mm wide and 1220 mm deep. The inside of the wall panels was lined with polished stainless steel to reduce side friction effects. There was a 220 mm wide slot in the front panel of the tank at mid-height to allow the geogrid to be pulled through. A piece of styrofoam was glued to both sides of the slot to prevent soil loss and to minimize stress concentrations.

The pullout mechanism consisted of a MTS function generator, a servo - hydraulic controller, an MTS actuator and a clamping assembly. This actuator system was the same as used in the direct shear test. A detailed description was given in section 4.2.2. The clamping assembly is shown in Fig. 4.16. Pieces of stiff rubber were inserted between the two steel plates to prevent the geogrid from slipping. The surcharge load consisted of a vertical load frame, an MST actuator, and a control system. In addition a reinforced concrete slab (1435x840x150 mm) was used as a rigid top cap to distribute the actuator load to the soil.

The data acquisition and control system was same as in the direct shear test. The pullout load was monitored by the built - in load cell. The front end displacement was monitored by two LVDTs (Fig. 4.17). Special efforts were made to monitor the strain development in the geogrid. Thin piano wires were attached at various locations of the geogrid and fed through plastic tubes (Fig. 4.18) to LVDTs located at the back of the concrete tank (Fig. 4.19) by means of wood box.

Figure 4.20 shows a schematic diagram of the test set - up. Figure 4.21 depicts a

front of view of the pullout test apparatus.

4.5. PULLOUT TEST PREPARATION AND PROCEDURES

4.5.1. Test Preparation and Procedures in Small Pullout Test

The sand was weighed and placed dry in layers into the box and tamped by hand to the required density. The specified geogrid was placed horizontally at the elevation of the slot and the free end of the grid was clamped to the pulling mechanism. Placing and compaction of the soil was continued until the tank was full. The soil was then excavated to slope towards the slot. A surcharge was placed at the soil surface. The load cell and displacement transducers were connected to a x-y plotter. After each test the tank was emptied and cleaned.

4.5.1. Test Preparation and Procedures in Large Pullout Test

The moist soil was deposited in the tank and compacted in layers by an electric impact hammer (Fig. 4.9). Each layer was 150 mm high. The geogrid was placed at midheight and fed through the slot of the tank. Piano wires were attached to three transverse members of the grid and threaded through plastic tubes (Fig. 4. 18). They were extended to the back of the tank and connected to LVDTs (Fig. 4.19). The compaction procedure was then continued until the tank was filled. The density was measured with a nuclear density meter. The soil surface was carefully levelled to ensure that the concrete slab was level and in uniform contact with the soil. The last phase of the test preparation was to install and connect the instruments and to perform an operational check.

The pullout actuator was switched on as soon as the operation check was finished.

The pullout rate was 1 mm/min., which corresponded to a strain rate of 0.065 per cent per minute. The selection of this strain rate was based on previous research (Mowafy 1986; Shang 1989). A plot of the pullout load versus the free-end geogrid displacement was recorded by a x-y plotter and the load and LVDTs readings were recorded by the computer. A test was terminated after either an almost constant pullout force was obtained or the pullout load reached the capacity of the actuator.

Fig. 4.1. Schematic Diagram of Standard Direct Shear Box

Fig. 4.2. Typical Direct Shear Test Results for Dense Sand

Fig. 4.3. Diagram of Direct Shear Box and Test Assembly

Fig. 4.4. Direct Shear Test Assembly

Fig. 4.5. Empty Direct Shear Box before Test

Fig. 4.6. Dead Load Surcharge Blocks

Fig. 4.7. Layout of LVDTs

Fig. 4.8. Data Acquisition System

Fig. 4.9. Electric Compaction Hammer

Fig. 4.10. Location of Reinforcement when Orientated at $\theta=0^\circ$

Fig. 4.11. Location of Reinforcement when Orientated at $\theta=90^\circ$

Fig. 4.12. Reinforcement Orientated at 30° or 45°

Fig. 4.13. Small Pullout Test Tank (after Mowafy 1986)

Fig. 4.14. Gear Box Assembly for Small Pullout Apparatus (after Mowafy 1986)

Fig. 4.15. A Schematic Diagram of the Small Pullout apparatus (after Mowafy 1986)

Fig. 4.16. Clamping Assembly for Large Pullout Apparatus (after Shang 1989)

Fig. 4.17 Gripping Clamp and Horizontal Displacement Monitoring

Fig. 4.18. Deformation Monitoring in Geogrid

Fig. 4.19. LVDTs Monitoring Geogrid Deformation

Note: All dimensions given in mm

Fig. 4.20. Schematic Diagram of Large Pullout Apparatus

Fig. 4.21. Front of the Large Pullout Apparatus

CHAPTER 5

PROPERTIES OF TESTED MATERIALS

Two granular soils, a coarse sand and a crushed limestone aggregate, were tested as backfill materials in direct shear and pullout. The coarse sand represents a free draining soil commonly specified as backfill while the well graded crushed stone aggregate represents a granular soil used for load bearing base courses. Coarse sand, in terms of shear strength, is a low to medium strength granular soil depending on density. On the other hand, crushed limestone aggregate is a high strength material. Three geogrids were chosen as reinforcements, Conwed 5033, Tensar UX 1500 and Mirafi 5-Ton. The decision was based on a literature review, previous research (Bauer and Mowafy 1989; Mowafy 1986) and on preliminary pullout test results (Chapter 7) in this investigation. Conwed 5033 represents a flexible, medium high strength geogrid (Mirafi 5-ton mesh is a similar geogrid as Conwed 5033) while Tensar UX 1500 is a stiff, high strength geogrid. This chapter will present the basic physical properties of these two granular soils investigated in this research. Six direct shear tests and four repeated tests were carried out to obtain the strength and deformation properties. The properties of geogrids are also given as obtained from a previous study (Razaqpur 1990).

5.1. BASIC PROPERTIES OF COARSE SAND AND CRUSHED LIMESTONE AGGREGATE

5.1.1. Properties of Coarse Sand

The coarse sand, commonly known as concrete sand, was selected because it is

commonly specified as backfill material and it is also readily available in most parts of the world. This soil has also been used in previous investigations (Mowafy 1986; Bauer and Mowafy 1988b) and therefore its properties are well established. The grain size distribution of the sand was obtained from sieve analysis and is shown in Fig. 5.1. The modified Proctor compaction test yielded an optimum moisture content of 11 percent corresponding to a maximum dry density of 19.8 kN/m^3 . The dry density corresponding to 90 percent of modified Proctor compaction was 18 kN/m^3 with a corresponding moisture content of 8 percent. This compaction effort is generally specified by the Ministry of Transportation for field placed soil. Previous investigations have also used the same specified density (Mowafy 1986; Chan 1987).

5.1.2. Properties of Crushed Limestone Aggregate

The aggregate was crushed from a good quality limestone rock meeting the specifications for Granular "A" (Ministry of Transportation of Ontario). It can be described as an angular gravel consisting entirely of hard limestone particles. Figure 5.1 also shows the grain size distribution for this material. The modified Proctor compaction test yielded an optimum moisture content of 6.8 percent with a corresponding maximum dry density of 22 kN/m^3 . In this investigation, as in previous studies, 85 percent of modified Proctor density was used. The corresponding dry density was about 19 kN/m^3 with a water content of 3 to 4 percent.

5.2. SHEAR BEHAVIOUR OF COARSE SAND AND CRUSHED AGGREGATE

Coarse granular soils have been widely used as backfill materials for embankment, trenches and earth retaining structure due to their high strength, good drainage and

excellent compaction properties. To test the shear strength of these soils in a conventional test apparatus is a problem due to the large particle sizes versus the limited size of standard shear boxes (60x60x30 mm). The maximum particle size for the crushed aggregates used in this study was 20 mm in diameter. The AASHTO-AGC-ARTBA Joint Committee Task Force 27 recommended that the particle size be limited to 20 mm.

If a large shear box is not available one generally sieves down the material until it meets the specification for a standard shear box and then tests the finer material. The minimum dimension of the test apparatus to the mean particle size should be in the order of 50 (Jewell 1987). Therefore it stands to question the sieved down finer soil will represent the actual mechanical properties of the original coarse soil. In order to overcome this problem and to evaluate the "actual" physical strength parameters of a coarse granular soil, a large shear box was built. The mechanical details of the shear box were explained in chapter 4 and are given in Appendix A.

5.2.1. Shear Behaviour of Coarse Sand

Three direct shear tests were carried out on the natural coarse sand. A number of tests were also repeated to verify the test results and the reliability of the test apparatus. These tests were run at different normal stresses to simulate different overburden pressures in the field. Shear strength parameters were obtained from the failure envelopes of peak shear stress versus normal stress relations. Figure 5.2 shows the result of shear force versus shear displacement. Initially the shear force increased linearly with the shear displacement, then it increased with the further shear displacement at a lower and different rate. A distinct peak value was observed for all three tests. The peak value

occurred between 6 to 9 mm of shear displacement, depending on the applied normal stress. The sand experienced a rapid decrease in strength after the peak value with further increase of shear displacement. The residual shear strength was in the order of 70 -80 percent of peak strength. This behaviour is common for dense to very dense granular soils during shear.

In order to obtain the strength parameter of coarse sand the overall normal stress versus shear stress curve was plotted (σ_{yy} versus τ_{xy}) in Fig. 5.3. The shear strength for a dense sand in its natural state can be written as

$$s = \sigma' \tan \phi' \quad (5.1)$$

where s = shear strength

σ' =effective normal stress

ϕ' =effective angle of internal friction

From the measured quantities equation 5.1 can be written for the coarse sand as:

$$s = 1.04 \sigma' \text{ (peak)} \quad (5.2)$$

$$s = 0.73 \sigma' \text{ (residual)} \quad (5.3)$$

Table 5.1. summarises the test results for the sand.

5.2.2. Shear Behaviour of Crushed Limestone Aggregate

The shear force versus shear displacement relationship of the crushed limestone aggregate is shown in Fig. 5.4. A peak value was also observed. Unlike coarse sand the aggregate experienced a gradual decrease in shear resistance after peak. The residual shear resistance was in the order of 75-85 percent of the peak shear resistance. A normal stress versus shear stress relationship was also obtained for the crushed limestone

aggregate as shown in Fig. 5.5. A summary of the test results for the aggregate is given in Table 5.2. A linear relationship was obtained from equation (5.1) for the range of normal stresses applied, which can be written as:

$$s = 1.38 \sigma' \text{ (peak)} \quad (5.4)$$

$$s = 0.88 \sigma' \text{ (residual)} \quad (5.5)$$

TABLE 5.1. SHEAR STRENGTH PROPERTIES OF COARSE SAND

Test No.	Normal Stress (kPa)	Shear Stress (kPa)		Friction Angle ($^{\circ}$)	
		Peak	Residual	Peak	Residual
1	23	23.7	16.3	45.8	34.5
2	28	27.1	21.5	44.1	37.5
3	35	36.3	25.2	46.0	36.8
		Avg: 45.3		36.3	

TABLE 5.2. SHEAR STRENGTH PROPERTIES OF CRUSHED LIMESTONE AGGREGATE

Test No.	Normal Stress (kPa)	Shear Stress (kPa)		Friction Angle ($^{\circ}$)	
		Peak	Residual	Peak	Residual
4	23	32.7	22.7	54.8	44.6
5	28	39.5	28.7	54.7	45.7
6	35	45.0	32.7	52.0	43.1
		Avg: 54.3		44.5	

5.3. DEFORMATION (DILATANCY) BEHAVIOUR OF TESTED GRANULAR SOILS

It is well known that dense granular soils tend to dilate, or to increase in volume when sheared. In a direct shear test the shear plane occurs in the region between the two halves of the shear box. The soil in this zone expands as the shearing takes place. This dilative behaviour was observed in all six tests and also in the repeated tests. The measurement of soil dilatancy is, however, difficult to accomplish. A boundary measurement technique was adopted in this investigation. One LVDT was mounted on each of the corners of the upper half of shear box just above the shear plane. A vertical displacement versus shear displacement curve was obtained. The results will be used to calculate the soil dilatancy (Chapter 3 and 8).

Figure 5.6 represents the dilatancy behaviour of coarse sand from three tests. In all tests an increase in volume occurred on the shear plane. Similar dilatant behaviour for the crushed limestone aggregate was observed as shown in Fig. 5.7.

There are several important observation which can be made from the test results as shown in Figs. 5.6 and 5.7.

- (1) Initially a small contraction in volume or very small expansion exists for all granular soils. It can be related to the rearrangement of soil particles in a dense pattern.
- (2) The soil starts to expand very rapidly with shear displacement after the small contraction.
- (3) The magnitude of the change of the vertical displacement in direct shearing

increases with normal stress, but a definite relationship was not established.

5.4. PROPERTIES OF TESTED GEOGRIDS

5.4.1. Property of Tensar UX 1500

From previous research (Chan 1987; Mowafy 1987; Bauer et al. 1988; Bauer et al. 1990) it has been shown that Tensar geogrid exhibits high strength and is therefore used extensively for soil reinforcement. There are two basic types of Tensar polymer grids, uniaxial and biaxial (Fig. 5.8), depending on the manufacturing process. Uniaxial grids are made by stretching a punched sheet of extruded polymer in one direction. This process aligns the polymer's long chain molecules in the direction of draw and results in a product with a high one-directional tensile strength and modulus. Biaxial grids on the other hand, are manufactured by heating and stretching the punched sheet in two orthogonal directions. This process results in a product with similar tensile strength and modulus in both directions.

The uniaxial geogrid, Tensar UX 1500 (Fig. 5.9), was used for this investigation. Table 5.3. presents some of the mechanical properties of Tensar UX 1500 geogrid, while Fig. 5.10 shows the force-strain relationship for this geogrid. The size of grid specimen used in this study in direct shear and pullout tests were different. Full details will be given later.

5.1.2. Properties of Conwed STRATAGRIDS¹

Conwed STRATAGRIDS are thermoplastic polymer grids. The basic chemical

¹. STRATAGRIDS is a registered tradename of Conwed Plastic Co. of Minneapolis, MN, U. S. A.

component is polyester and the strands are knitted in orthogonal directions, leaving a predetermined aperture. The grid is coated with polyolefin to resist chemical and moisture attacks and to give additional protection against abrasion. Four different kinds of Conwed Stratagrids were used in the initial investigation, namely 9027, 5033, 3024 and 3022 (Fig. 5.11). Tables 5-4 through 5-7 show the strength and stiffness characteristics of these grids. These properties were determined in previous investigations at Carleton University (Razaqpur 1990).

TABLE 5.3. PROPERTIES OF TENSAR UX 1500

TEST TYPE	TENSILE STRENGTH(kN/m)	STRAIN PERCENT (5%)
Liquid Metal M. S. D. C. O. V.	88.25 0.74 0.83 %	10.2 0.53 5.21 %
Concrete M. S. D. C. O. V.	93.47 N/A N/A	13.6 N/A N/A
Wedge M. S. D. C. O. V.	84.88 2.45 2.88 %	10.7 0.13 1.16 %
Overall M. S. D. C. O. V.	88.29 3.77 4.27 %	11.21 1.49 13.00 %

Note: M = mean; S. D. is standard deviation; C. O. V. is the coefficient of variance

A typical force - displacement curve for Stratagrid 5033 in machine direction (MD) are shown in Fig. 5.12. From these curves and Table 5.5, the following

observations can be made. Stratagrid 5033 has a mean MD strength of 5.3 ton/m. The mean stiffness (secant modulus) is in the range of 2.4 to 2.8 kN/m. It also can be noticed that the strength in transverse direction (TD) of the same grid varies from 4.2 to 4.6 ton/m. Therefore one can state that the TD strength is about 80% of its MD strength. All the tests in this investigation were carried out along MD because of the higher strength and the recommendation given by the manufacturer. The overall dimensions of grid specimens vary according to test purpose.

Conwed Stratagrid 5033 was selected in direct shear tests because it has high tensile strength and modulus according to the preliminary pullout test results (Chapter 7). In all tests the width of the Stratagrid specimens was 975 mm. The length varied according to the orientation of the grid in the shear box and the embedded length in a pullout test.

TABLE 5.4. STRENGTH AND STIFFNESS PROPERTIES OF STRATAGRID 9027 IN MD DIRECTION

Type of Stratagrid	No. of Strands	Length (mm)	Width (mm)	Max. Strain %	Strength			Secant Modulus	
					(T/m)	(kN/m)	(lbs/ft)	(kN/m)	(lbs/ft)
10T	5	248	146	13.6	8.6	84.3	5777	5.9	404
10T	5	232	146	14.0	8.8	86.2	5911	5.9	404
10T	5	230	146	14.8	8.6	84.3	5777	5.4	370
10T	5	218	146	15.2	9.0	88.2	6045	5.5	377
10T	5	225	146	15.5	8.9	87.2	5978	5.3	366
Mean	5	230.6	146	14.5	8.8	86.2	5898	5.6	384
Standard Deviation	0.0	29.3	0.0	0.8	0.2	1.8	119.9	0.3	18.5

**TABLE 5.5. STRENGTH AND STIFFNESS PROPERTIES OF STRATAGRID 5033
IN MD DIRECTION**

Type of Stratagrids	No. of Strands	Length (mm)	Width (mm)	Max. Strain %	Strength			Secant Modulus	
					(T/m)	(kN/m)	(lbs/ft)	(kN/m)	(lbs/ft)
5T	5	245	145	16.7	5.1	50.0	3426	2.6	178
5T	5	247	145	16.0	5.2	51.0	3493	2.8	192
5T	5	217	145	15.3	5.3	51.9	3560	3.0	206
5T	5	185	144	16.5	5.3	51.9	3560	2.8	192
5T	5	182	144	17.2	5.3	51.9	3560	2.6	178
Mean	5	215.2	144.6	16.3	5.2	51.3	3520	2.8	189
Standard Deviation	0.0	31.3	0.6	0.7	0.1	0.8	59.9	0.2	11.7

**TABLE 5.6. STRENGTH AND STIFFNESS PROPERTIES OF STRATAGRID 3024
IN MD DIRECTION**

Type of Stratagrid	No. of Strands	Length (mm)	Width (mm)	Max. Strain %	Strength			Secant Modulus	
					(T/m)	(kN/m)	(lbs/ft)	(kN/m)	(lbs/ft)
X34	5	215	83	14.7	3.2	31.4	2149	1.9	130
X34	5	220	83	16.9	3.4	33.3	2284	1.7	117
X34	5	210	83	17.4	3.3	32.3	2217	1.7	117
X34	5	215	82	14.5	3.4	33.3	2284	2.1	144
X34	5	200	82	15.8	3.6	35.3	2418	2.0	137
Mean	5	212	82.6	15.9	3.4	33.1	2270	1.9	129
Standard Deviation	0.0	7.6	0.5	1.3	0.2	1.5	99.7	0.2	12.0

**TABLE 5.7. STRENGTH AND STIFFNESS PROPERTIES OF STRATAGRID 3022
IN MD DIRECTION**

Type of Stratagrid	No. of Strands	Length (mm)	Width (mm)	Max. Strain %	Strength			Secant Modulus	
					(T/m)	(kN/m)	(lbs/ft)	(kN/m)	(lbs/ft)
X22	5	205	85	15.5	2.9	28.4	1948	1.6	110
X22	5	205	85	15.0	3.0	29.4	2015	1.6	110
X22	5	205	84	13.7	2.9	28.4	1948	1.9	130
X22	5	205	85	15.6	2.9	28.4	1948	1.6	110
X22	5	205	84	15.5	3.1	30.4	2082	1.7	117
Mean	5	205	84.6	15.1	3.0	29.4	2015	1.7	117
Standard Deviation	0.0	0.0	0.5	0.8	0.1	1.0	67.0	0.1	8.9

5.1.3. Properties of Mirafi 5-Ton Mesh

The Mirafi 5-ton mesh is basically similar to Coated Stratagrid 5033. It is made of knitted polyester strands and is covered with a black polyolefin coating. The mesh is shown in Fig. 5.13.

Fig. 5.1 Grain Size Distribution for Tested Soils

Fig. 5.2 Shear Force versus Shear Displacement (Coarse Sand)

Fig. 5.3 Shear Stress versus Normal Stress (Coarse Sand)

Fig. 5.4 Shear Force versus Shear Displacement
(Crushed Limestone Aggregate)

Fig. 5.5 Shear Stress versus Normal Stress (Crushed Limestone Aggregate)

Fig. 5.6 Dilatancy Behaviour (Volume Change) of Coarse Sand

Fig. 5.7 Dilatancy Behaviour (Volume Change)
of Crushed Limestone Aggregate

a) Tensar Biaxial Geogrids

b) Tensar Uniaxial Geogrids

Fig. 5.8 Tensar Uniaxial and Biaxial Geogrids

Fig. 5.9 Sample of Tensar UX 1500

Fig. 5.10 Force versus Strain Relationship of Tensar UX 1500

a) STRATAGRID 9077

b) STRATAGRID 5033

c) STRATAGRID 3024

d) STRATAGRID 3022

Fig. 5.11 Conwed 9037, 5033, 3025 and 3022 Geogrids

Fig. 5.12 Force Strain Relationship of Conwed 5033

Fig. 5.13 Sample of Mirafi 5 Ton Geogrid

CHAPTER 6

EXPERIMENTAL PROGRAM AND TEST RESULT OF DIRECT SHEAR TESTS

As stated in Chapter 1 the main objective of this research was to investigate the shear strength and deformation behaviour of reinforced soils. In order to achieve this objective an extensive experimental program was conducted over the last three years. Chapter 4 and Chapter 5 gave the description of the test apparatus and the properties of the test material. This Chapter will present the results of direct shear tests as part of the experimental program, while the results of the pullout test will be given in Chapter 7. The analysis of these test results is then presented in Chapter 8 and Chapter 9, respectively.

6.1. EXPERIMENTAL PROGRAM

6.1.1. Test Objectives

One of the main objectives of this research was to investigate the effect of the orientation of the reinforcement, with respect to the shear plane, on the strength behaviour of the composite. Figure 6.1 shows a vertical reinforced retaining wall. The angle of orientation, θ , of the reinforcement to the shear plane varies along the critical failure surface of the reinforced wall (Fig. 6.1a). This angle ranges from zero degrees at the bottom of the wall to about ninety degrees at the top of the wall. Several studies (Jewell 1980; Gray et al. 1983; Palmeira 1989) have shown that the orientation of the reinforcement with the shear plane has an important effect on the shear strength and deformation behaviour of the composite. However in the past reinforcing elements which were used in soil reinforcement consisted of steel bars, steel strips and membranes. With

the introduction of polymer geogrids, additional strength is imparted to the soil mainly due to the open structure of the grids providing interlock between the soil particles of a granular backfill. On the other hand polymer geogrids are very flexible compared to other reinforcement consisting of metal elements. Due to this structure and constitutive difference between the conventional metal reinforcement and polymer geogrids the interaction mechanism between reinforcing element and soil is also different. This investigation was aimed at conducting comprehensive experiments to study the interaction mechanism of geogrids with granular soils. This chapter presents the experimental results of the direct shear tests. The interaction mechanism will be discussed in Chapter 8 based on the test results presented in this chapter and theoretical considerations given in Chapter 3.

There were four main variables which were investigated in this study,

a) soil properties: two different granular soils were tested representing a large range of particle sizes (based on the analysis of previous research (Chapter 4)). A coarse sand and a crushed limestone aggregate were used. Their grain size distribution, composition, shear strength and deformation behaviour were discussed in the previous chapter.

b) geogrid properties: three different types of geogrids were investigated, Tensar UX 1500, a fairly stiff geogrid; Conwed 5033 and Mirafi are considered to be flexible geogrids. Their mechanical and "in soil" properties also vary.

c) orientation of geogrid: four different angles of orientation of geogrid with respect to the normal to the shear plane were investigated , 0° , 30° , 45° , and 90° .

d) normal loads: Three surcharge intensities were used in this study, 23, 28 and 35 kN/m².

In order to investigate the various factors influencing the shear behaviour only one variable at a time was changed and the remaining conditions were kept the same. For example, four tests were carried out with one particular soil, one geogrid and one surcharge load but only the orientation of the geogrid was varied. Another three tests were conducted by increasing the surcharge intensity and the other factors were kept the same. Using this methodology all variables in a series of tests were evaluated.

6.1.2. Experimental Program

Sixty-eight tests were conducted to study the effect of the orientation of the geogrid on the dilatancy and the strength response of reinforced soil. The soil dilatancy, shear strength and their interrelationships were of primary interest in this investigation. The influence of soil properties, the normal stress intensities and the properties of geogrid were also investigated. Fourteen tests were carried out with the objective of monitoring the deformation within the natural soil and the reinforced composite. Table 6.1 lists the experimental program for direct shear tests. In Table 6.1 the orientation of the geogrid refers to the angle between the geogrid and the normal to the shear plane in order to compare the test results with previous research (Jewell, 1980).

6.2. EXPERIMENTAL RESULTS

When reinforcing elements are added to a natural soil the shear strength and deformation behaviour of the soil will be modified. This modification can cause an

TABLE 6.1 EXPERIMENTAL PROGRAM FOR DIRECT SHEAR TESTS

Soil Type	Geogrid Type	Orientation of geogrid	No. of surcharge loads	No. of Tests
Coarse Sand	CONWED 5033	0°	3	3
		45°	3	3
		30°	3	3
		90°	3	3
	TENSAR 1500	0°	3	3
		45°	3	3
		30°	3	3
		90°	3	3
	MIRAFI	0°	2	2
		90°	2	2
		Repeat Test		6
		Total		34
Soil Type	Geogrid Type	Orientation of geogrid	No. of Surcharge	No. of Tests
CRUSHED LIMESTONE AGGREGATE	CONWED 5033	0°	3	3
		45°	3	3
		30°	3	3
		90°	3	3
	TENSAR 1500	0°	3	3
		45°	3	3
		30°	3	3
		90°	3	3
	MIRAFI	0°	2	2
		90°	2	2
		Repeat Test		6
		Total		34

increase or decrease in strength depending on the orientation of the reinforcement with the possible shear plane. In this investigation the reinforcement was placed in such a way (i. e. $0^\circ \leq \theta \leq 90^\circ$, θ refers to the angle between the geogrid and the normal to the shear plane) that the tensile force was mobilized in the reinforced elements. Therefore, beneficial effects (strength increase) were obtained in all of the tests except for the test when $\theta = 90^\circ$, i. e. sliding test. Fatani et al (1992) and Jewell (1980) have shown that when $\theta > 90^\circ$ that the shear strength of the composite is less than that of natural soil. In such cases ($\theta > 90^\circ$) the reinforcement element will actually undergo compression.

The test results for shear strength behaviour of reinforced soil presented here are in the same form of shear force versus shear displacement relationship as the corresponding results for soil. As a matter of fact the results obtained from unreinforced soil were used as control and for comparison. Shear strength parameters were then obtained from shear stress versus normal stress relations. The soil deformation behaviour which was of paramount interest in this investigation was soil dilatancy, or the plastic deformation behaviour of the soil. The inclusion of the reinforcement modified the dilatancy behaviour of the natural soil. The same technique to monitor the dilatancy behaviour of natural soil was used to monitor the composite. A set of vertical displacement versus horizontal shear displacement relationships was obtained and is also presented in this chapter.

6.2.1. Shear Strength Behaviour of Reinforced Soils

Figures 6.2 through 6.5 show the results of 12 direct shear tests on coarse sand reinforced with Conwed Stratagrid 5033. The test results revealed that at a geogrid

inclination of 90° with respect to the normal of the shear plane (i. e. sliding test), initially the shear force increased linearly with the shear displacement (elastic behaviour) for all three tests. A non-linear shear force increase with continued shear displacement was observed after this point in all three tests. At about 6 mm to 7 mm of shear displacement a peak shear force was reached in each of the tests. The shear force then decreased rapidly with further shearing. After 15 to 17 mm of total shear displacement, the shear force remained fairly constant. The total shear force decrease from the peak to the lowest point was in the order of 15 to 18 percent. Figures 6.3. to 6.4. show the results of six tests of coarse sand reinforced with Conwed 5033 orientated at an angle of 45° and 30° to the normal to the shear plane. These two figures show a different pattern of shear force versus shear displacement relationship from those of natural soils and from the sliding tests. Initially, like in natural soil, elastic behaviour dominated. A non-linear increase of shear force with continuing shear was then observed. At a shear displacement of 10 to 12 mm a slight decrease in slope occurred with continued shear displacement. Beyond a shear displacement, in the order of 15 mm, a modest constant increase in shear force was observed. In general, the behaviour of the composite at these two grid orientations can be idealized with a three-part relationship: a steep linear part during small shear displacement, a curved part at intermediate displacement followed by a linear part under continued shear. During the latter phase a slight increase at a rate of 0.2 to 1.0 kN per 10 mm was observed. The linear increase occurred for the higher surcharge intensity (35kN). Figure 6.5 shows the results of three shear tests on reinforced coarse sand when the geogrid was orientated at perpendicular to the shear plane. The results show a similar

pattern as in the sliding test. However the shear force at any displacement was about 10 to 20 percent higher than the corresponding response in the sliding tests.

Figures 6.6 through 6.9 show the result of twelve direct shear tests on coarse sand reinforced with Tensar UX 1500. Unlike the relationships for the coarse sand reinforced with Conwed 5033, all twelve tests show a similar response of shear force versus shear displacement. A distinct peak was observed in each of these tests. The shear displacement corresponding to the peak was much larger than the corresponding displacements for sand reinforced with Conwed 5033. This observation gives an indication of the influence of the properties of the reinforcement on the shear strength. This aspect will be discussed further in chapter 8.

Table 6.2 summarises all the above test results in terms of "peak" and "residual" shear force. As mentioned before some of the tests (Figs. 6.3 and 6.4) did not yield a peak value. The value used in the table were taken at the displacement corresponding to peak displacement of other tests. In some cases the "residual" values were higher than "peak" values due to the strain hardening behaviour of the composite. These residual values were taken at the end of a test, generally at a displacement of 60 mm.

The test results of reinforced coarse sand yielded some interesting phenomena. They can be presented as follows.

- 1) At 90° degree inclination, i. e. the reinforcement was placed on the shear plane (i.e. sliding test), the reinforced soil exhibited the same or slightly higher shear strength than the natural soil. Even in this location the geogrid still shows some reinforcing effect (increase in shear strength) or at least the same shear strength as unreinforced soil.

TABLE 6-2 TEST RESULTS FOR COARSE SAND

Test Series	Geogrid & Orientation (θ)	Surcharge (kPa)	Shear Strength Characteristics	
			Peak Value (kN)	Residual Value (kN)
7	Conwed* $\theta=90^\circ$	23	22.98	19.96
8		28	27.72	24.44
9		35	34.18	28.66
10	Conwed $\theta=45^\circ$	23	33.79	33.79
11		28	42.31	42.31
12		35	45.31	45.31
13	Conwed $\theta=30^\circ$	23	35.61	37.15
14		28	41.92	44.66
15		35	46.53	56.81
16	Conwed $\theta=0^\circ$	23	27.13	23.17
17		28	39.30	38.38
18		35	45.27	43.89
19	Tensar** $\theta=90^\circ$	23	21.74	16.10
20		28	24.62	19.34
21		35	33.53	26.91
22	Tensar $\theta=45^\circ$	23	41.76	36.92
23		28	45.72	43.96
24		35	56.26	54.51
26	Tensar $\theta=30^\circ$	23	41.76	42.64
27		28	47.03	46.69
28		35	58.02	52.51
29	Tensar $\theta=0^\circ$	23	32.97	35.60
30		28	37.80	40.44
31		35	46.59	47.03

Note: θ refers to the angle of geogrid intersection with the normal of the shear plane

* refers to Conwed Stratagrid 5033, ** Tensar UX 1500

2) At other grid orientations, i. e. 45° , 30° and 0° , the reinforced soil yielded much higher shear strength than that of unreinforced soil, not only for "peak" strength but also the so called "residual" strength increased significantly. For 45° and 30° inclinations no distinct "peak" values were observed (The "peak value" listed in Table 6.2 was taken at the corresponding horizontal displacement of "peak value" of the natural soil, and the "residual value" was taken when the horizontal displacement reached the maximum displacement of the test apparatus, which was 60 mm). The shear strength increased with shear displacement.

The shear strength behaviour of coarse sand reinforced with Conwed 5033 and Tensar UX 1500 is given in Figs. 6.10 to 6.17 which show the normal stress versus shear stress relationship. Table 6.3 is a summary of the results. Several observation can be made from the strength envelopes.

1) An approximately linear relationship between the normal stress and shear stress existed over the narrow range of normal stresses used in this study. All reinforced soils showed an intercept with the shear stress axis. This intercept represented the shear strength increase due to introducing the reinforcing elements. This behaviour was similar to that of cohesive soil and can be termed as "apparent cohesion" (Schlosser and Vidal 1969). Therefore, this relationship can be written as:

$$s = c_r' + \sigma' \tan \phi_r' \quad (6-1)$$

where s = shear strength of reinforced soil,

c_r' = "apparent cohesion"

ϕ_r' = effective friction angle of reinforced soil,

σ' = effective normal stress.

**TABLE 6.3. SHEAR STRENGTH PARAMETERS FOR REINFORCED
COARSE SAND**

Test Series	Geogrid & Orientation (θ)	Shear Strength Parameters			
		Friction Angle ($^\circ$)		Cohesion (kPa)	
		Peak	Residual	Peak	Residual
7-9	Conwed $\theta=90^\circ$	45	40	0	0
10-12	Conwed $\theta=45^\circ$	46	46	10	10
13-15	Conwed $\theta=30^\circ$	46.5	46	12.3	16.2
16-18	Conwed $\theta=0^\circ$	46	40	5.5	4.5
19-21	Tensar $\theta=90^\circ$	44	36	0	0
22-24	Tensar $\theta=45^\circ$	46	41	18.0	20.5
26-28	Tensar $\theta=30^\circ$	46	40	19.6	22.9
29-31	Tensar $\theta=0^\circ$	46	46	9.4	10.8

2) The reinforced soil and unreinforced granular soil sample yielded more or less the same internal friction angle. This means that the reinforcement does not seem to affect the shear mechanism of the natural soil (internal friction on plane inclined at ψ , see chapter 3) but only contributes an additional strength increase by adding a cohesive part. This apparent cohesion is due to the mobilized tensile strain in the reinforcement mobilized by the overall shear displacement as well as by the dilatancy of the soil.

Figures 6.18 through 6.21 show the shear force versus shear displacement relationships for crushed limestone aggregate reinforced with Conwed 5033. Comparing

the corresponding results of each test with those of coarse sand reinforced with the same reinforcement, a different behaviour was observed. For the sliding test the behaviour of crushed limestone composite, before the peak value, was similar to that of the coarse sand composite. After the peak value the crushed limestone composite showed a gradually decrease of shear force with further shearing while the coarse sand composite exhibited a distinct decline beyond the peak. A constant value in shear force was observed beyond that. When the geogrid was orientated at 45° and 30° , the crushed limestone aggregate showed a continuous increase in shear force with shear displacement. No distinct change in slope was observed while for coarse sand composite the slope change was quite obvious. Figure 6.21 shows the results of three tests when the geogrid (Conwed 5033) was placed perpendicular to the shear plane ($\theta=0^{\circ}$). No distinct peak value existed but the change of slope of shear force with displacement was pronounced in the range of 5 to 15 mm.

Figures 6.22 to 6.25 give the test results for crushed limestone aggregate reinforced with Tensar UX 1500. The behaviour of this composite was similar to that of crushed limestone aggregate reinforced with Conwed 5033. The normal stress versus shear stress relationships for crushed limestone aggregate are shown in Figs. 6.26 to 6.33. A linear relationship can be approximated. The shear strength parameters for reinforced crushed limestone aggregate can be represented by an "apparent cohesion, C_r " and an internal friction angle, which had about the same numerical value as that of unreinforced soil. Tables 6.4 and 6.5 present the summary of test results.

As shown from these figures and tables, a considerable shear strength increase was

observed in all tests of reinforced limestone aggregate compared with the corresponding results of reinforced coarse sand. The most likely reason for this increase is not only due

TABLE 6.4. TEST RESULTS FOR CRUSHED LIMESTONE AGGREGATE

Test Series	Geogrid & Orientation (θ)	Surcharge (kPa)	Shear Strength Characteristics	
			Peak Value (kN)	Residual Value (kN)
7	Conwed $\theta=90^\circ$	23	24.67	16.67
8		28	32.67	22.00
9		35	40.67	26.00
10	Conwed $\theta=45^\circ$	23	50.00	50.00
11		28	57.33	60.00
12		35	64.67	67.33
13	Conwed $\theta=30^\circ$	23	50.00	52.67
14		28	60.00	63.33
15		35	64.67	69.33
16	Conwed $\theta=0^\circ$	23	40.17	41.34
17		28	44.00	44.67
18		35	51.33	50.67
19	Tensar $\theta=90^\circ$	23	26.00	17.33
20		28	28.67	22.00
21		35	39.34	28.00
22	Tensar $\theta=45^\circ$	23	56.33	52.67
23		28	65.33	62.67
24		35	74.90	78.00
26	Tensar $\theta=30^\circ$	23	59.33	62.67
27		28	67.33	68.00
28		35	76.67	78.00
29	Tensar $\theta=0^\circ$	23	43.33	43.33
30		28	51.33	53.33
31		35	58.00	58.00

**TABLE 6.5. SHEAR STRENGTH PARAMETERS FOR REINFORCED
CRUSHED LIMESTONE AGGREGATE**

Test Series	Geogrid & Orientation (θ)	Shear Strength Parameters			
		Friction Angle ($^{\circ}$)		Cohesion (kPa)	
		Peak	Residual	Peak	Residual
7-9	Conwed $\theta=90^{\circ}$	54	37	0	0
10-12	Conwed $\theta=45^{\circ}$	54	48	17.6	25.8
13-15	Conwed $\theta=30^{\circ}$	54.5	50	20.0	27.90
16-18	Conwed $\theta=0^{\circ}$	54	48	7.7	15.1
19-21	Tensar $\theta=90^{\circ}$	54	38	0	0
22-24	Tensar $\theta=45^{\circ}$	54	51	21.0	21.5
26-28	Tensar $\theta=30^{\circ}$	55	50	27.5	30.4
29-31	Tensar $\theta=0^{\circ}$	54	50	11.3	18.0

to the higher inherent shear strength of the soil itself (internal friction angle of 54° compared to 46° for the sand) but also due to a much higher interlock of the larger soil particles with the apertures of the reinforcement.

6.2.2. Deformation Behaviour of Reinforced Soils

Figs. 6.34 to 6.41 present vertical displacement versus horizontal shear displacement relationships for coarse sand reinforced with Conwed 5033 and Tensar UX 1500. In all 34 tests a considerable increase in volume occurred on the shear plane as the geogrids were inclined at various orientations to the shear plane. This volume increase was due to soil dilatancy. It was also observed that orientation of the geogrid to the shear plane had a significant influence on the soil dilatancy whereas the magnitude of the surcharge load did not seem to have much effect on this volume change for the narrow

range of normal stress used in this study.

In reinforced soil, the movement of the soil particles during shear is restrained by the presence of the geogrid. Particles "locked" into the aperture will transfer the shear stress to the grid members by interlock and surface friction. Tensile strains and corresponding tensile forces are, therefore, mobilized in the reinforcement. Consequently, the soil dilatancy is governed by the extensibility of the geogrid and of course the mobilized tensile strain is dependent on the amount of soil dilatancy. The tensile strain mobilized in a reinforcement element can be directly related to the increase of shear strength of the reinforced soil. Furthermore, this mobilized tensile strain is dependent on the orientations of the geogrid with respect to the shear plane (chapter 3).

The dilatancy behaviour of reinforced coarse sand with Conwed 5033 and Tensar UX 1500 has shown that at a the geogrid orientation of 90° , i. e. in sliding tests, the volume change was similar in magnitude to that of natural sand. A very small contraction occurred at an initial shear displacement (2 to 3 mm). A linear increase in volume was observed after that. After a certain shear displacement (10 mm for sand reinforced with Conwed 5033, 12 mm for sand reinforced with Tensar UX 1500) no additional volume change would occur. The vertical displacement had also levelled off at this shear displacement. For geogrids inclined at other orientations the deformation (dilatancy) behaviour for reinforced coarse sand followed different patterns compared to unreinforced soil. A continuous increase in volume was observed with the continuing shear an all these tests.

The deformation (dilatancy) behaviour of limestone aggregate reinforced with

Conwed 5033 and Tensar UX 1500 were also monitored and are shown in Figs. 6.42 to 6.49. Comparing these with the corresponding behaviour of reinforced coarse sand, the limestone aggregate yielded much larger vertical displacement, especially when the geogrids were orientated at 45° and 30° to the normal to the shear plane. A linear relationship between vertical and horizontal displacement was observed in all these twelve tests. The possible reason for this phenomenon will be discussed in Chapter 8 of this thesis.

Fig. 6.1. Analogy between Prototype Shear Failure and Direct Shear Test

Fig. 6.2. Direct Shear Sliding Test on Conwed 5033

Fig. 6.3. Direct Shear Test with Conwed 5033 (Orientated at 45°)

Fig. 6.4. Direct Shear Test with Conwed 5033
(Orientated at 30°)

Fig. 6.5. Direct Shear Test with Conwed 5033
(Orientated at 0°)

Fig. 6.6. Direct Shear Sliding Test on Tensar UX1500

Fig. 6.7. Direct Shear Test with Tensar UX 1500
(Orientated at 45°)

Fig. 6.8. Direct Shear Test with Tensar UX 1500
(Orientated at 30°)

Fig. 6.9. Direct Shear Test with Tensar UX 1500
(Orientated at 0°)

Fig. 6.10. Strength Envelopes for Natural Sand
and Sliding Test (Conwed 5033 Grid)

Fig. 6.11. Strength Envelopes for Natural and Reinforced Sand
(Conwed 5033 at 45°)

Fig. 6.12. Strength Envelope for Natural and Reinforced Sand
(Conwed 5033 at 30°)

Fig. 6.13. Strength Envelope for Natural and Reinforced Sand
(Conwed 5033 at 0°)

Fig. 6.14. Strength Envelopes for Natural Sand and Sliding Test (Tensar UX 1500 Grid)

Fig. 6.15. Strength Envelopes for Natural and Reinforced Sand
(Tensar UX 1500 at 45°)

Fig. 6.16. Strength Envelope for Natural and Reinforced Sand
(Tensar UX 1500 at 30°)

Fig. 6.17. Strength Envelope for Natural and Reinforced Sand
(Tensor UX 1500 at 0°)

Fig. 6.18. Direct Shear Sliding Test on Conwed 5033

Fig. 6.19. Direct Shear Test with Conwed 5033
(Orientated at 45°)

Fig. 6.20. Direct Shear Test with Conwed 5033
(Orientated at 30°)

Fig. 6.21. Direct Shear Test with Conwed 5033
(Orientated at 0°)

Fig. 6.22. Direct Shear Sliding Test on Tensar UX1500

Fig. 6.23. Direct Shear Test with Tensar UX 1500
(Orientated at 45°)

Fig. 6.24. Direct Shear Test with Tensar UX 1500
(Orientated at 30°)

Fig. 6.25. Direct Shear Test with Tensar UX 1500
(Orientated at 0°)

Fig. 6.26. Strength Envelopes for Natural Aggregate
and Sliding Test (Conwed 5033 Grid)

Fig. 6.27. Strength Envelopes for Natural and Reinforced Aggregate
(Conwed 5033 at 45°)

Fig. 6.28. Strength Envelope for Natural and Reinforced Aggregate
(Conwed 5033 at 30°)

3

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Fig. 6.29. Strength Envelope for Natural and Reinforced Aggregate
(Conwed 5033 at 0°)

Fig. 6.30. Strength Envelopes for Natural Aggregate and Sliding Test
(Tensar UX 1500 Grid)

Fig. 6.31. Strength Envelopes for Natural and Reinforced Aggregate
(Tensar UX 1500 at 45°)

Fig. 6.32. Strength Envelope for Natural and Reinforced Aggregate
(Tensar UX 1500 at 30°)

Fig. 6.33. Strength Envelope for Natural and Reinforced Aggregate
(Tensar UX 1500 at 0^0)

Fig. 6.34. Dilation of Reinforced Coarse Sand During Sliding Test
(with Conwed 5033 Grid)

Fig. 6.35. Dilation of Reinforced Coarse Sand during Shear
(with Conwed 5033 at 45°)

Fig. 6.36. Dilation of Reinforced Coarse Sand during Shear
(with Conwed 5033 at 30°)

Fig. 6.37. Dilation of Reinforced Coarse Sand during Shear
(with Conwed 5033 at 0°)

Fig. 6.38. Dilation of Reinforced Coarse Sand during Sliding Test
(with Tensar UX 1500 Grid)

Fig. 6.39. Dilation of Reinforced Coarse Sand during Shear
(with Tensar UX 1500 at 45°)

Fig. 6.40. Dilation of Reinforced Coarse Sand during Shear
(with Tensar UX 1500 at 30°)

Fig. 6.41. Dilation of Reinforced Coarse Sand during Shear
(with Tensar UX 1500 at 0°)

Fig. 6.42. Dilation of Reinforced Aggregate During Sliding Test
(with Conwed 5033 Grid)

Fig. 6.43. Dilation of Reinforced Aggregate Sand during Shear
(with Conwed 5033 at 45°)

Fig. 6.44. Dilation of Reinforced Aggregate during Shear
(with Conwed 5033 at 30°)

Fig. 6.45. Dilation of Reinforced Aggregate during Shear
(with Conwed 5033 at 0°)

Fig. 6.46. Dilation of Reinforced Aggregate during Sliding Test
(with Tensar UX 1500 Grid)

Fig. 6.47. Dilation of Reinforced Aggregate during Shear
 (with Tensar UX 1500 at 45°)

Fig. 6.48. Dilation of Reinforced Aggregate during Shear
(with Tensar UX 1500 at 30°)

Fig. 6.49. Dilation of Reinforced Aggregate during Shear
(with Tensar UX 1500 at 0°)

CHAPTER 7

PULLOUT RESISTANCE OF GEOGRIDS

7.1. INTRODUCTION

Pullout tests have been commonly accepted as an effective method to evaluate the bond resistance of reinforced soil systems and the peak pullout force that can be generated in the reinforcement embedded in the soil. Typically a tensile force is applied directly to one end of the reinforcement and whereas the other end is embedded. A pullout force versus front end displacement of reinforcement can be obtained from a test (Ingold 1982, 1983, 1985; Mowafy 1986; Shang 1989). In order to have a comprehensive understanding of the soil reinforced with geogrid and to obtain a whole picture of the behaviour of reinforced soil structure (Fig. 1.2) a series of pullout tests were carried out in addition to extensive experiments on direct shear. The pullout tests being conducted in this investigation consisted of two parts, preliminary small and large pullout tests.

The preliminary pullout tests were carried out in a relative small test rig (1040x230x380 mm). The main objectives of these tests were to evaluate the four types of Conwed Stratagrids in terms of pullout resistance and to select one or two suitable geogrids for further investigation. The large pullout tests were conducted in a fairly large test tank which had a dimensions of 1550x900x1220 mm. The main objectives of the large pullout tests were to investigate the interaction mechanism between granular soils and geogrids by monitoring the relative displacements (or strains) in the geogrid, and to study the anchorage capacity of geogrid reinforced soil in field specified conditions.

7.2. PRELIMINARY PULLOUT TESTS

7.2.1. Test Program

Fifty one pullout tests were conducted in the preliminary stage, including six repeat tests. The variables in this experiment were soil types, geogrid types and number of surcharge loads. Three soils, i.e. fine quartz sand, coarse sand and crushed limestone aggregate, four Conwed geogrids, namely Conwed 9027, 5033, 3024, 3022, and Tensar UX 1500 and three different surcharges were used in this investigation. The three surcharge loads were applied corresponding to the following normal stress on the geogrid 14 kPa, 20 kPa and 25 kPa. The maximum surcharge load would correspond approximately to 1.5 m of backfill in the field. Table 7.1 summarizes this test program.

TABLE 7.1. EXPERIMENTAL PROGRAM FOR PULLOUT TESTS

Geogrid	Soil Type	No. of Surcharge Loads	Number of Tests
Conwed 9027 9027 (repeat)	3	3	9
	3	1	3
5033 5033 (repeat)	3	3	9
	2	1	3
3024	3	3	9
3022	3	3	9
Tensar UX 1500	3	3	9

7.2.2. Test Results

The preliminary pullout tests were recorded as pullout force versus the front horizontal displacement relationships. The following sections present the results of the fifty one tests for different composites.

Coarse Sand Reinforced with Conwed Stratagrids

Coarse sand is a commonly used backfill material in reinforced soil structure and its properties was well understood by several researches (Bauer 1987, 1988; Chan 1987; Mowafy 1986). This soil was also used in this preliminary pullout test program of this investigation. The test conditions and variables are listed in Table 7.2. Figures 7.1. to 7.5 present the pullout load versus horizontal displacement relationships for fifteen pullout tests on coarse sand with Conwed grids and Tensar UX 1500. All geogrids failed by pullout. The tests were terminated when the displacement of the first node of a grid had reached 25 mm. It was observed that the stronger geogrid, such as Conwed 9027 and Tensar UX 1500 had reached their maximum anchorage capacity at about 8-12 mm depending on the normal stresses, whereas for the other two Conwed, 3024 and 3022, the pullout resistance were still increasing beyond that displacement. The behaviour of Conwed 5033 fell into these two extremes. Where a peak value was observed, this was taken as the maximum pullout resistance (anchorage capacity), otherwise the value at 25 mm displacement was used.

TABLE 7.2. PULLOUT TEST PROGRAM FOR COARSE SAND

Soil Type	Geogrid Type	No. of Surcharge Intensities	No. of Tests
Coarse Sand	Conwed 9027	3	3
" "	5033	3	3
" "	3024	3	3
" "	3022	3	3
	Tensar UX 1500	3	3

Figures 7.1 to 7.5 also show that an increase in surcharge load leads to an increase of pullout force for all five geogrids. The reason is that the friction force are directly proportional to applied normal stresses and high normal stress causes high interlocking stress at the junctions of the stratagrids. The stratagrids 9027, 5033 and Tensar UX 1500 had reached peak pullout resistance in coarse sand. This is an indication of good interlock between the soil particles and the geogrid apertures. The Stratagrids 3024 and 3022 continue to increase their pullout resistance at the maximum horizontal displacement which means that a large displacement would be needed for these two geogrids to mobilize the maximum pullout or anchorage resistance. The difference between Tensar UX 1500, Conwed 9027, 5033 and 3024, 3022 is in their longitudinal stiffness (which can be defined as the slope of the pullout force versus front displacement relationship). The effect of stiffness on the pullout resistance will be discussed in Chapter 9 of this thesis. Table 7.3 summarizes the tests results for coarse sand with four different Conwed Stratagrids at three different surcharge loads.

**TABLE 7.3. PULLOUT TEST RESULTS FOR COARSE SAND
WITH CONWED STRATAGRIDS**

Geogrid	Normal Stress kPa	Pullout Resistance kN	Displacement at Failure mm
Conwed 9027	14.0	1.86	7.0
	20.0	2.87	19.0
	25.0	3.54	11.0
5033	14.0	2.07	14.0
	20.0	2.70	17.0
	25.0	3.32	21.0
3024	14.0	1.79	22.0
	20.0	2.14	25.0
	25.0	2.45	25.0
3022	14.0	1.89	24.0
	20.0	2.24	25.0
	25.0	2.59	25.0
Tensar UX 1500	14.0	2.24	8.4
	20.0	3.11	10.2
	25.0	3.62	12.0

Crushed Limestone Aggregate Reinforced with Conwed Stratagrids

Fifteen tests were conducted on crushed stone aggregate with Conwed stratagrids and Tensar UX 1500. Table 7.4. summarizes the test conditions.

TABLE 7.4. PULLOUT TEST PROGRAM FOR CRUSHED LIMESTONE

Soil Type	Geogrid Type	No. of Surcharge Intensities	No. of Tests
Aggregate	Conwed 9027	3	3
" "	5033	3	3
" "	3024	3	3
" "	3022	3	3
" "	Tensar UX 1500	3	3

Figures 7.6 to 7.10 show the pullout force versus horizontal displacement relationships for the meshes. Similar patterns as those of coarse sand were observed but this reinforced soil composites yielded a higher pullout resistance compared to the corresponding values of coarse sand composites. A summary of tests results are presented in Table 7.5.

Fine Quartz Sand Reinforced with Conwed Stratagrids

The fine quartz sand is a white flint fine granular soil having rounded to subrounded particles. The quartz sand used in this study had a density of 16 kN/m^3 , a internal friction angle of 26° and mean grain size D_{50} of 0.25 mm. The selection of this soil is to study the pullout resistance of Conwed Stratagrids and Tensar UX 1500 under the full range of particle sizes of granular soils, from the largest particles of aggregate to smallest particles of fine quartz sand. The conditions for this series of tests are summarized in Table 7.6.

The test results in the form of pullout force versus geogrid horizontal displacement are presented in Figs. 7.11 to 7. 15. Table 7.7 summarizes the test results.

TABLE 7.5 PULLOUT TEST RESULTS FOR CRUSHED STONE AGGREGATE WITH CONWED STRATAGRIDS

Geogrid	Normal Stress kPa	Pullout Resistance kN	Displacement at Failure mm
Conwed 9027	14.0	2.59	13.0
	20.0	3.26	24.0
	25.0	4.03	21.0
5033	14.0	2.57	21.0
	20.0	3.32	25.0
	25.0	3.99	25.0
3024	14.0	2.10	25.0
	20.0	2.28	25.0
	25.0	2.38	25.0
3022	14.0	2.14	25.0
	20.0	2.27	25.0
	25.0	2.36	25.0
Tensar UX 1500	14.0	2.84	12.9
	20.0	3.60	18.8
	25.0	4.57	24.2

TABLE 7.6 PULLOUT TEST PROGRAMS FOR FINE QUARTZ SAND

Soil Type	Geogrid Type	No. of Surcharge Intensities	No. of Tests
Fine Sand	Conwed 9027	3	3
" "	5033	3	3
" "	3024	3	3
" "	3022	3	3
" "	Tensar UX 1500	3	3

7.2.3. Pullout Resistance

In all fifty one tests the Conwed stratagrids and Tensar UX 1500 failed by pullout. The stronger stratagrids, 9027 and 5033 and Tensar UX 1500 reached their maximum

anchorage capacity (peak values were observed) whereas for the other two Conwed stratagrids, 3024 and 3022 the pullout resistance was not fully mobilized. The pullout force at 25 mm displacement, however, were taken as maximum pullout resistance.

TABLE 7.7. PULLOUT TEST RESULTS FOR FINE SAND WITH CONWED STRATAGRID

Geogrid	Normal Stress kPa	Pullout Resistance kN	Displacement at Failure mm
Conwed 9027	14.0	1.65	7.0
	20.0	2.59	9.0
	25.0	2.91	10.0
5033	14.0	1.47	9.0
	20.0	2.49	16.0
	25.0	3.05	24.0
3024	14.0	1.37	17.0
	20.0	2.07	25.0
	25.0	2.28	25.0
3022	14.0	1.09	10.0
	20.0	1.61	24.0
	25.0	2.28	25.0
Tensar UX 1500	14.0	1.84	6.8
	20.0	2.49	8.8
	25.0	2.96	9.2

The pullout resistance of geogrids is a function of soil properties, external surcharge and geogrid properties. A soil with larger particles will provide better friction and interlock giving a higher pullout resistance. Higher normal stresses (surcharge) will yield higher shear and interlock stresses as exhibited by higher pullout resistances. The pullout resistance was determined by plotting the peak pullout stress (maximum pullout force divided by the nominal grid area) against normal stress. A straight line relationship was obtained which can be expressed as:

$$r=na+b \quad (7.1)$$

where r = pullout resistance

n = normal stress on stratagrid

a = coefficient of interlock and friction, $a = \tan \alpha$, where α is the slope angle of the r versus n plot

b = intercept with r -axis.

The coefficient of interlock, a , and intercept, b , can be considered as constant parameters for a given composite and are obtained experimentally in order to estimate pullout resistance from equation (7.1). Table 7.8 lists these parameters for the tested soil-reinforcement system together with the coefficient of goodness of fit. These parameters were calculated based on the three tests, each with different normal stresses. The graphical presentation of these results are shown in Figures 7.16 through 7.18.

TABLE 7-8 PARAMETERS FOR PULLOUT RESISTANCE

Geogrid	Soil Type	Angle of Interlock α°	Coefficient of interlock, a	Intercept, b kPa	Coefficient of Fit
Conwed 9027	Crushed Limestone Aggregate	19.0	0.345	1.88	0.991
5033		18.8	0.340	1.98	1.000
3024		4.0	0.070	4.76	0.987
3022		3.4	0.059	4.99	0.998
Tensar 1500		22.2	0.409	1.57	0.993
Conwed 9027	Coarse Sand	22.0	0.404	0	0.996
5033		16.6	0.298	25.7	0.998
3024		9.2	0.162	2.58	1.000
3022		9.8	0.172	2.70	0.997
Tensar 1500		22.3	0.410	1.35	0.999
Conwed 9027	Fine Sand	20.8	0.305	0.247	0.961
5033		17.6	0.380	0	0.987
3024		12.8	0.227	0.732	0.939
3022		9.8	0.293	0	0.985
Tensar 1500		14.9	0.267	1.11	0.994

7.2.4. Observations

Based on the results of these preliminary tests, the following observation are made.

- 1). The pullout resistance of a geogrids depends on the ratio of the aperture of the grid to particle size diameter. A ratio of 2 to 3 seems to provide the best soil/grid interlock (Which were the cases of coarse sand and crushed limestone aggregate reinforced with Conwed 9027, 5033, and Tensar UX 1500)
- 2). Conwed stratagrids 9027 and 5033 showed similar pullout resistance in the three granular soils. The magnitude of the pullout resistance depends on the normal stress acting on the stratagrid. Therefore, stratagrid 9027 and 5033 was recommended as the reinforcement for further study in large pullout tests and direct shear box tests for its best performance in pullout tests. Tensar UX 1500 performed well in small pullout tests and it was also recommended to be used in further study.
- 3). Two stratagrids 3024 and 3022, exhibited similar pullout behaviour, but the maximum resistance were considerably less than those of the other two grids (9027, 5033 and Tensar UX 1500). Therefore these two geogrids were eliminated from further testing.
- 4). The pullout resistance of a given stratagrid and soil can be estimated from Equation (7.1). The two parameters, a and b, can be considered constant for a given composite and these parameters must be determined experimentally.

7.3. LARGE PULLOUT TESTS

7.3.1. Modification of Large Pullout Test Tank

In light of the scale effect mentioned in Chapter 2 and the results of the preliminary tests, a large scale pullout test tank was modified in the second stage of pullout investigation of this research. Figure 4.20 shows a schematic diagram of this test apparatus which has a dimension of 1550x900x1220 mm. The major difference between the preliminary and large pullout tests were not only the scale difference but also different in the main purpose. The preliminary pullout tests were to evaluate the five geogrids in terms of pullout resistance and select one or two geogrids for further research while the large pullout tests were to study the interaction mechanism of the geogrid reinforced soil in pullout in cooperation with the test results from the direct shear. The large test facility allowed the soil to be compacted to the field conditions, i.e. field dry density and moisture content. The large test apparatus were also permitted to use a large size of representative sample of reinforcement and to be placed in a field case, for example, 0.8 - 1 m width. Thus, the large pullout tests were aimed to determine the pullout resistance or anchorage capacity of soil - geogrid systems, the soil-geogrid interface strength and strain and tensile force distribution in the geogrid. In order to do so four LVDTs were used with piano wire attached to the nodes of the geogrid. The deformations (extension) of the geogrid during pullout, therefore, were monitored. The average strain between nodes in the geogrid was calculated. This information was used to verify the interaction model for pullout presented in Chapter 3.

7.3.2. Test Program

Fifteen large pullout tests were carried out in this investigation. The main variables being studied were a) soil properties; two granular soils, a coarse sand and a crushed limestone aggregate were used in the pullout test program. These two soils have been used extensively in previous research (Mowafy 1986; Shang 1989) and in the direct shear tests of this investigation. b) geogrid properties; three types of geogrids, Tensar UX 1500, Conwed 9017 and 5033, were tested in the large pullout rig based on the recommendation of the results of fifty one preliminary pullout tests. c) the normal load; it has been shown in the preliminary pullout test that the pullout force increased with the increase of normal load. Three surcharge intensities were used in this study. They were 10, 20 and 30 kN/m². d) the embedded length of geogrid; since the maximum pullout resistance or anchorage capacity is a function of mobilized tensile force in the geogrid and this force is dependent on the embedded length of the geogrid. This effect was also investigated in this study. Table 7.9 summarizes the large pullout test program and conditions.

7.3.3. Test Results

All geogrids in fourteen tests failed by pullout. One geogrid failed by tensile breakage in one of the tests. Figures 7.19 through 7.33 show the results of fifteen large pullout tests. The test results were in the same form of pullout force versus front end horizontal displacement relationship as those in the preliminary tests. The test results show a general pattern of increase of pullout force with the continuing increase of horizontal displacement except those in test 1, 2, 3, 8, and 9. In test 1, because of tensile breakage of the geogrid, a sudden drop of the pullout force was observed after the failure.

TABLE 7.9 LARGE PULLOUT TEST PROGRAM

No. of Test	Geogrids	Soil Type	Normal Loads(kPa)	Embedded Length (mm)
Test 1	Conwed 5033	Coarse Sand	10	1300
Test 2	Conwed 9027	" "	10	1300
Test 3	" "	" "	20	1300
Test 4	" "	" "	30	1300
Test 5	Conwed 5033	" "	10	1300
Test 6	" "	" "	20	1300
Test 7	" "	" "	30	1300
Test 8	Tensar UX 1500	" "	10	1300
Test 9	" "	" "	20	1300
Test 10	" "	" "	30	1300
Test 11	Conwed 5033	" "	30 (repeat)	1300
Test 12	Conwed 9027	Aggregate	10	1300
Test 13	" "	Coarse sand	10	1300
Test 14	" "	" "	10	1000
Test 15	" "	" "	10	800

In tests 2 and 3 plastic flow (constant pullout force with the further increase of displacement) occurred at 40 mm and 60 mm of horizontal displacement, respectively. In

test 8 and 9 peak values of pullout force were obtained at 58 mm and 68 mm, respectively. Slight decreases of pullout force were observed after peaks. Table 7.10 gives the summary of the test results.

TABLE 7.10. LARGE PULLOUT TEST RESULTS

Geogrid	Normal Stress kPa	Pullout Resistance kN	Displacement at Failure mm
Conwed 5033	10.0	18.09	87.5
	20.0	25.15	120.0
	30.0	26.91	120.0
Conwed 9027	10.0	16.84	45.0
	20.0	28.46	95.0
	30.0	37.50	120.0
Tensar UX 1500	10.0	14.56	56.0
	20.0	24.49	74.5
	30.0	36.62	120.0

The pullout resistance versus normal stress was also plotted as shown in Fig. 7.34. An approximately straight line relationship can be fitted using the curve fitting method. An angle of interlock, α , or a coefficient of interlock, a , and intercept constant, b , were obtained from the plot. Table 7.11 summarizes the results.

TABLE 7-11 PARAMETERS FOR PULLOUT RESISTANCE

Geogrid	Soil Type	Angle of Interlock α°	Coefficient of interlock, a	Intercept, b kPa	Coefficient of Fit
Conwed 9027	Coarse Sand	23.32	0.431	2.893	0.997
Conwed 5033	Coarse Sand	16.85	0.303	4.231	0.945
Tensar 1500	Coarse Sand	42.58	0.919	1.32	0.998

Figures 7.35 to 7.48 show the test results of monitoring the nodal displacements in the geogrid during pullout. The results are presented in the form of nodes displacement

versus the front end horizontal displacement of the geogrid. This information will be used to develop a relationship between the progressive development of the displacement in the geogrid with the pullout force and mobilized tensile force distribution in the geogrid. The figures show some patterns of progressive development of deformation in the geogrid. It was observed that at a certain value of front end displacement the first node started to move, then gradually transferred to the second node, and the third node. The figures also show that a great variation for nodes displacements occurred since the development of relative displacement (or strain) in the geogrid is a function of soil properties , geogrid properties , normal stresses and embedded length of the geogrid. All these aspects need further analysis which will be presented in Chapter 9 of this thesis.

Fig. 7.1. Pullout Test Results on Coarse Sand with Conwed 9027

Fig. 7.2. Pullout Test Results on Coarse Sand with Conwed 5033

Fig. 7.3. Pullout Test Results on Coarse Sand with Conwed 3024

Fig. 7.4. Pullout Test Results on Coarse Sand with Conwed 3022

Fig. 7.5. Pullout Test Results on Coarse Sand with Tensar UX 1500

Fig. 7.6. Pullout Test Results on Coarse Sand with Conwed 9027

Fig. 7.7. Pullout Test Results on Aggregate with Conwed 5033

Fig. 7.8. Pullout Test Results on Aggregate with Conwed 3024

Fig. 7.9. Pullout Test Results on Aggregate with Conwed 3022

Fig. 7.10 Pull out Test Results on Aggregate with Tensar UX 1500

Fig. 7.11. Pullout Test Results on Fine Sand with Conwed 9027

Fig. 7.12. Pullout Test Results on Fine Sand with Conwed 5033

Fig. 7.13. Pullout Test Results on Fine Sand with Conwed 3024

Fig. 7.14. Pullout Test Results on Fine Sand with Conwed 3022

Fig. 7.15. Pullout Test Results on Fine Sand with Tensar UX 1500

Fig. 7.16. Pullout Resistance of Coarse Sand with Geogrids

Fig. 7.17. Pullout Resistance of Aggregate with Geogrids

Fig. 7.18. Pullout Resistance of Fine Sand with Geogrids

Fig. 7.19. Large Pullout Test Result for Test 1

Fig. 7.20. Large Pullout Test Result for Test 2

Fig. 7.21. Large Pullout Test Result for Test 3

Fig. 7.22. Large Pullout Test Result for Test 4

Fig. 7.23. Large Pullout Test Result for Test 5

Fig. 7.24. Large Pullout Test Result for Test 6

Fig. 7.25. Large Pullout Test Result for Test 7

Fig. 7.26. Large Pullout Test Result for Test 8

Fig. 7.27. Large Pullout Test Result for Test 9

Fig. 7.28. Large Pullout Test Result for Test 10

Fig. 7.29. Large Pullout Test Result for Test 11

Fig. 7.30. Large Pullout Test Result for Test 12

Fig. 7.31. Large Pullout Test Result for Test 13

Fig. 7.32. Large Pullout Test Result for Test 14

Fig. 7.33. Large Pullout Test Result for Test 15

Fig. 7.34. Pullout Resistance of Coarse Sand with Geogrids in Large Pullout Tests

Fig. 7.35. Development of Displacement in Geogrid in Test 2

Fig. 7.36. Development of Displacement in Geogrid in Test 3

Fig. 7.37. Development of Displacement in Geogrid in Test 4

Fig. 7.38. Development of Displacement in Geogrid in Test 5

Fig. 7.39. Development of Displacement in Geogrid in Test 6

Fig. 7.40. Development of Displacement in Geogrid in Test 7

Fig. 7.41. Development of Displacement in Geogrid in Test 8

Fig. 7.42. Development of Displacement in Geogrid in Test 9

Fig. 7.43. Development of Displacement in Geogrid in Test 10

Fig. 7.44. Development of Displacement in Geogrid in Test 11

Fig. 7.45. Development of Displacement in Geogrid in Test 12

Fig. 7.46. Development of Displacement in Geogrid in Test 13

Fig. 7.47. Development of Displacement in Geogrid in Test 14

Fig. 7.48. Development of Displacement in Geogrid in Test 15

CHAPTER 8

DISCUSSION AND ANALYSIS OF THE RESULTS FROM DIRECT SHEAR TESTS

8.1. INTRODUCTION

A direct shear test is a standard method to measure the plane strain strength parameters for soils. It remains popular in practice although it is a relatively crude test and it has been criticized by several people (Potts, 1987), but the results from a direct shear test are comparable to those from fairly complex tests (triaxial or simple shear). It is a new practice to test for reinforced soil strength parameters in direct shear. There is no standard test apparatus or procedure to follow. The size of shear box varies and the rates of shear force application are also not standardized. The objective of this research study was to investigate the influence of geogrids on the shear strength and deformation behaviour of granular soils. In order to do so, a study of the strength and deformation (dilatancy) behaviour of unreinforced soil was carried out first. Secondly, a proper method to interpret the test results from a direct shear test was established and finally, an investigation into the effect of the geogrid on the modification of strength and deformation (dilatancy) behaviour of natural soils was conducted. This chapter will discuss the result from direct shear tests on unreinforced soils in terms of strength and deformation (dilatancy). The effect of geogrids on the shear strength and deformations (dilatancy) behaviour of reinforced soil will also be dealt with. In addition, the load transfer mechanism between soil and geogrid is discussed in this chapter. A comparison between prediction using the proposed model and test results will be presented. Some other factors such as soil dilatancy are also discussed here.

8.2. INTERPRETATION OF RESULTS FROM DIRECT SHEAR TEST . STRENGTH AND DEFORMATION

The conventional analysis of a reinforced soil structure (e. g. retaining wall) is performed using either a force equilibrium method (Coulomb) or stress equilibrium (Rankine). The critical potential failure surface found from the force equilibrium (Coulomb's method) is identical to that obtained from the maximum mobilized resistance (Rankine's method). These methods are still valid for the analyses of reinforced soil structures with metallic reinforcement. However, since these two methods make no reference to the deformation or strain in the soil, the methods will need some modification in the application of extensible geosynthetic reinforcement. Because the mobilized soil shear resistance and reinforcement force are both dependent on the deformation developed in the soil, deformation behaviour is equally important to the understanding of the fundamental mechanism of reinforced soil.

8.2.1. Plane Strain and Direct Shear Angle of Friction

Two angles of friction are discussed by Jewell and Wroth (1987) for the plane strain deformation condition of granular soil under shear. They are the plane strain angle of friction, ϕ_{ps} , and the direct shear angle of friction, ϕ_d . The physical significance of these two angles is that ϕ_{ps} defines the Mohr - Coulomb failure envelope for the soil and ϕ_d represents the shear resistance mobilized on a plane in the soil along which there is no linear incremental strain (a direction of zero extension) as shown in Fig. 8.1 (Jewell and Wroth 1987; Jewell 1989). The relationship between these two angles can be obtained either by examining a smooth retaining wall rotated outwards about the wall toe (Fig. 8.2,

Jewel! 1989) or by using Mohr's circle of stress and incremental strain (Fig. 8.1).

$$\sin\phi_{ps} = \frac{\tan\phi_{ds}}{\cos\psi(1+\sin\phi\tan\phi_{ds})} \quad (8.1)$$

Figure 8.3 shows the relationship between ϕ_{ps} and ϕ_{ds} for different values of dilation angle ψ obtained from expression (8.1) (Palmeira 1987). It is seen that ϕ_{ps} is always greater or equal to ϕ_{ds} depending on the value of ψ . For ψ values between 0° to 30° , the two values are close to equal. Also when ψ_{max} equals ϕ_{ds} , $\phi_{ps} = \phi_{ds}$. It needs to be emphasized that Equation (8.1) is based on the assumption of coaxiality of stress and strain increment directions. In this case ϕ_{ps} would be equal or close to ϕ_{ds} . This problem can only be resolved by carrying out experiments where the direction of principal stress and strain increments can be measured. However, according to Palmeira (1987) those experimental results, as shown from a literature survey, have been conflicting. In some cases, for the same type of equipment, coaxiality was observed by some but not by other researches. At present, this ambiguity is still unsolved.

8.2.2. Interpretation of Direct Shear Test Results for Unreinforced Soil

1. Conventional Analysis - Maximum Obliquity Criterion

The conventional method to interpret the results from direct shear tests is to plot the peak shear stress versus normal stress and to calculate the direct shear angle of friction by Equation (5.1).

$$\tan\phi_{ds} = \tau_{yx}/\sigma_{yy} \quad (5.1)$$

This method was used as a preliminary analysis of the test results from direct shear tests of natural coarse sand and crushed limestone aggregate (Chapter 5). This

method is still used in engineering practice because the value of the friction angle calculated from direct shear tests by the maximum obliquity criterion is close to the one obtained by other plane strain devices (such as a biaxial test device).

2. Plane Strain Angle of Friction

An alternate method, suggested by Jewell (1989), to interpret the results from direct shear tests is to determine the plane strain angle of friction by Equation (8.1), using the information from the direct shear angle of friction from Equation (5.1) and the angle of dilation from Equation (3.34).

$$\Delta y / \Delta x = \tan \psi \quad (3.34)$$

This analysis, according to Jewell (1989), is sensitive to the angle of dilation, and the angle of dilation obtained from the boundary measurements of soil deformation may be underestimated. Therefore, the drawback of this analysis is to overestimate the plane strain angle of friction (Jewell 1989).

3. Flow Rule Analysis

Taylor's (1948) flow rule, as shown by Equation (3.19), successfully correlates the strength - deformation characteristics of granular soil in direct shear. This relationship can also be used to interpret the results from direct shear tests. Equation (3.19) can be rewritten as:

$$\tan \phi_{ds} - \tan \psi = \sin \phi_{cv} \quad (8.2)$$

The direct shear angle of friction can be calculated from the boundary deformation measurement by using this equation.

A simple expression for the flow rule of granular soils has been proposed by Bolton (1986) as

$$\phi_{ps} - 0.8\psi = \phi_{cv} \quad (8.3)$$

Using the flow rule for soil, the plane strain and the direct shear angles of friction can be determined. The only requirements are the critical state angle of friction and the measured angle of dilation. The critical state angle of friction as suggested by Bolton (1986) can be estimated with reasonable accuracy for granular soil from soil mineralogy, or from the slope angle of a loosely tipped heap of the dry soil subject to the excavation at the toe of the slope. The critical state angle of friction for coarse sand was taken as 38° , according to Bolton's (1986) suggestion. The critical state angle for crushed limestone aggregate was taken as 40° .

The peak angle of friction and residual angle of friction are all calculated and summarized in Table 8.1.

TABLE 8.1. SHEAR STRENGTH PARAMETERS FOR TESTED SOILS

Test No.	Normal Stress (kPa)	Dilation Angle ($^\circ$)		Friction Angle ($^\circ$)	
		Peak	Residual	Peak	Residual
1	23	19.0	6.4	43.8	36.2
2	28	16.8	6.4	42.5	36.2
3	35	16.8	6.0	42.5	35.8
4	23	12.5	5.7	46.7	36.6
5	28	25.1	7.0	46.9	37.5
6	35	29.6	13.8	50.5	41.6

8.2.3. Interpretation of Results from Reinforced Soil in Direct Shear Tests

1. Conventional Analysis

The same approach, as used to analyze the test results from unreinforced soil, can be applied to interpret the experimental results of reinforced soil tested in direct shear. A "peak" shear stress versus normal stress can be plotted from a series of tests (at least 3) under identical test conditions as for the natural soils. An approximate linear relationship using a curve fitting method will yield two parameters, a direct shear angle of friction ϕ_a and an intercept C_r , so called " apparent cohesion " (Schlosser and Vidal 1969). The direct shear angle of friction is more or less the same as that of unreinforced soil, while the intercept C_r represents the shear strength increase due to the reinforcement. This technique was used as a preliminary analysis in Chapter 6. As explained earlier for reinforced soil there was no distinct "peak" value observed in the shear force versus shear displacement relationship. The "peak" value was taken at the corresponding displacement of the unreinforced soil's peak value.

2. Flow Rule Analysis

The conventional analysis makes no reference to soil deformation or soil strain. This concept, however, is important to the calculation of the mobilized tensile force in the reinforcement (Eqns. 3.33, 3.35). The tensile force, in turn, will contribute to an increase of shear strength C_r (Equation 3.30).

Following Jewell (1980) and Taylor (1948) the flow rule analysis is used in the interpretation of the reinforced soil.

Equations (3.17) and (3.18) can be rewritten as

$$P\Delta Y + S\Delta X = \mu^* P\Delta X \quad (8.4)$$

where P, S = normal load and shear load, respectively

$\Delta X, \Delta Y$ = horizontal displacement and vertical displacement increments, respectively

μ^* = frictional constant

The right hand side of the Equation implies that an equivalent frictional constant is assumed for geogrid reinforced soil system and the net work input is assumed to be dissipated by this equivalent frictional mechanism.

The flow rule analysis can be combined with the static model proposed in Chapter 3 of this thesis to analysis the test results from direct shear. From Equations (3.24) and (3.25) it is noticed that the normal force applied on the shear plane is modified by the reinforcement by an amount of

$$\Delta P = P_r \cos(\theta + \Delta\theta) \quad (8.5)$$

and the shear force mobilized on the shear plane is decreased by

$$\Delta S = -P_r \sin(\theta + \Delta\theta) \quad (8.6)$$

where $\Delta P, \Delta S$ = incremental normal and shear forces on the shear plane due to reinforcement, respectively

P_r = mobilized tensile force in the reinforcement

$\theta, \Delta\theta$ = original and increment of the reinforcement orientation with respect to the normal of the shear plane, respectively.

Substituting Equations (8.5.) and (8.6) into (8.4) and rearranging, the following expression can be obtained

$$\frac{S}{P} = \left(\mu - \frac{\Delta Y}{\Delta X} \right) \left[1 + \frac{P_r}{P} \cos(\theta + \Delta\theta) \right] + \frac{P_r}{P} \sin(\theta + \Delta\theta) = P_1 + P_2 \quad (8.7)$$

where

$$P_1 = \left(\mu - \frac{\Delta Y}{\Delta X} \right) \left[1 + \frac{P_r}{P} \cos(\theta + \Delta\theta) \right],$$

$$P_2 = \frac{P_r}{P} \sin(\theta + \Delta\theta)$$

P_r can be calculated approximately by Equation (3.39), where μ was taken as the unreinforced soil friction constant. Figures 8.4 through 8.11 show the comparison between the prediction from the flow rule analysis (Equation 8.7) and the experimental data from direct shear tests of coarse sand reinforced with Tensar UX 1500 and Conwed 5033 at different orientations, 0° , 30° , 45° and 90° . Figures 8.12 to 8.19 show the comparison of this analysis with test results for the reinforced crushed limestone aggregate under the same conditions of reinforcement type and orientation. The comparison reveals that the model, as given by Equation (8.7), provides a promising tool for the correlation of mobilized tensile force in the reinforcement, soil dilatancy angle, reinforcement orientation and mobilized shear strength of the soil. From a closer look at these figures it can be seen that the predictions from the model gives a good correlation with the experimental results when the geogrid is orientated at 30° and 45° . A slight overestimation of the shear force was found for most cases and the behaviour given by the model's prediction yielded slightly higher initial tangent moduli. The model did not give a very good correlation when the geogrid was orientated at 90° , especially in the case of

reinforced crushed limestone aggregate when using post-peak values. In the case of sand reinforced with Conwed 5033 the prediction shows some fluctuation and in the case of sand reinforced with Tensar UX 1500 the prediction underestimates the test results. The prediction for both sand and crushed limestone aggregate at an orientation of 0° correlates quite well with the experimental results.

8.3. STRAIN IN SOIL AND IN THE REINFORCEMENT

8.3.1. Strains Mobilized in the Soil

The relationship of mobilized shear resistance and plane strain deformation pattern in granular soil has been given by Stroud as illustrated by Jewell (1990) with experimental data from simple shear tests (Figure 8.20). The pattern shows that as the sheared granular soil approaches the critical state, at an initially rapid increase in stress ratio, little volume change and relative small strain will occur. Only after the peak shearing resistance has been mobilized does the soil begin to shear and dilate vigorously, thereby increasing the specific volume, but with only a gradual change in shearing resistance.

As stated in Chapter 3, the strain (deformation) in the soil during shearing can be divided into elastic strain (recoverable) and plastic strain (irrecoverable). It has been shown that the elastic strain is very small and can be neglected (Stroud 1971; Jewell 1990), the observed deformation is therefore essentially plastic. Plastic deformation is described by an angle of dilation ψ (Equation 3.15, 3.21) which can be rewritten as:

$$\sin \psi = \frac{dv^p}{d\gamma^p} = \frac{\frac{\Delta y}{\Delta x}}{\sqrt{1 + \left(\frac{\Delta y}{\Delta x}\right)^2}} \quad (8.6)$$

The main source of deformation and plastic strain is in particle movement and reorientation of soil particles under increasing shear stress. It is only after a relatively high proportion of peak shearing resistance has been mobilized ($\phi_m' > \phi_{cr}$) that these particle reorientations develop and thereby cause the significant plastic tensile strain necessary to mobilize the tensile force in the reinforcement (Jewell 1990).

The relation between mobilized shearing resistance and tensile strain is crucial for the analysis of reinforced soil, since the tensile soil strain provides the link between the soil and the reinforcement deformation. The study conducted by Jewell (1990) indicated that shearing resistance in compacted granular soil is mobilized rapidly and at a small magnitude of tensile strain. For typical compacted granular fills used in reinforced soil walls and steep slopes, a significant proportion of the available shearing resistance will have been mobilized by a cumulative tensile strain in the order of 1 percent or less, although the peak angle of friction might typically be mobilized at about 3 percent to 5 percent of cumulative tensile strain.

Eight special tests were conducted to attempt to measure the deformation or strain pattern in the soil for unreinforced and reinforced soil in order to have an understanding of the modification of the deformation pattern of the natural soil by the reinforcement. Figure 8.21 shows a photo of one of these tests. The side panel of the shear box was replaced with stiff plexiglass panels. The displacement of embedded markers was recorded

during shear as explained earlier. The bright vertical line are "parachute cords" to highlight the horizontal shear deformation. Figure 8.23 shows a view of the deformed shape of the sheared soil. In this figure the white markers are embedded steel pins to trace the movement of the soil during shear. The results of the tests are represented as displacement vectors as shown in Figs. 8.23 and 8.24. The average strain in the soil between two markers can then be calculated by the following equations:

$$\epsilon_x = \delta U_x / \Delta x \quad (8.9)$$

$$\epsilon_y = \delta U_y / \Delta y \quad (8.10)$$

$$\gamma_{xy} = \delta U_x / \Delta y + \delta U_y / \Delta x \quad (8.11)$$

A strain at the middle of four conjunctural markers can be averaged. Table 8.2 shows an example of the calculated strain in the soil for crushed limestone aggregate reinforced with Tensar UX 1500 orientated at 0° to the normal of the shear plane. The rest of the test results are presented in Appendix B of this thesis. The calculated strains show that

TABLE 8.2. MOBILIZED STRAIN IN THE SOIL IN A DIRECT SHEAR TEST

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_y	1	0.00625	0.00625	0.00125	0.01875	0.00125	0.00625	0.00625	0.0000
		-0.0060	-0.0050	-0.0050	-0.0025	-0.0040	-0.0025	-0.0050	-0.0075
		-0.0050	-0.0050	-0.0100	-0.0100	-0.0120	-0.0150	-0.0200	-0.005
ϵ_x	2	0.00167	0.0025	0.0075	0.0000	0.0100	-0.0025	0.0100	0.00167
		0.00333	0.0025	0.0025	0.0050	0.0150	0.0050	0.0000	0.0000
		0.00333	0.0025	0.0050	0.0050	0.0050	0.0075	0.0000	0.0000
γ_{xy}	3	0.00292	0.00125	-0.0204	-0.0175	-0.0067	-0.0254	-0.0142	-0.0129
		0.00833	0.02250	-0.0125	-0.0175	-0.003	0.0075	0.01500	-0.0016
		0.00333	0.01750	-0.0500	-0.015	0.0000	0.0025	-0.010	0.0050

negative vertical strains were observed in all tests. It indicated that the soil dilated during shearing. The horizontal strain and shear strain, however, did not show any consistent pattern.

The strain in the soil in the direction of the reinforcement can be calculated by the following Equation (Juran and Chen 1988C) assuming that the shear plane is a plane of zero extension (Fig.8.25).

$$d\epsilon_p = \tan v \left[1 - \frac{\cos(\theta - v) \sin \theta}{\sin v} \right] d\gamma_{xy} \quad (8.12)$$

where $d\epsilon_p$ = the strain in the soil at the plane inclined at an angle of θ to the normal of the shear plane,

v = dilation angle,

γ_{xy} = shear strain.

Figures 8.26 through 8. 29 show the shear force versus shear displacement relationships for reinforced soil together with the developed tensile strain in the direction of the reinforcement.

8.3.2. Strain Mobilized in the Reinforcement

The strain mobilized in the reinforcement during shearing can also be calculated by Equation (3.33) proposed in Chapter 3. Figures 8.30 to 8.33 show the presentations for reinforced soils together with the calculated tensile strain in the reinforcement similar to those given in Figs 8.26 to 8.29. The measurement of the mobilized tensile strains in the reinforcement during shear was attempted in six direct shear tests. Pianowires fed through plastic tubes were attached at the nodes of the reinforcement at various locations

and connected to LVDTs. The LVDTs were mounted outside of the shear box on a fixed steel beam (Figure 8.34).

8.3.3. Strain Compatibility

If perfect adherence between the soil and reinforcement would exist then the strain increment in the reinforcement should be equal to the extension strain increment in the soil in the same direction. A comparison of Figs. 8.26 through 8.29 and Figs. 8.30 through 8.33 show that this assumption is not true. This means then that a certain relative displacement between soil and reinforcement takes place before the tensile strain is developed in the reinforcement. This also implies that the strain compatibility condition for the soil and reinforcement should be used with caution. Generally speaking, the difference between the two displacements is relatively small. Therefore, strain compatibility can be assumed as a first approximate analysis for a reinforced soil structure.

8.3.4. Deformation Pattern of the Reinforcement

Shewbridge et al. (1989, 1990) published some results from direct shear tests. They obtained a deformation pattern of the shear zone as shown in Figure 8.35. They proposed an exponential decay function to describe the deformed reinforcement - soil shape as presented by Equation (2.14):

$$X_1 = B - Be^{-b|X_2|} \quad (8.13)$$

X_1 is the axis parallel to the direction of shear, X_2 is the axis perpendicular to the direction of shear with the origin at the centre of the shear zone, B is equal to one - half of the externally applied shear displacement, and b is a deformation decay constant.

Figure 2.20 gave a schematic diagram showing the geometry of the deformed reinforcement. The slope of the deformed shape of the reinforcement at the origin is given by the deformation decay constant b , multiplied by one-half of the externally applied shear displacement B . Thus, b is a measure of the thickness of the shear zone and of the distribution and magnitude of shear strain in this zone. For specimens with relatively thin shear zones, which contain large shear strains, the value of b is large. For samples with large shear zones containing smaller shear strains, the value of b is small. Based on the experimental results they concluded that the shear zone increases with increasing reinforcement concentration, reinforcement stiffness and soil-reinforcement bond strength.

Similar curvilinear symmetric deformed soil-reinforcement patterns were also observed in this investigation. Flexible cords and steel markers were used to monitor the deformation pattern during shearing. Figures 8.36 through 8.39 show some examples of the measured deformation pattern where the reinforcement was inclined at different orientations. The mapped shape of the deformed soil-reinforcement was digitized and the value of b was determined by a curve fitting method. Figure 8.40 shows a relationship between the measured decay constant b and the reinforcement orientation θ . It can be seen that the deformation constant b is in reality not a constant but depends on the geogrid orientation with regard to the shear plane. From the experimental investigation it was found that b varies from 0.012 ($\theta=30^\circ$) to 0.062 ($\theta=90^\circ$).

4

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

8.4. EFFECT OF REINFORCEMENT ORIENTATION ON SHEAR STRENGTH

8.4.1. Shear Strength Increase in Reinforced Soil

The increase in shear strength of a reinforced composite material using tensile reinforcement can be explained in several ways. Schlosser and Vidal (1969) proposed to use an "apparent cohesion" concept given by the tensile strength of the reinforcement due to the interaction between the reinforcement and soil. The second concept (Bassett and Last 1978) considered that the reinforcement provide an anisotropic restraint to the soil deformation in the direction of the reinforcement. This leads to a rotation of the principal strain and stress directions. Another concept was proposed by Gray (1983) followed by Jewell (1980, 1989) it denoted the additional change of shear strength as ΔS or τ_{EXT} .

The strength increase due to the concept of "apparent cohesion" is probably most logical as it can be explained by the Mohr circle theorem shown in Fig. 8.41. The intercept C'_R is the apparent cohesion generated by the reinforcement, and σ_{1R} is the increase in major principal stress at failure. The friction angle of the reinforced granular soil can be taken as that for the unreinforced soil, which is a reasonable assumption (Mitchell 1987; Schlosser and Long 1972). For reinforcement having a tensile breaking resistance R_T , and a vertical spacing between horizontal layers of reinforcements S_v , referring to the geometry of Fig. 8.41 leads to

$$C'_R = \frac{R_T \sqrt{K_p}}{2S_v} \quad (8.14)$$

$$\text{where} \quad K_p = \tan^2(45 + \phi/2) \quad (8.15)$$

Another approach to account for the increase in effective confinement was given by Yang

(1972) and it is shown by the Mohr circle in Fig. 8.42. The apparent increase of $\Delta\sigma_{3R}$ in minor effective confining pressure at failure is given by

$$\Delta\sigma_{3R} = R_T/S_v \quad (8.16)$$

The equation of the failure envelope is then

$$\Delta\sigma_{1R} = (\sigma_{3R} + R_T/S_v)K_p \quad (8.17)$$

The approach used by Gray (1983) and Jewell (1980, 1989) were presented by Equations (2.4) and (2.7), respectively. The approach used in this thesis was illustrated in Chapter 3. It is a modification of Gray and Jewell' shear strength increase approach and Schlosser's apparent cohesion concept.

8.4.2. Orientation Effect

Equation (3.30) indicated clearly that the increase of shear strength using the "apparent cohesion" concept depends on the mobilized tensile force in the reinforcement. The development of tensile force in the reinforcement during shearing of a reinforced soil mass in turn depends strongly on the orientation of the reinforcement with respect to the shear plane. Therefore, the increase in shear strength is dependent on the orientation of the reinforcement with respect to the shear plane. The maximum shear strength increase will be obtained when the tensile reinforcement is orientated in the direction of the principal tensile strain increment which would occur in the unreinforced soil at failure. Orientation of the reinforcement in a direction of compression strain may result in a decrease in shear strength of the soil because of a reduction in the average normal stress on the shear plane (Bassett and Last 1978; Jewell 1989; Palmeira 1987). The experimental results obtained in this investigation seem to support the above statements. This can be

shown in Figures 8.43 through 8.46 in which the normalised shear force versus horizontal displacement relationships are plotted. It is clearly shown that the mobilized shear force is a function of geogrid orientation. The maximum shear force is obtained when the geogrid is orientated at an angle of 30° to the normal of the shear plane. The shear stress versus normal stress relationship (shear strength envelope) for all tested soils and reinforcements are shown in Figures 8.47 through 8.50. These figures show that at certain inclinations the reinforced soils exhibit an increase in shear strength as given by the intercept with the shear stress axes. This intercept was previously defined as "apparent cohesion". For a given soil with the reinforcement inclined at different angles, all the shear strength envelopes are approximately parallel to one another indicating that the mobilized peak friction angles are similar. The only difference between various strength envelope is the magnitude of the intercepts, or "apparent cohesion".

8.4.3. Comparison with Experiment Results

The experimental results for the relationship between the increase of shear strength, or apparent cohesion, and the orientation of the reinforcement are plotted in Figures 8.51 through 8.54. The theoretical relationship was given in Chapter 3 by Equation (3.30) and are shown in Figures 8.55 through 8.58.

$$C_r = \sigma_r [\sin(\theta + \Delta\theta) + \cos(\theta + \Delta\theta) \tan\phi] \quad (3.30)$$

In these figures, the apparent cohesion is normalized by the tensile stress mobilized in the reinforcement. Figures 8.55 to 8.58 indicate that the shear strength increase, or apparent

cohesion is a function of the original orientation of the reinforcement and the angle of dilatancy. For a given soil the maximum shear strength increase depends on the angle of dilatancy of the soil and the amount of shear displacement expressed by the additional geogrid rotation angle $\Delta\theta$.

As mentioned before, this shear strength increase in the reinforced composite also depends on the mobilized tensile stress in the reinforcement. This is incorporated in Equation (3.30). The relationship between mobilized tensile stress and original orientation of the reinforcement, as expressed by Equation (3.39) is plotted in Fig. 8.59. The combination of Equations (3.30) and (3.39) results in Equation (3.40).

$$C_r = K \frac{\cos\theta \tan\delta}{\cos\Delta\theta} [\sin(\theta + \Delta\theta) + \cos(\theta + \Delta\theta)] \quad (3.40)$$

The comparison between theoretical analysis and experimental results are plotted in Figures 8.60 through 8.63. The experimental data and theoretical analysis correlate quite well for peak strength values of the soil.

It is necessary to mention that the shear strength increase in the reinforced soil is also a function of the soil properties (for granular soil given by the friction angle ϕ), reinforcement properties, as represented by the constant K in Equation (3.40), and the interface properties, δ . These interrelationships were shown in the Figures 8.51 through 8.54. It also can be obtained by examining Equation (3.40).

Fig. 8.1. Mohr's Circles of Strains and Stresses for Dense Sand in Direct Shear Apparatus (after Jewell 1987)

Fig. 8.2. Active Equilibrium State for Smooth Retaining Wall (after Jewell 1989)

Fig. 8.3. Theoretical Relationship between ϕ_m and ϕ_{ds} (after Palmeira 1987)

**Fig. 8.4. Comparison between Model's Prediction with Test Results
(Sand with Conwed 5033 at 90°)**

**Fig. 8.5. Comparison between Model's Prediction with Test Results
(Sand with Conwed 5033 at 30°)**

Fig. 8.6. Comparison between Model's Prediction with Test Results
 (Sand with Conwed 5033 at 45°)

**Fig. 8.7. Comparison between Model's Prediction with Test Results
(Sand with Conwed 5033 at 0°)**

Fig. 8.8. Comparison between Model's Prediction with Test Results
(Sand with Tensar UX 1500 at 90°)

Fig. 8.9. Comparison between Model's Prediction with Test Results
(Sand with Tensar UX 1500 at 30°)

Fig. 8.10. Comparison between Model's Prediction with Test Results
(Sand with Tensar UX 1500 at 45°)

**Fig. 8.11. Comparison between Model's Prediction with Test Results
(Sand with Tensar UX 1500 at 0°)**

Fig. 8.12. Comparison between Model's Prediction with Test Results
(Aggregate with Conwed 5033 at 90°)

**Fig. 8.13. Comparison between Model's Prediction with Test Results
(Aggregate with Conwed 5033 at 30°)**

Fig. 8.14. Comparison between Model's Prediction with Test Results (Aggregate with Conwed 5033 at 45°)

**Fig. 8.15. Comparison between Model's Prediction with Test Results
(Sand with Conwed 5033 at 0°)**

**Fig. 8.16. Comparison between Model's Prediction with Test Results
(Aggregate with Tensar UX 1500 at 90°)**

**Fig. 8.17. Comparison between Model's Prediction with Test Results
(Aggregate with Tensar UX 1500 at 30°)**

**Fig. 8.18. Comparison between Model's Prediction with Test Results
(Aggregate with Tensar UX 1500 at 45°)**

Fig. 8.19. Comparison between Model's Prediction with Test Results (Aggregate and with Tensar UX 1500 at 0°)

Fig.8.20. Simple Shear Test Results on Sand (after Strout 1971)

Fig. 8. 21. Special Direct Shear Test (before the Test)

Fig. 8.22. Special Direct Shear Test (after the Test)

Fig. 8.23.. Displacement Vector in Unreinforced Soil

Fig. 8.24. Example of Displacement Vectors in Reinforced Soil

(a)

(b)

Fig. 8.25. Mohr's Circles for Strains and Stresses in the Soil

Fig. 8.26. Mobilized Tensile Strain in the Soil in the Direction of the Reinforcement (Sand with Conwed 5033 at 90°)

Fig. 8.27. Mobilized Tensile Strain in the Soil in the Direction of the Reinforcement (Sand with Conwed 5033 at 30°)

Fig. 8.28. Mobilized Tensile Strain in the Soil in the Direction of the Reinforcement (Sand with Conwed 5033 at 45°)

**Fig. 8.29. Mobilized Tensile Strain in the Soil
in the Direction of the Reinforcement
(Sand with Conwed 5033 at 0°)**

**Fig. 8.30. Mobilized Tensile Strain in the Reinforcement
(Sand with Conwed 5033 at 90°)**

**Fig. 8.31. Mobilized Tensile Strain in the Reinforcement
(Sand with Conwed 5033 at 30°)**

Fig. 8.32. Mobilized Tensile Strain in the Reinforcement
(Sand with Conwed 5033 at 45°)

**Fig. 8.33. Mobilized Tensile Strain in the Reinforcement
(Sand with Conwed 5033 at 0°)**

Fig. 8.34. Shear Test Measuring the Strains in the Reinforcement

Fig. 8.35. Shear Deformation Patterns (after Shewbridge et al. 1988)

Fig. 8.36. Deformation Patterns of the Reinforced Soil (Geogrid at 90°)

Fig. 8.37. Deformation Patterns of the Reinforced Soil (Geogrid at 30°)

Fig. 8.38. Deformation Patterns of the Reinforced Soil (Geogrid at 45°)

Fig. 8.39. Deformation Patterns of the Reinforced Soil (Geogrid at 0°)

Fig. 8.40. Decay Constant b versus Geogrid Orientation Relationship

Fig. 8.41. Concept of Apparent Cohesion (after Schlosser et al. 1972)

Fig. 8.42. Concept of Increase of Confinement (after Yang 1972)

Fig. 8.43. Normalized Shear Strength Behaviour (Sand with Conwed 5033)

Fig. 8.44. Normalized Shear Strength Behaviour (Sand with Tensar UX 1500)

Fig. 8.45. Normalized Shear Strength Behaviour (Aggregate with Conwed 5033)

Fig. 8.46. Normalized Shear Strength Behaviour (Aggregate with Tensar UX 1500)

Fig. 8.47. Shear Strength Envelopes (Sand with Conwed 5033)

Fig. 8.48. Shear Strength Envelopes (Sand with Tensar UX 1500)

Fig. 8.49. Shear Strength Envelopes (Aggregate with Conwed 5033)

Fig. 8.50. Shear Strength Envelopes (Aggregate with Tensar UX 1500)

Fig. 8.51. Shear Strength Increase C_r versus Orientation Relationship
(Sand with Conwed 5033)

Fig. 8.52. Shear Strength Increase C_r versus Orientation Relationship
(Sand with Tensar UX 1500)

Fig. 8.53. Shear Strength Increase C_r versus Orientation Relationship
(Aggregate with Conwed 5033)

Fig. 8.54. Shear Strength Increase C_r versus Orientation Relationship
(Aggregate with Tensar UX 1500)

Fig. 8.55. Theoretical Relationship between C_r and Orientation θ

Fig. 3.56. Theoretical Relationship between C_r and Orientation θ

Fig. 8.57. Theoretical Relationship between C_r and Orientation θ

Fig. 8.58. Theoretical Relationship between C_r and Orientation θ

Fig. 8.59. Theoretical Relationship between P_r and Orientation θ by Equation (3.39)

Fig. 8.60. Comparison between Theoretical Prediction with Test Results
 (Sand with Conwed 5033)

**Fig. 8.61. Comparison between Theoretical Prediction with Test Results
(Sand with Tensar UX 1500)**

Fig. 8.62. Comparison between Theoretical Prediction with Test Results
(Aggregate with Conwed 5033)

Fig..8.63. Comparison between theoretical Prediction with Test Results
(Aggregate with Tensar UX 1500)

CHAPTER 9

ANALYSIS AND DISCUSSION OF PULLOUT TESTS

9.1 INTRODUCTION

The bond strength, or pullout resistance, is an important consideration in a stability analysis of a reinforced earth structure. The pullout test is commonly used to determine the bond strength (pullout resistance) of a reinforcement embedded in soil and to investigate the load transfer mechanism between the soil and reinforcement (Jewell 1984; Juran and Chen 1988b). The rapid development of polymer geogrid reinforcement initiated reinforced soil composites with extensible material. Due to its extensibility the load transfer mechanism is different from that of inextensible reinforcement. In a standard pullout test very little information is obtained about this load transfer mechanism or how the bond strength is mobilized along the reinforcement. In order to investigate the interaction between polymer geogrids and various granular soils, a series of pullout tests were carried out in a large pullout tank (1.55 m×0.9 m×1.22 m) (Chapter 7). The relative displacements of the nodal points of the embedded geogrids were monitored during pullout by means of LVDT's. From these measurements, the average strains, tensile forces and interfacial shear forces mobilized between nodal points were calculated. This chapter will discuss the experimental results (Chapter 7) and those from the proposed theoretical model (Chapter 3). A numerical comparison will also be presented. A new method, based on a simple static model (Chapter 3) to interpret the pullout data will be presented in the next section. This method can be used to calculate the tensile force along the geogrid and the shear stress distribution at the interface along the geogrid. This method is also used

to verify the interface shear stress - displacement relationship which was given in Chapter 3 by Equation 3.43.

9.2 FORCE IN REINFORCEMENT AND INTERFACE SHEAR STRESSES

The geogrid is discretized into a number of elements. The force equilibrium for each element is given as

$$(F-dF)+2*q*dx-F=0 \quad (9.1)$$

$$q=dF/2*dx \quad (9.2)$$

$$\tau=dF/2*W*dx \quad (9.3)$$

Where F = the tensile force in the geogrid,

W = the width of geogrid.

From the nodes displacement versus horizontal displacement relationship (Figs. 7.35 to 7.48), a average tensile strain ϵ_k can be obtained

$$\epsilon_k = (\delta_i - \delta_j)/\Delta x \quad (9.4)$$

where δ_i, δ_j = the displacement at nodes i and j , respectively,

Δx = the length of element k .

A strain distribution curve can then be plotted along the geogrid length. The results from Test 15 are used as an example to illustrate this approach. Figure 7.48 showed the nodal displacement versus front horizontal displacement relationship and Figure 7.33 gave a pullout force - front horizontal displacement relationship. These two figures can be used to calculate the average strains along the geogrid. Figures 9.1 through 9.5 show these distributions at different loading stages. Figure 9.1 shows the strain

distribution at pullout force reaching a value of 24.3 kN (The corresponding horizontal displacement is 60 mm). From Figs. 7.33 and 7.48 it can be seen that at this stage of pullout all five LVDTs recorded displacements, which means that the whole geogrid started to move. Figures 9.1 through 9.5 show that non - linear tensile strain distributions existed along the geogrid at any stage of loading, but a trend of linearlization is gradually formed.

The geogrid is assumed as linear elastic.

$$\epsilon = F / EA_c \quad (9.5)$$

Where ϵ = strain developed in the geogrid,

E = Young's module of the geogrid,

A_c = cross section area of the geogrid.

The force - strain relationship of the geogrid can be used to determine E and A_c . For example, Figure 9.6 shows a force versus strain diagram for Conwed 9027 (Razaqpur 1990).

The average tension force in the geogrid in each element can be calculated from Equations (9.4) and (9.5) in a discretized form as

$$F_k = EA_c(\delta_i - \delta_j) / \Delta x \quad (9.6)$$

where F_k = average tension force in element k .

It should be noted that the Equation 9.6 will be highly approximate and possibly in gross error unless the length of the element k is very small because it ignores the shear along the element. Figures 9.7 through 9.11 show the tension force distribution for Test 15 at different loading stages.

The mobilized interface shear stress distribution along the geogrid can be derived from the tension force in the geogrid by Equation (9.3) in a discretized form as

$$\tau_i = (F_k - F_m) / 2W\Delta x \quad (9.7)$$

where τ_i = interface shear stress,

F_k, F_m = Average tension force in element k and m, respectively.

Figures 9.12 through 9.16 show the shear stress distribution along the geogrid at different loading stages. It can be seen that along the geogrid length the maximum shear stress was gradually mobilized with continuous application of the pullout force. The maximum shear stress occurred at about 250 mm from the front of the geogrid at a pullout force of 24.3 kN corresponding to a horizontal displacement of 60 mm. This maximum shear stress shifted gradually to about 575 mm at a pullout force of 28.9 kN corresponding to a horizontal displacement of 110 mm. The shear stress versus shear displacement relationship can also be plotted. Figure 9.17 shows an example of first element shear stress versus shear displacement relationship. The test results from a direct shear test (sliding test) are also plotted in this figure. The results from the interface model proposed in Chapter 3, is also presented in this figure. This comparison shows that the direct shear test gave an upper bound while the pullout test provided a lower bound. This has probably to do with the fact that in direct shear the shear stresses are mobilized to the same degree along the shear plane for a given displacement, whereas in pullout the shear stresses are mobilized progressively along the upper and lower shear planes. The proposed model results falls between these two bounds.

The analysis of the pullout test results for the geogrid illustrates the non -

uniformity of the displacement along the geogrid, the developed tension force in the geogrid and the mobilized shear stress at the interface between geogrid and soil. It also shows that the mobilized maximum shear stress at the interface is a function of geogrid length.

9.3. NUMERICAL SIMULATION OF PULLOUT TEST RESULTS

Chapter 3 presented a soil-reinforcement interaction model. An example of the numerical simulation of the pullout force versus front displacement relationship has also been given to illustrate the applicability of the model to analyses the pullout test results. This section will give more details of the numerical analysis and parametric study of the proposed model.

9.3.1 Pullout Force versus Front End Displacement Relationship

Figures 9.18 through 9.32 show the comparison of experimental results from the pullout tests and numerical simulations from the proposed theoretical model given in Chapter 3. In these figures it is shown that the model correlates well in most cases with the experimental results whereas in some cases, especially the tests at lower normal stresses after the peak the predictions from the model overestimates the pullout forces. The model can not predict any strain softening. But one should keep in mind that the pullout tests were conducted with different soils (coarse sand and crushed limestone aggregate) and with different geogrids (Conwed 5033, 9027 and Tensar UX 1500), as well as under different normal stresses (10, 20 30 kPa). All these variables contribute to the final pullout force versus displacement relationship.

9.3.2 Extensibility Effect

The longitudinal stiffness or the extensibility of the reinforcement has a significant effect on the load transfer mechanism. It has been shown (Section 9.2) that the interaction between soil and extensible reinforcement results in a non-uniform shear stress at the interfaces. Consequently, the concept of a uniformly mobilized interface shear stress, which is generally used in the design of reinforced earth structures with metallic reinforcements, is not adequate for the interpretation of pullout tests on extensible geogrids and geotextiles.

Figure 9.33 shows the effect of the extensibility of the reinforcement on the pullout force. It illustrates that pullout resistance increases with the elastic modulus. Figure 9.34 shows the effect of the extensibility on the displacement distribution along the length of the reinforcement. The quasi-inextensible reinforcement undergoes a movement similar to a rigid reinforcement and the displacement (strain) mobilized in the reinforcement is quasi-linear, and consequently the shear stress developed at the interface is rather uniform. However, on the other hand, for extensible reinforcement ($E=100$ kPa or less), the mobilized strain and the force in the reinforcement, as well as the mobilized shear stress at the interface are non-uniformly distributed.

It is necessary to point out that the relationships shown in Figs. 9.33 and 9.34 are only valid under two conditions: 1) a perfect bonding exists between the geogrid and soil; 2) there is a upper limit beyond that these relationship would not valid.

9.3.3. Embedded Length of the Reinforcement

It has been demonstrated that scale of the apparatus in the pullout test has a

significant effect on the results (Palmeira 1989). Parameters to be used in the design of reinforced soil structures should be independent of the dimension of the geogrid specimen size in a pullout test. Figure 9.35 shows the effect of the embedded length on the shear stress mobilized at the interfaces during the peak of the pullout curve. Figure 9.36 shows the effect of the embedded length on the peak pullout displacement. The results of these numerical simulations illustrate that, with quasi-inextensible reinforcement, a uniformly mobilized interface shear stress can be adequately used. The design limit shear stress is independent of the specimen dimension, and the results of pullout tests can therefore be used in the design of actual structures. With extensible reinforcement, because of the nonuniform shear stress distribution, the average limit shear stress mobilized at the peak of the pullout curve is a function of the embedded length. Therefore extrapolation of pullout test results for the design of soil structures, reinforced with extensible elements, requires a careful evaluation of scale effect.

9.4. INTERPRETATION OF RESULTS FROM PULLOUT TEST

9.4.1 Conventional Analysis

The conventional method to interpret the results from pullout tests is an analogy to the method used to interpret the results from direct shear tests. A peak value of pullout force is taken from a pullout force versus pullout displacement relationship. A normal stress versus pullout resistance (peak pullout force divided by the nominal area) can then be plotted, if at least three tests are conducted under the same condition varying only the normal stress. A curve fitting technique is used to fit the experimental points of the

normal stress versus pullout resistance curve. A linear relationship can be approximated in many cases as given by Equation (7.1). Two parameters, a coefficient of interlock and an intercept parameter are obtained from the analysis. These can be considered as constants for a given composite. Chapter 7 presented this basic analysis of results from small and large pullout tests.

9.4.2. Flow Rule Analysis

Flow rule is the relationship between plastic strain increment ratio and stress ratio. It relates the soil strength behaviour to the deformation behaviour. One approach to define the flow rule is to balance the energy dissipation within the composite and the work done in changing the shape of the composite sample. For pullout test part of the energy dissipates by friction and bearing during failure at the interface between soil and reinforcement and part of the energy is stored in the reinforcement as elastic energy. As stated in Chapter 2, both the friction and the bearing failure is a function of overburden pressure. The bond coefficient f_b (Eqn. 2.2), which includes the friction and bearing, can be used in the analysis. Therefore, the energy within the composite in pullout can be rewritten as

$$\delta E = f_b N \delta x + \int_0^l \frac{1}{2} \sigma \epsilon A_c dx \quad (9.8)$$

where δE = energy dissipation,

f_b = bond coefficient,

δx = relative displacement between the reinforcement and soil (slip)

N = normal load.

σ = tensile stress in the reinforcement

ϵ = tensile strain in the reinforcement

A_c = cross section area of the reinforcement

l = length of the reinforcement.

The work input by the applied normal load N , and pullout force F on the soil and reinforcement are

$$\delta W = F\Delta x + -N\Delta y \quad (9.9)$$

where δW = the work done by the force on the composite,

Δx = front displacement of the reinforcement which include the extension of the reinforcement and the slip between reinforcement and soil,

Δy = vertical displacement of the soil at the interfaces

Since there are many unknown parameters involved in Equations 9.8 and 9.9 direct evaluation seems difficult. A parameter of total work, W_T , is defined which is actually the work done by pullout force, and is given as Equation 9.10.

$$W_T = \int F d(\Delta x) \quad (9.10)$$

The reason to define the work done by the pullout force as total work is that it is believe that negative work done by the normal stress is very small, because that soil dilatancy is restrained by the surrounding soil in large pullout test apparatus. Equation (9.10) represents the total area under the pullout force versus front displacement relationship, which can be easily calculated. The total work versus normal load relationship can then be plotted. Figures 9.37 to 9.40 show the analysis of the pullout test results by the flow rule approach. These figures reveal that a linear relationship exists

between the work done by the applied pullout force and the normal load. By using a curve fitting technique, two parameters can be obtained. These parameters can then be used to describe the behaviour of the composite. Tables 9.1 through 9.4 summarize the test results from the small pullout apparatus while Tables 9.5 and 9.6 give the results for the large pullout tests.

The advantage of this analysis over a conventional analysis is that it has taken the whole range of pullout force and displacement into account, instead of just selecting a peak value. The parameters obtained from the flow rule analysis will still be meaningful if there is no distinct peak value for the pullout force versus front end displacement relationship. The conventional analysis, on the other hand, has little meaning when the test curve does not exhibit a peak value.

Table 9.1 SUMMARY OF PULLOUT TEST RESULTS FOR COARSE SAND

Geogrid	Normal Stress (kPa)	Total Work (J)
Conwed 9027	14	38.78064
	20	56.80524
	25	67.78010
Conwed 5033	14	39.98388
	20	51.01776
	25	62.04517
Conwed 3024	14	31.92842
	20	35.34089
	25	37.11173
Conwed 3022	14	31.52459
	20	35.80635
	25	39.25039
Tensar UX 1500	14	60.14598
	20	71.12974
	25	85.32770

Table 9.2 SUMMARY OF PULLOUT TEST RESULTS FOR AGGREGATE

Geogrid	Normal Stress (kPa)	Total Work (J)
Conwed 9027	14	55.21425
	20	65.73718
	25	77.90411
Conwed 5033	14	48.75777
	20	55.21425
	25	58.83289
Conwed 3024	14	33.32377
	20	36.16893
	25	38.63787
Conwed 3022	14	36.51126
	20	38.07719
	25	40.34073
Tensar UX 1500	14	48.92998
	20	65.76446
	25	75.89071

Table 9.3 SUMMARY OF PULLOUT TEST RESULTS FOR FINE SAND

Geogrid	Normal Stress (kPa)	Total Work (J)
Conwed 9027	14	32.77512
	20	49.51496
	25	55.53336
Conwed 5033	14	30.39193
	20	45.81057
	25	53.89688
Conwed 3024	14	25.84308
	20	34.01332
	25	36.32533
Conwed 3022	14	22.84970
	20	28.57248
	25	36.38514
Tensar UX 1500	14	41.06352
	20	55.35650
	25	65.68149

TABLE 9.4 PARAMETERS FOR PULLOUT RESISTANCE

Geogrid	Soil Type	Coefficient of a (mm)	Intercept, b (J)	Coefficient of Fit
Conwed 9027	Crushed Limestone Aggregate	2.05253	25.91873	0.99556
5033		1.48534	22.44562	0.99372
3024		0.48281	26.54833	0.99930
3022		0.34526	31.51960	0.98767
Tensar 1500		1.99564	25.94750	0.99997
Conwed 9027	Coarse Sand	2.33601	9.62878	0.99953
5033		2.14952	8.20839	0.99991
3024		0.53799	24.45429	0.99931
3022		0.70272	21.70700	0.99995
Tensar 1500		2.27413	27.47662	0.99920
Conwed 9027	Fine Sand	2.09270	4.78468	0.97737
5033		2.15109	1.06175	0.99210
3024		0.96641	7.14635	0.96647
3022		1.22137	5.24878	0.99003
Tensar 1500		2.24275	9.92642	0.99919

TABLE 9.5 LARGE PULLOUT TEST RESULTS

Geogrid	Normal Stress kPa	Total Work J
Conwed 5033	10.0	1633.91522
	20.0	1850.52014
	30.0	1867.73831
Conwed 9027	10.0	1719.33667
	20.0	2503.37978
	30.0	3004.32582
Tensar UX 1500	10.0	1457.45646
	20.0	2236.57315
	30.0	2974.38414

TABLE 9.6 PARAMETERS FOR LARGE PULLOUT RESISTANCE

Geogrid	Soil Type	Coefficient of a (mm)	Intercept, b kJ	Coefficient of Fit
Conwed 9027	Coarse Sand	0.06425	1.12397	0.99201
Conwed 5033	Coarse Sand	0.01169	1.55021	0.94511
Tensar 1500	Coarse Sand	0.07585	0.70593	0.99988

9.4.3. Soil Dilatancy

Dense granular soil will dilate under any normal stress when sheared. In a pullout test, the shear takes place on two interfaces, above and below the reinforcement, with the soil. For ribbed strip reinforcement the soil will dilate in the fashion as shown in Figure 9.41. In the vicinity of the reinforcement the soil tends to increase its volume (i.e. dilate) during shear. This positive volume change is restrained by the surrounding soil. In the ribbed strip case, the ribs cause the shearing zone to increase in size. Both the increase of the shear volume and the increase in local stress, because of soil dilatancy, result in an increase in bond coefficient. For grid reinforcement the soil may dilate as shown in Figure 9.42. The change of local confining stresses may be distributed as shown in this figure.

However, the above analysis methods (conventional and flow rule analysis) of the pullout test results made no reference to the soil volume change or soil dilatancy. The reason is that in the large pullout test setup, the volume of soil above and below the reinforcement is far larger than the volume of soil around the reinforcement. The local change of stresses does not affect the overall behaviour of the soil. The measurement of vertical displacement at the top of the soil specimen was attempted in this investigation. The results showed that a very small amount of vertical displacement at the surface occurred during pullout. On the other hand, if a relatively shallow test apparatus is used, the soil dilatancy will have some visible effect. Therefore, the scale of the test apparatus plays a role in assessing the degree of soil dilatancy from boundary measurements.

9.5. DIRECT SHEAR AND PULLOUT TESTS

The research carried out in this study not only included a comprehensive experimental program on direct shear test (including direct sliding) but also a series of pullout tests. The purpose of these experimental programs was to study the strength and deformation behaviour of the reinforced soil composite by geogrid on a representative scale in order to have a comprehensive understanding of the behaviour of a reinforced soil structure. This experimental program was designed in such a way that most possible failure mechanisms were dealt with (Fig. 1.2), which included direct shear of the composite, direct sliding on the interface between geogrid and soil and pullout of the geogrid through the soil. In all these tests, the direct shear tests were prominent not only because there is a lack of data in the literature but also because the test could evaluate several variables. For example, referring to Fig. 9.43, the orientation of the geogrid varied from 90° at the toe of the retaining wall to 0° at the top of the wall to the normal to the shear plane. Thus, a large number of direct shear tests had to be conducted to evaluate these variables. Actually, 102 large direct shear tests were carried out over the past three years, while sixty six pullout test were conducted in the previous year. The pullout tests included fifty one small pullout tests and fifteen large pullout tests.

There are several differences between a direct shear and a pullout test. In a direct shear test the shear and normal loads are applied to the soil, while in a pullout test, although the normal load is exerted to the soil, the pullout force is applied directly to the reinforcement. In a direct shear test the shear occurs at a predefined shear plane and the whole composite fails by shear. In a pullout test the shear takes place at two interfaces

between the reinforcement and soil. In a direct shear test, the soil volume changes, due to dilatancy. This phenomenon plays an important role in the mobilization of tensile force in the geogrid since the whole composite is involved. However, in a pullout test, the dilatancy of the soil occurs only in the vicinity of the reinforcement and is restrained by the surrounding soil. Only local stresses are modified.

Perhaps the most important difference between a direct shear and a pullout test resides in the stress ratio experienced by the soil, as pointed out by Jewell (1980). In a direct shear test the reinforcement improves the strength of the soil by modifying the strains which develop as the soil deforms plastically under an applied load. The sequence is that: an increment of shear load causes the soil to undergo strain (strain hardening). The strain in the soil causes displacement in the adjacent soil particles in the direction of the reinforcement. This displacement induces strain in the reinforcement due to friction and bearing of the reinforcement elements on the soil. Consequently, the mobilized tensile strain and tensile force in the reinforcement modify the strains and stresses in the soil and cause the shear strength to increase. On the other hand, in a pullout test the soil is "at rest" (Jewell 1980) and, therefore, experiences a relative low stress ratio. As the pullout force is applied to the reinforcement, the soil immediately adjacent to the reinforcement experiences a rapid increase of stress ratio. Once this ratio exceeds the critical state value, significant plastic strains occur in this portion of the soil. But these strains are constrained by the surrounding soil.

However, the direct shear and the pullout tests are related to each other. First, they are two phenomena happening in the same reinforcement soil structure if we isolate a unit

cell of one layer of reinforcement with half metre soil above and below, which was the basis of designing the test apparatus as indicated in Fig. 9.43. Secondly, the interaction mechanism between the reinforcement and the soil, in either direct shear or pullout, is essentially the same no matter how the load is applied, either to the soil or to the reinforcement. Thirdly, the direct shear and pullout tests are connected through the mobilized tensile strain and tensile force in the reinforcement. In Chapters 3 and 8, it was demonstrated that the shear strength increase in a reinforced soil depends directly on the maximum tensile force mobilized in the geogrid. Furthermore, it was also established that the force in the geogrid is a function of the orientation of the geogrid. This tensile force generated by direct shear along a potential shear zone (BC in Fig 9.43.) is a driving force for the embedded geogrid in the soil. This part of the geogrid (DE as shown in Fig. 9.43) is under the action of pullout. The stability (internal) of a reinforced soil structure (Fig. 1.2, or 6.1) will be dependent on the embedded length or the breakage strength of the reinforcement. If the embedded length is sufficient and the mobilized tensile force is below the limit strength of the reinforcement, the reinforced soil will not fail which would happen without the reinforcement. Therefore, a reasonable estimation of the tensile force that will be mobilized in reinforcement in a direct shear test will give an idea for the embedded design length of the reinforcement. A better evaluation of the embedded length is obtained in a pullout test.

Fig. 9.1. Strain Distribution at 24.3 kN of Pullout Force

Fig. 9.2. Strain Distribution at 26.3 kN of Pullout Force

Fig. 9.3. Strain Distribution at 27.6 kN of Pullout Force

Fig. 9.4. Strain Distribution at 28.7 kN of Pullout Force

Fig. 9.5. Strain Distribution at 28.9 kN of Pullout Force

Fig. 9.6. Force Strain Relationship of Comwed 9027
(after Razaqpur, 1990)

Fig. 9.7. Tensile Force Distribution at 24.3 kN of Pullout Force

Fig. 9.8. Tensile Force Distribution at 26.3 kN of Pullout Force

Fig. 9.9. Tensile Force Distribution at 27.6 kN of Pullout Force

Fig. 9.10. Tensile Force Distribution at 28.7 kN of Pullout Force

Fig. 9.11. Tensile Force Distribution at 28.9 kN of Pullout Force

Fig. 9.12. Shear Stress Distribution at 24.3 kN of Pullout Force

Fig. 9.13. Shear Stress Distribution at 26.3 kN of Pullout Force

Fig. 9.14. Shear Stress Distribution at 27.6 kN of Pullout Force

Fig. 9.15. Shear Stress Distribution at 28.7 kN of Pullout Force

Fig. 9.16. Shear Stress Distribution at 28.9 kN of Pullout Force

Fig. 9.17. Shear Stress vs. Shear Displacement Relationship

Fig. 9.18. Numerical Simulation of Large Pullout Test 1

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MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Fig. 9.19. Numerical Simulation of Large Pullout Test 2

Fig. 9.20. Numerical Simulation of Large Pullout Test 3

Fig. 9.21. Numerical Simulation of Large Pullout Test 4

Fig. 9.22. Numerical Simulation of Large Pullout Test 5

Fig. 9.23. Numerical Simulation of Large Pullout Test 6

Fig. 9.24. Numerical Simulation of Large Pullout Test 7

Fig. 9.25. Numerical Simulation of Large Pullout Test 8

Fig. 9.26. Numerical Simulation of Large Pullout Test 9

Fig. 9.27. Numerical Simulation of Large Pullout Test 10

Fig. 9.28. Numerical Simulation of Large Pullout Test 11

Fig. 9.29. Numerical Simulation of Large Pullout Test 12

Fig. 9.30. Numerical Simulation of Large Pullout Test 13

Fig. 9.31. Numerical Simulation of Large Pullout Test 14

Fig. 9.32. Numerical Simulation of Large Pullout Test 15

Fig. 9.33. Effect of Extensibility on Pullout Force

**Fig. 9.34. Effect of Extensibility on the Shear Displacement
Distribution along the Geogrid**

Fig. 9.35. Effect of Reinforcement Length on Average Shear Stress (after Juran and Chen 1988)

Fig. 9.36. Effect of Reinforcement Length on Peak Pullout Displacement (after Juran and Chen 1988)

Fig. 9.37. Total Work vs. Normal Stress Relationship
(Coarse Sand in Small Test Apparatus)

Fig. 9.38. Total Work vs. Normal Stress Relationship
(Aggregate in Small Pullout Apparatus)

Fig. 9.39. Total Work vs. Normal Stress Relationship
(Fine Sand in Small Pullout Apparatus)

Fig. 9.40. Total Work vs. Normal Stress Relationship
(Coarse Sand in Large Pullout Apparatus)

$$\mu = \frac{\bar{\tau}}{\delta_z} > \mu^*_{real} = \frac{\tau_{real}}{\sigma_{v, real}}$$

μ^* : Apparent friction coefficient
 $\bar{\tau}$: Mobilized average shear stress
 γ_z : Overburden stress

Fig. 9.41. Soil Dilatancy Effect (after Schlosser et al. 1978)

Fig. 9.42. Soil Dilatancy Effect in Pullout of Geogrid

Fig. 9.43. Schematic Diagram for Unit Cell Test

Fig. 9.44. Local Stress Equilibrium (after Mitchell 1986)

CHAPTER 10

SUMMARY AND CONCLUSIONS

10.1. SUMMARY

A literature review indicated that there exists a substantial lack of data on the shear behaviour of soils reinforced with geogrids, especially the shear strength and deformation behaviour of reinforced soil from direct shear tests. Furthermore, most of the existing experimental results were obtained from fairly small tests and the scale or boundary effects seem to influence the test results in order to warrant an extrapolation of these experimental results for design purpose. In order to overcome these limitations, two large experimental apparatuses were designed and built. There are four potential failure mechanisms which a reinforced earth structure can undergo as shown in Fig. 1.2. (1) tensile failure of the reinforcement (which is beyond the scope of this investigation), 2) sliding along the soil - reinforcement interface, 3) shearing through the soil - reinforcement composite material along a failure surface, and 4) pullout of reinforcement behind the failure surface if the anchorage length is insufficient or the anchorage capacity is exceeded.

In order to have a complete overview and a better understanding of the behaviour of a reinforced earth structure it is justified to choose the direct shear test (special case is the sliding test) and the pullout test to simulate the load transfer mechanisms in the laboratory. Based on these considerations a comprehensive experimental program was carried out over the last three years. In the first stage, 64 preliminary small pullout tests were carried out to investigate the properties of Conwed Stratagrids and to select the

proper geogrids for further investigation. The second stage of the test program consisted of over 100 large direct shear and 15 pullout tests which were the main part of this study. Some special tests were performed to investigate the mobilized strain in the reinforcement during shear and pullout and to monitor the mobilized strains in the soil. Based on the experimental results theoretical analyses were also conducted to model the load transfer in direct shear and pullout between soil and reinforcement. The predictions from the proposed models correlated well with the experimental results in most cases. As part of the interpretation of the test results a flow rule method was adopted to analyze the experimental data. It was shown that the flow rule analysis is more rational and yields a better correlation than a conventional Mohr-Coulomb analysis. It is necessary to point out that in a reinforced soil system the global behaviour is dependent on the strength and deformation mobilized in both soil and reinforcement. It also depends on the compatibility (or slip) at the interface of these two components. The tensile strain mobilized in the soil, in the direction of the reinforcement, is the cause of the tensile forces developed in the reinforcement. The interface friction between and the bearing of the transverse members of the reinforcement on the soil play an important role in the process.

10.2. MAIN CONCLUSIONS

Based on the results and analysis of the previous chapters the main conclusions are given in the following sections.

Direct Shear Tests

1. In most reinforced soil structures the orientation of the reinforcement with

respect to the shear plane varies. The evaluation of the orientation on the shear strength and deformation of a composite was a major concern in this investigation. The experimental results, from over 100 direct shear tests in combination with three soils, three geogrids, three different normal stresses and four orientations (0° , 30° , 45° and 90° to the normal of the shear plane) have shown that the shear strength mobilized in the composite depends on the orientation of the geogrid. Maximum shear strengths were observed in the tests when the reinforcement was inclined at 30° to the normal of the shear plane while minimum shear resistances were obtained when the geogrid was placed in the shear plane (90° to the normal of the shear plane). The theoretical model, proposed in this thesis, confirmed the experimental observations and showed that the shear strength of a composite is not only a function of the orientation of the reinforcement, but also a function of the soil friction angle. Further analysis of the strain mobilized in the soil, in the direction of the reinforcement, and the strain in the reinforcement have shown that when the reinforcement is placed close to the minor principal strain direction a large tensile strain (force) is mobilized in the reinforcement and a substantial increase in the overall shear strength for the composite was obtained. Therefore, it is concluded that it is necessary to take the orientation of the reinforcement into account in the design of reinforced soil structures. The reinforcement, if possible, should be placed close to the direction of the minor principal strain (tensile strain) in order to achieve optimum efficiency. If the reinforcement has to be placed horizontally, the shear strength of the composite should be integrated over

the probable failure plane to account for the orientation effect.

2. Dense granular soil will dilate (expand in volume) when sheared. The soil dilatancy is an indication of mobilization of soil plastic strains. The soil dilatancy can be quantified as an angle of dilation which is a function of the horizontal and vertical displacements of the soil under shear. Soil dilatancy was observed in all direct shear tests. Unlike conventional metallic reinforcements, geogrids are considered to be flexible elements. They have small bending stiffness and their interaction with soil are also different from that of flat strip reinforcement. They become effective only when stretched to mobilize tensile force (strain). The dilatancy of the soil provides a link between the soil and reinforcement by inducing tensile force (strain) in the reinforcement. The tensile force (strain) in the reinforcement in turn will cause the shear strength of the whole composite to increase by adding an additional component. The amount of tensile strain mobilized in the reinforcement and the overall increase of the shear strength of the composite depends on many parameters, such as soil properties, geogrid properties, normal stresses and the geogrid orientation with respect to the shear plane. The maximum shear strength will be obtained when the maximum tensile strains in the reinforcement is mobilized. It can be concluded that soil dilatancy (plastic deformation) in dense granular soil is a major cause of tensile force (strain) mobilization in geogrid reinforcement. The tensile force (strain) in a geogrid, in turn, adds an additional shear strength component to the reinforced soil composite. Therefore, in the design of a reinforced soil structure, the backfill

materials should be specified as dense in order to achieve the best reinforcing effect through soil dilatancy.

3. The relationship between mobilized shear strength and tensile strain (in the soil and reinforcement) is crucial for the analysis of reinforced soil since the tensile strain provides a link between the soil and reinforcement deformation. The tensile strain in the soil in the direction of the reinforcement can be calculated by Equation 8.12 and the tensile strain in the reinforcement can be obtained from Equation 3.33. A comparison between these strains has indicated that there was a relative movement (slip) between these two components. Therefore perfect bonding does not exist between reinforcement and soil. Since the difference was not very large, the strain compatibility condition can be used as a first approximation but caution needs to be exercised.

4. Flow rule analysis combined with the theoretical model proposed in this thesis yielded Equation 8.7. This equation can be used to simulate the shear force versus shear displacement relationship for a direct shear test. A comparison with the experimental results has shown a good agreement. It, therefore, provides a good tool to correlate the mobilized tensile force in the reinforcement, soil dilatancy angle and shear strength of the composite. Therefore, it is concluded that this method should be used in the analysis of test results from direct shear test for reinforced soil where deformation data is available.

5. Unlike dense granular soils, the shear strength of a reinforced soil increases continuously with shear displacement. This phenomenon is due to the fact that

under continued shear displacement the tensile strength of the reinforcement becomes dominant. The resulting shear stress versus shear displacement relationship rarely exhibited a distinct peak value. Therefore, the conventional Mohr-Coulomb's analysis, which is based on peak (or residual) strength seems ill suited to characterize the shear behaviour of reinforced soils. A flow rule or energy method seems more suitable to analyze the strength behaviour of the composite.

6. A conventional Mohr - Coulomb's analysis was carried out for the range of normal stresses used in this investigation in order that the results could be compared with those published by other researchers. All reinforced soils showed an intercept with the shear stress axis. This intercept represents the shear strength increase due to the reinforcement elements. The intercept was termed "apparent cohesion" in this study after Schlosser and Vidal (1969). A reinforced composite and the natural soil yielded more or less the same internal friction angles at soil displacement corresponding to peak shear stress of the natural soil. The angle of shear resistance of the composite can therefore be taken as that of the unreinforced soil at the same conditions (density and moisture content of the soil).

7. In the selection of a proper size of shear test apparatus, a criterion proposed by Jewell and Wroth (1987) suggested that the ratio of the minimum dimension of the shear box to the mean particle size should be in the order of 50. The large direct shear apparatus of 1000x1000x960 mm used in this investigation met this criterion and the assembly has been used effectively to test granular soils with a

particle size to 20 mm in diameter. This apparatus will also duplicate the quasi plane strain deformation and shear conditions for long reinforced soil structures. In this test box granular soils could be compacted to site specific density and one could accommodate large representative geogrid specimens. Therefore, in order to assess the shear strength and deformation behaviour of granular and reinforced soils the type and size of the shear apparatus used in this research was found to be adequate and suitable for testing materials similar to conditions specified in the field.

Pullout Tests

1. The pullout resistance of a geogrid reinforced composite is a function of soil properties, geogrid properties and normal stress. A soil with large particles provides better friction and interlock resulting in a high pullout resistance. Higher normal stresses result in higher shear and interlock stresses thereby giving high pullout resistance. The pullout resistance of a given composite can be estimated from Equation 7.1 similar to a Mohr - Coulomb's analysis for direct shear test results. The two parameters, a coefficient of interlock and an intercept value are obtained from Equation 7.1 . These parameters can be considered constants for a given composite and could be used in the design of reinforced soil structures.
2. The flow rule analysis relates the soil strength to the deformation behaviour by considering the balance of the energy dissipation within the soil and the work done by the external load on the soil sample. It was shown that the total work done by the pullout force is a function of the normal load. An approximate linear

relationship exists for a given soil and corresponding geogrid composite. The advantage of this analysis over a conventional analysis is that it takes the whole range of the pullout force and displacement relation into account. The parameters obtained from this analysis are particularly meaningful when there is no distinct peak value in the pullout force versus geogrid displacement relationship. From the flow rule method it was shown that the unreinforced soil will collapse after peak due to a sudden reduction in shear strength since the soil cannot absorb any additional external applied energy. On the other hand, the strength of a reinforced soil will continue to increase with geogrid displacement and the system will absorb additional external applied energy. Therefore it can be concluded that the flow rule analysis will describe the pullout behaviour of a geogrid more adequately.

3. The proposed load transfer model considers the important aspects of a geogrid reinforced soil subjected to pullout. A non-linear elastoplastic, strain hardening interface model was adopted. The model correlated well with the experimental results in most cases. The predictions overestimated the pullout force for cases of low normal stresses. Therefore, this model can be used to predict the behaviour of geogrid reinforced soil subject to pullout force and to study the interaction between soil and reinforcement.

4. The displacement of nodes along the embedded length of the geogrid were monitored and the average sectional tensile strains in the reinforcement were calculated. A numerical method, based on the experimental data to calculate the

distribution of the tensile strains and forces in the reinforcement and the distribution of the interface shear stresses was proposed. This method can be extrapolated to calculate the embedded length and maximum pullout force for a reinforced soil structure. Calculated interface shear stresses correlated well with the results from a sliding test in a direct shear. This method, therefore, could be used to determine the embedded length needed to proportion prototype structures.

5. The pullout resistance of a geogrid depends on the ratio of the aperture of the geogrid to particle size diameter. A ratio of 2 to 3 seems to provide the best soil/grid interlock (which is the case of coarse sand and crushed limestone aggregate reinforced with Conwed 9027, 5033 and Tensar UX 1500). Two other Conwed geogrids 3024 and 3022 exhibited smaller pullout resistances than those of soil reinforced with Conwed 9027 and 5033. The ratios of the aperture of these geogrids to the maximum particle size were 0.5 to 1, respectively for grids 3024 and 3022. Therefore, in order to achieve best interlock and thereby maximum pullout resistance in soil, a ratio of 2 to 3 is suggested.

6. The scale of the pullout apparatus has a significant effect on the pullout resistance. A large scale test is beneficial in the sense that boundary effects are minimized and a representative size of reinforcement can be used. The soil can also be placed at site specific density and moisture content. The pullout resistance of a given composite is also a function of the embedded length. The scale of the apparatus has a direct influence on the embedded length. Furthermore the embedded length is an important parameter in the design of a reinforced soil

structure. Therefore, the extrapolation of test results from a small pullout test apparatus to prototype structures should be done with caution.

APPENDIX MOBILIZED STRAIN IN DIRECT SHEAR TESTS

TABLE A1. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 30

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	-0.0042	-0.0125	-0.0083	-0.0063	-0.0083	-0.0063	-0.0063	-0.0104
		-0.0050	-0.0025	-0.0050	-0.0075	-0.0025	-0.0050	-0.0050	-0.0025
		-0.0050	-0.0100	-0.0050	0.0000	0.0000	-0.0110	-0.0060	-0.0050
ϵ_z	2	-0.0017	0.0100	0.0000	0.0050	0.0000	0.0150	0.0025	0.0050
		0.0017	0.0075	0.0000	0.0000	0.0000	0.0100	0.0000	0.0083
		0.0017	0.0025	0.0050	0.0000	-0.0050	0.0075	0.0025	0.0067
γ_{xy}	3	-0.0021	-0.0042	0.0083	0.0029	-0.0010	-0.0017	-0.0037	-0.0079
		0.0083	-0.0025	0.0050	0.0100	0.0010	0.0150	0.0200	0.0133
		0.0067	0.0050	0.0075	0.0100	0.0035	0.0105	-0.0080	0.0033

TABLE A2. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 31

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0000	0.0000	0.0021	0.0042	0.0042	0.0000	0.0042	-0.0021
		-0.0025	-0.0100	-0.0075	0.0000	0.0000	-0.0050	-0.0025	0.0025
		-0.0100	-0.0050	-0.0050	0.0050	0.0150	-0.0050	-0.0200	-0.0100
ϵ_z	2	-0.0017	0.0025	0.0100	0.0015	0.0000	0.0013	0.0025	-0.0033
		-0.0017	0.0000	0.0075	0.0250	0.0000	0.0075	0.0050	-0.0017
		-0.0017	0.0025	0.0050	0.0250	0.0000	0.0050	0.0050	0.0000
γ_{xy}	3	-0.0021	0.0021	0.0125	0.0179	-0.0020	-0.0121	0.01292	-0.0017
		0.0042	0.0050	0.0000	0.0175	-0.0030	0.0025	0.01250	0.0008
		-0.0017	0.0025	-0.0250	0.0100	-0.0150	0.0125	0.00000	0.0000

TABLE A3. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 32

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	-0.0021	0.0021	0.0063	0.0042	0.0042	0.0042	0.0042	0.0042
		-0.0075	-0.0025	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		-0.0150	-0.0150	-0.0100	0.0100	0.0100	-0.0000	-0.0100	-0.0100
ϵ_z	2	0.0017	0.0125	0.0000	0.0010	0.0010	0.0025	0.0025	0.0033
		0.0000	0.0075	0.0025	0.0010	0.0150	0.0075	0.0000	0.0017
		0.0000	0.0025	0.0050	0.0010	0.0200	0.0075	0.0000	0.0017
γ_{xy}	3	0.0038	0.0145	-0.0025	0.0000	0.0000	0.0000	-0.0021	0.0000
		0.0017	0.0071	0.0068	0.0023	0.0023	-0.0023	0.0000	0.0000
		0.0017	-0.0025	0.0000	0.0050	0.0100	0.0050	0.0000	0.0000

TABLE A4. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 40

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0000	0.0065	0.0125	0.0146	0.0125	0.0125	0.0083	0.0021
		0.0000	0.0100	0.0000	0.0000	0.0025	0.0025	0.0025	-0.0025
		0.0050	-0.0050	-0.0015	-0.0200	-0.0150	-0.0200	-0.0250	-0.0150
ϵ_z	2	0.0050	0.0050	0.0050	0.0050	0.0000	0.0050	0.0050	0.0017
		0.0033	0.0025	0.0000	0.0100	-0.0050	0.0100	0.0050	0.0033
		0.0033	0.0050	0.0000	0.0000	0.0050	0.0075	0.0050	0.0017
γ_{xy}	3	0.0096	0.0020	0.0058	0.0379	0.0308	0.0067	0.0125	0.0013
		0.0033	0.0025	-0.0018	0.0100	0.0150	-0.0150	0.0100	0.0058
		0.0150	0.0075	-0.0075	0.0200	0.0150	0.0050	0.0075	0.0017

TABLE A5. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 41

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0063	0.0063	0.0125	0.0188	0.0013	0.0063	0.0063	0.0000
		0.0000	-0.0050	-0.0050	-0.0025	-0.0050	-0.0025	-0.0050	-0.0075
		0.0000	-0.0050	-0.0100	-0.0100	0.0000	-0.0150	-0.0200	0.0050
ϵ_z	2	0.0017	-0.0025	0.0075	0.0000	0.0100	-0.0025	0.0100	0.0017
		0.0033	0.0025	0.0025	0.0050	0.0015	0.0050	0.0000	0.0000
		0.0033	0.0025	0.0050	0.0050	0.0050	0.0075	0.0000	0.0000
γ_{xz}	3	0.0029	-0.0025	-0.0204	-0.0175	-0.0067	-0.0254	-0.0142	-0.0129
		0.0083	0.0225	-0.0125	0.0175	0.0300	0.0075	0.0150	-0.0067
		0.0033	0.0175	-0.0500	-0.0150	0.0000	0.0025	-0.0100	0.0050

TABLE A6. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 42

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0083	0.0021	0.0042	0.0083	0.0042	0.0042	0.0042	0.0021
		-0.0075	-0.0075	0.0000	0.0050	0.0025	-0.0050	-0.0050	-0.0025
		-0.0200	0.0050	-0.0050	-0.0200	0.0050	0.0050	0.0000	0.0150
ϵ_z	2	-0.0012	-0.0025	0.0050	-0.0100	-0.0150	0.0050	0.0225	-0.0017
		0.0083	0.0050	0.0025	0.0050	0.0050	0.0075	0.0100	-0.0050
		0.0050	0.0100	0.0000	0.0100	0.0200	0.0050	0.0000	-0.0017
γ_{xz}	3	-0.0013	0.0046	-0.0100	0.0000	0.0038	-0.0175	-0.0104	-0.0004
		0.0075	0.0025	-0.0125	-0.0150	0.0200	-0.0275	0.0025	0.0008
		0.0150	-0.0025	-0.0175	0.0150	0.0300	0.0125	0.0000	0.0033

TABLE A7. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 53

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0042	0.0021	0.0083	0.0125	0.0083	0.0042	0.0062	0.0042
		-0.0050	-0.0125	-0.0125	-0.0150	-0.0050	-0.0050	-0.0125	-0.0075
		-0.0300	0.0500	-0.0150	-0.0400	-0.0450	-0.0350	-0.0150	-0.0200
ϵ_z	2	0.0100	-0.0025	0.0050	-0.0100	0.0150	-0.0550	0.0650	-0.0033
		0.0067	-0.0050	0.0075	-0.0200	0.0150	0.0050	0.0025	0.0017
		0.0033	-0.0075	0.0150	-0.0400	0.0250	0.0075	-0.0025	0.0050
γ_{xy}	3	-0.0067	0.0154	-0.0092	-0.0083	0.0038	-0.0517	-0.0400	0.0042
		-0.0012	0.0075	-0.0100	0.0050	0.0075	-0.0100	-0.0200	-0.0058
		-0.0013	0.0125	-0.0025	-0.0200	-0.0300	0.0075	0.0025	0.0083

TABLE A8. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 54

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	-0.0083	-0.0063	0.0000	0.0063	0.0083	0.0042	-0.0063	-0.0479
		0.0075	0.0000	0.0025	0.0025	0.0025	0.0025	0.0150	0.0125
		-0.0300	-0.0100	-0.0100	-0.0200	0.0150	-0.0200	-0.0300	-0.0350
ϵ_z	2	0.0050	-0.0075	-0.0150	0.0050	0.0100	0.0100	0.0100	-0.0050
		-0.0017	0.0000	0.0100	-0.0050	-0.0025	0.0125	0.0125	0.0000
		-0.0033	0.0050	0.0300	-0.0500	-0.0025	0.0025	0.0025	0.0050
γ_{xy}	3	-0.0113	-0.0121	0.0008	0.0071	0.0063	0.0017	0.0050	-0.0321
		0.0142	0.0000	-0.0050	-0.0250	-0.0275	-0.0300	-0.0325	-0.0267
		0.0100	-0.0150	-0.0050	0.0000	-0.0150	0.000	-0.0200	0.0100

TABLE A9. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 87

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0042	0.0104	0.0167	0.0146	0.0063	0.0021	0.0042	0.0042
		-0.0075	-0.0025	-0.0025	-0.0025	0.0000	0.0000	-0.0125	-0.0200
		-0.0450	-0.0400	-0.0150	-0.0200	-0.0150	-0.0050	0.0050	-0.0050
ϵ_z	2	0.0017	0.0150	-0.0125	0.0000	0.0100	0.0050	0.0000	0.0017
		0.0017	0.0100	-0.0125	-0.0050	0.0150	0.0175	-0.0075	0.0017
		0.0017	-0.0075	0.0000	0.0100	0.0300	0.0050	-0.0075	0.0050
γ_{xy}	3	-0.0088	-0.0008	-0.0163	0.0092	0.0033	-0.0125	-0.0042	-0.0375
		-0.0025	0.0100	0.0175	-0.0025	0.0075	-0.0275	0.0150	-0.0292
		-0.0067	-0.0200	0.0250	0.0000	0.0050	-0.0175	0.0375	0.0133

TABLE A10. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 88

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	-0.0104	0.0021	0.0083	0.0021	-0.0063	-0.0063	0.0042	0.0104
		-0.0225	-0.0450	-0.0450	-0.0425	-0.0375	-0.0275	-0.0400	-0.0325
		-0.0450	-0.0250	0.0000	0.0150	0.0050	0.0000	0.0050	-0.0250
ϵ_z	2	-0.0083	-0.0025	0.0125	-0.0100	0.0150	-0.0025	0.0175	-0.0050
		-0.0100	-0.0025	0.0025	-0.0050	0.0200	0.0025	0.0150	-0.0033
		0.0033	0.0000	-0.0075	0.0300	0.0250	0.0000	0.0075	0.0033
γ_{xy}	3	-0.0488	-0.0213	-0.0246	0.0108	-0.0129	-0.0146	-0.0154	-0.0171
		-0.0200	-0.0200	-0.0300	0.0075	-0.0625	-0.0550	-0.0350	-0.0733
		-0.0083	-0.0275	-0.0400	0.0450	0.0550	-0.0400	0.0075	-0.0217

TABLE A11. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 92

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0030	0.0000	0.0050	-0.0200	0.0015	0.0050	0.0175	0.0000
		-0.0015	0.0000	0.0025	-0.0200	0.0200	0.0100	-0.0175	0.0020
		-0.0030	0.0000	0.0000	0.0000	0.0150	0.0050	0.0025	0.0035
ϵ_z	2	0.0020	-0.0060	-0.0060	-0.0060	-0.0105	0.0000	0.0105	0.0060
		0.0075	0.0050	0.0050	0.0100	0.0075	0.0100	0.0025	0.0025
		-0.0300	0.0000	0.0200	-0.0100	0.0000	0.0200	0.0125	-0.0100
γ_{xy}	3	-0.0010	0.0085	-0.0035	0.0185	0.0330	0.0065	-0.0190	0.0050
		-0.0110	0.0100	-0.0025	-0.0250	0.0000	-0.0300	-0.0230	-0.0100
		-0.0320	0.0110	-0.0100	0.0600	0.0350	0.0150	0.0025	0.0000

TABLE A12. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 93

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0023	-0.0043	0.0020	0.0020	0.0000	0.0000	0.0000	0.0000
		0.0023	0.0050	0.0000	0.0050	0.0050	0.0000	0.0000	0.0025
		0.0050	0.0150	-0.0100	-0.0300	-0.0200	-0.0050	-0.0100	0.0150
ϵ_z	2	-0.0115	0.0000	-0.0075	-0.0050	0.0100	-0.0100	0.0100	-0.0150
		-0.0150	-0.0050	-0.0050	-0.0200	0.0300	-0.0150	0.0150	-0.0250
		0.0000	0.0100	-0.0025	0.0150	0.0300	-0.0075	0.0025	-0.0023
γ_{xy}	3	-0.0268	0.0022	0.0035	0.0013	0.0163	-0.0018	0.0012	-0.0194
		0.0400	0.0275	0.0075	-0.0025	-0.0150	-0.0250	0.0200	-0.0290
		0.0250	0.0100	-0.0075	-0.0050	0.0150	0.0225	0.0175	-0.0010

TABLE A13. MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 94

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	0.0021	-0.0021	0.0063	0.0042	-0.0021	-0.0021	0.0063	0.0148
		0.0000	0.0000	0.0050	0.0150	0.0200	-0.0125	0.0025	0.0075
		-0.0450	-0.0400	-0.0550	-0.0650	-0.0500	-0.0250	-0.0150	-0.0150
ϵ_z	2	0.0033	0.0075	-0.0025	-0.0200	0.0100	0.0000	0.0175	0.0083
		0.0083	0.0100	0.0000	-0.0050	0.0200	0.0100	-0.0050	0.0100
		0.0100	0.0025	-0.0075	-0.0100	0.0400	0.0075	-0.0100	0.0233
γ_{xy}	3	0.0058	0.0096	-0.0042	0.0017	-0.0075	-0.0083	-0.0071	-0.0071
		0.0075	-0.0025	-0.0075	0.0050	-0.0150	-0.0050	-0.0050	-0.0342
		-0.0183	0.0000	0.0075	0.0600	0.0500	0.0625	0.0325	0.0233

TABLE A14 MOBILIZED STRAIN IN THE SOIL IN SHEAR TEST 96

Strain	Vertical Position	Horizontal Positions							
		1	2	3	4	5	6	7	8
ϵ_x	1	-0.0083	-0.0104	0.0000	0.0000	-0.0083	-0.0083	-0.0063	-0.0063
		-0.0025	0.0025	0.0050	0.0050	0.0075	-0.0025	-0.0025	0.0025
		-0.0050	0.0000	-0.0100	-0.0250	-0.0500	-0.0050	-0.0250	-0.0200
ϵ_z	2	0.0000	0.0025	-0.0025	0.0050	0.0100	0.0075	-0.0025	0.0000
		0.0033	0.0025	0.0000	-0.0050	0.0300	0.0100	-0.0075	-0.0017
		0.0000	0.0000	-0.0025	0.0050	0.0300	0.0150	-0.0050	-0.0033
γ_{xy}	3	-0.0025	0.0096	-0.0129	0.0021	0.0000	-0.0113	0.0042	-0.0075
		0.0017	-0.0075	-0.0125	-0.0050	0.0000	-0.0100	-0.0050	-0.0193
		-0.0017	0.0000	0.0000	-0.0250	0.0500	0.0050	-0.0375	-0.0167

TABLE A15 SUMMARY OF THE SPECIAL TESTS

Test Series	Soil Type	Geogrid	Geogrid Orientation (θ)	Surcharge (kPa)
30 31 32	Coarse Sand	Tensar UX 1500	$\theta=90^\circ$	23 28 35
40 41 42	Coarse Sand	NA	NA	23 28 35
53 54	Crushed Limestone Aggregate	NA	NA	23 35
87 88	Crushed Limestone Aggregate	Conwed 5033	$\theta=90^\circ$	23 35
92 93	Crushed Limestone Aggregate	Tensar UX 1500	$\theta=90^\circ$	23 35
94 96	Crushed Limestone Aggregate	NA	NA	23 35

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