A NEW SOIL STABILISATION TECHNIQUE

by

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This work is dedicated to my Father who as a

Civil Engineer will appreciate its context.

ABSTRACT

This work describes a novel method of soil stabilisation at meso-scale which involves mixing into the soil molecularly oriented mesh elements in the form of squares, rectangles or ribbons. Laboratory compaction, CBR, permeability, triaxial, repeated loading, long term loading (creep) and model footing tests are detailed in which 50 mm square, or 50 x 100 mm rectangular mesh elements are mixed with various soils in order to identify the important properties of the mesh and the effect of the mesh element content on the behaviour of the stabilised soils. The results indicate that the basic operating mechanism is that each mesh interlocks with the adjacent soil particles to form an aggregation and these aggregations are locked together by the surrounding mesh elements to form a coherent matrix with improved stress resistant properties, increased ductility and unaffected permeability. These benefits are obtained even when the mesh element content is small.

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The following is a list of the more important symbols used in the text.

A	Area
B	Width
C.B.R.	California bearing ratio
C_v	Coefficient of consolidaton
CM_1, CM_2	Various methods and levels of compaction
D	Diameter, Depth factor, Stress intensity
D_{80}	Effective particle size
E	Modulus of elasticity
F	Force, Factor of safety
G	Particles' specific gravity
H	Height, Thickness
H_w	Vertical height of ground-water table
I	Index
I_r	Improvement ratio at residual stress state

I_p	Improvement ratio at peak stress state
K	Factor ratio, constant
K_1, K_2	Dimensionless parameters, constants
K_a	Coefficient of active earth pressure
K_p	Coefficient of passive earth pressure
L	Length
L.L.	Liquid Limit
M	Moment
M_r	Resilient Modulus
N_c, N_q, N_γ	Bearing capacity coefficients
P.I.	Plasticity index
P.L.	Plastic limit
Q	Total quantity of flow in time t
R	Radius, Weight of roots
R_1, R_2, R_3	Electrical resistances 1, 2 and 3
R_t	Tensile strength of polymeric thread
R.D.	Relative density

S	Degree of saturation, Total shear strength of plant-roots, Section of polymeric thread
T	Time factor
T_s	Shear strength per unit area of slope surface due to wind on trees
U	Average degree of consolidation
V	Volume, Voltage
V_a	Volume of air
V_w	Volume of water
V_{AB}	Voltage between AB
W	Weight
W_w	Weight of water
Z	Section modulus, Depth
a	Area, Dimensionless parameter accounting for root reinforcement, Angle
b	Shear stress intercept due to presence of polymer, Dimensionless parameter accounting for vertical slope surcharge

c	Cohesion with respect to total stress
c'	Cohesion with respect to effective stress
d	Depth, Dimensionless parameter accounting for soil moisture state
d_s	Deviator stress
e	Void ratio, Eccentricity, Dimensionless parameter accounting for physical properties of soil.
f	Dimensionless parameter accounting for water level
g	Gravitational acceleration
h	Hydrostatic head, Dimensionless parameter accounting for physical properties of soil.
i	Hydraulic gradient
j	Dimensionless parameter which accounts for wind in the trees
k	Coefficient of permeability
l	Length
m	Moisture content, slope of a straight line
m_v	Coefficient of volume compressibility
n	Porosity

p	Pressure
p_a	Active earth pressure
p_p	Passive earth pressure
p_c	Preconsolidaton pressure
p_o	Overburden pressure
q	Unit quantity of flow, Ultimate load
q_o	Vertical tree surcharge
r	Radius
t	Time, Thickness
u	Pore water pressure
w	Fibre concentration by soil weight
x	Horizontal distance
y	Horizontal, vertical distance
z	Depth
α	Angle
β	Angle
γ	Unit weight,

γ_t	Volumetric weight of geotextiles
δ	Settlement, Penetration
ε	Strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	Principal strains
ε_a	Axial strain
η	Performance efficiency, Dynamic viscosity
θ	Angle
λ	Angle, Strain gauge factor
μ	Angle, Coefficient, One micron, Quanitity of geotextile per soil weight
ν	Poisson's ratio
ρ	Density, Electrical resistivity
ρ_d	Dry density of soil
ρ_{sat}	Saturated density of soil
ρ_{bulk}	Bulk density of soil
ρ_w	Density of water
σ'	Effective stress
σ_N, or σ_n	Normal stress

$\sigma_1, \sigma_2, \sigma_3$	Major, intermediate and minor principal stresses
τ	Shear stress
ϕ	Angle of internal friction, with respect to total stress
ϕ'	Angle of internal friction, with respect to effective stress
Ω	Diameter of fibres
$\Delta\ell$	Change in length l
ΔV	Change in volume V
ΔR	Change in electrical resistance
$\Delta\tau$	Shear stress increment
$(\sigma_1 - \sigma_3)$	Deviator stress

CHAPTER ONE

INTRODUCTION

CHAPTER ONE
INTRODUCTION

1.1 GENERAL

Natural soil is both a complex and variable material. The alteration of its properties to meet specific engineering requirements is known as SOIL STABILISATION. The chief properties of a soil with which the construction engineer is concerned are volume stability, permeability, durability and strength.

There are chemical, thermal, mechanical and other forms of soil stabilisation acting at three different scalar levels, (macro, meso and micro), as indicated in Figure 1.1. It must be realised, however, that because of the great variability of soils no one method is ever successful in more than a limited number of soils (see Figure 1.2).

Piling, Reinforced Earth or ground anchors, can be considered to be forms of soil stabilisation on a MACRO-SCALE. So are steel wire mesh, stone, gravel and straw when mixed with soil and this latter material has been used since Biblical times to strengthen structures. In China mattresses made of wood branches have been placed in soil to form dikes and revetments along the Yellow River for more than a thousand years. The California Division of Highways have for many years utilised Redwood logs as reinforcing elements to build highway embankments. Similarly roads have been constructed on bamboo fascines, logs or timber, or on bush, or small trees for many centuries.

It has been almost twenty years since Henri Vidal, the French architect and engineer developed the first commercial use of Reinforced Earth (i.e. the use of steel strips to reinforce soil) and since then a large number of structures have been completed. Some of these structures were constructed on sites having relatively poor foundations. This technological break-through provided foundation engineers with a new tool to improve soil properties at the macro-scale and designing the material to suit the structure rather than the standard practice of adapting structures to suit the site.

Methods such as heating, freezing or mixing with lime, cement and bitumen are commonly used to strengthen soils at the MICRO-SCALE. Their action is to bind individual soil particles together to form relatively homogeneous masses. Although these commonly used methods result in greatly increased soil strengths, they are usually associated with significant losses in ductility and permeability.

A natural process of soil strengthening at the MESO-SCALE is that developed by plant roots intertwining with soil particles. Since 1968 Kassiff and Kopelovitz at the Technion-Israel Institute of Technology have devoted a large amount of time to laboratory experiments with the purpose of determining the strength parameters of this composite material and their effect on the resistance to cutting tools. Gray (1973) recognised that a root system provides mechanical reinforcement to the soil, although the role of the tree root strength in the stability of slopes has been the subject of speculation and experimentation for several years, and Waldron (1977) suggested that the mechanical stabilisation of soil on slopes can be attributed to plant roots. At the same time Ziemer and Swanston (1977), studied the root strength changes after logging, in Southeast Alaska. They found that a crucial factor in the stability of steep forested slopes, is the role of plant roots in maintaining the shear strength of soil mantles. Roots add strength to the soil by vertically anchoring through the soil mass tying the slope together across zones of weakness. Once the covering vegetation is removed, these roots deteriorate and much of the soil strength is lost. Similarly Burroughs and Thomas (1977) studied the declining root strength in Douglas-fir after felling, as a factor in slope failure.

Other engineers have attempted to simulate the results of this natural process of soil strengthening at the meso-scale by plant roots. Thus man-made fibres and other materials "come to mind" to replace the plant roots. Hoare (1979) used randomly oriented discrete fibres as a soil-inclusion, to improve the soil's properties. He discovered considerable increases in both strength and ductility of the soil, provided that sufficiently heavy compaction was applied. Andersland and Khattak (1979) obtained an increase in the modulus of elasticity and the shear strength parameter of clay ~~be~~ by mixing pulp fibres into Kalonite. Leflaive (1982) also examined the behaviour of a granular soil reinforced by a continuous textile polymer thread injected into its mass by a jet of compressed air. The soil's bearing capacity showed a considerable improvement. The granular material mixed with the polymer showed an apparent "cohesional behaviour" due to the introduction of polymer in it. Hausmann (1978) had previously called this pseudo-cohesion.

All these researchers investigating the meso-scale strengthening effects of the various man-made soil-improving materials, have shown that fibres like roots, develop tensile stresses when the soil is strained and so act as tension resistant inclusion. This action depends principally upon surface friction between the roots or fibres, and the soil particles, but there can also be a measure of interlock occurring when the roots or fibres, are long and are present in large proportions.

1.2 A PROPOSED NEW METHOD FOR SOIL STABILISATION AT MESO-SCALE

Recently the use of randomly distributed polymeric mesh elements in soils has been advocated by Mercer et al (1984). (Figure 1.3 shows a typical polymeric mesh element structure). They have shown that these meshes interlock with the soil particles and produce a strengthening at the meso-scale. As with the root or fibre soil strengthening, the ductility and permeability of the soil are not reduced, and a relatively homogeneous composite is produced. The principal difference between the techniques, is the predominance of the interlock action when mesh elements are employed. This occurs at two levels, with the ribs of individual mesh elements interlocking with groups of soil particles to form an aggregation of particles, then adjacent aggregations interlocking to form a coherent matrix. (See Figures 1.4 a & b and 1.5). In a similar way to conventional soil-stabilisation techniques, the mixing, batching and generally the handling of polymeric mesh elements can be done using conventional engineering plant, such as concrete mixers or rotavators, with much less overall cost involved, McGown et al (1986).

In order to investigate the behaviour of the proposed composite material many parameters had to be considered;

(a) MESH TYPE

A very wide range of mesh types are available. To establish the most efficient form of soil stabilisation, twelve types of mesh elements were examined differing in grid hole opening size, flexural (or bending) stiffness, polymer tensile strength, weight per unit area and interface properties. The grid size and interface properties are thought to play an important role in the soil and mesh element interlocking mechanism and the flexural stiffness affects the ability of the polymer to bend, or wrap itself, round the soil mass forming a dense aggregation. The weight of the polymer is also an important factor governing the economy of the total composite structure and the polymer strength governs the final overall strength of the composite mass.

(b) TYPE OF SOIL

The range of soils that can be mixed with mesh elements is obviously very important It should include both granular and cohesive types. Very little work has, however, been done in this project with mesh elements mixed with cohesive soils. The only work of this kind is mentioned in APPENDIX E. This was due to the restrictions of time and apart from that, to ease the initial experimental problems associated with the project. The granular soils selected were chosen to cover many areas of geographical occurrence in the construction world. They include a well-graded material, a very widely found soil, a uniformly-graded material which although rare type in this country, is plentiful in arid climate countries and an ideal granular soil for laboratory research work. Some other less exhaustive work was also carried out on two silty soils having a wide range of fine particles in order to examine any possible strengthening results when mixed with mesh elements. Finally two coarse grained materials were examined to further extend the test program.

(c) MESH CONTENT

Determination of the optimum amount of mesh elements mixed with the soil mass must be established in order that the behaviour of the composite material can be optimised and the economics of the process identified. The behavioural criteria are that the composite material be sufficiently strong and dense, improve its stress-strain characteristics and yet be sufficiently ductile. A range of mesh contents were tested to measure the influence of this factor.

(d) MESH ELEMENT SIZE

An optimum mesh element size has to be investigated so that there should be adequate "anchorage" between the mesh and the soil in order that the interlocking mechanism is effective. At the same time, however, the mesh element size has to be kept to a minimum for handling convenience during construction. A wide variety of sizes and shapes of mesh elements were therefore tested.

(e) MESH ORIENTATION

Another parameter to be considered was the mesh orientation. There was no doubt that maximum strength benefit would be achieved if the mesh elements were placed along the principal tensile strain axis in a manner similar to macro-scale reinforcements. Although a case like this has been examined later in this work, the mesh elements were mostly mixed randomly in the soil mass because this seemed to be the most convenient way for them to be utilised in construction works.

Preliminary work at Strathclyde University on the composite material was conducted by performing CBR tests, triaxial testing and model footing tests, in order to examine the soil and mesh element interaction mechanism and behaviour. Some of this work has been published by Mercer et al (1984), McGown et al (1985) and McGown et al (1986). This work produced results which suggest the strengthening action of the mesh elements differs significantly from that of other types of inclusions employed for soil strengthening, such as textile fibres, metal or plastic rods, as the stress transfer mechanism between the soil particles and the mesh relies upon "interlock" and not simply surface friction. Thus it appears that strengthening action of the proposed material to be investigated depends upon the following factors:

(i) Interlocking mechanism

The interlock occurs at two levels with the ribs of the individual mesh elements interlocking with groups of particles and then adjacent aggregations interlocking to form a coherent matrix. (See Figures 1.2, 1.3 and 1.4).

(ii) Polymer flexural stiffness

The flexural stiffness of every individual mesh element affects its ability to "wrap" itself around a soil mass forming a coherent matrix and therefore influences their strengthening action.

(iii) Polymer tensile strength

Mesh-elements act only in tension inside the soil mass and therefore their tensile strength capacity is an important factor that governs the overall strength of the composite material.

1.3 SCOPE AND OBJECTIVES OF PRESENT RESEARCH

The scope of this research is to investigate the following points:

1. The most appropriate type of mesh that strengthens a wide range of soils, or alternatively, establishing general design criteria of appropriate type of meshes to be mixed with certain groups of soils.

2. The most appropriate mesh size that, combines maximum strength benefit in a range of soils, with convenience in handling by construction practice.

3. The optimum mesh element content in terms of economy and

strength for use.

4. The effect of the above factors on the degree of soil improvement as far as strength, durability and permeability are concerned.

One of the main objectives of this research is to provide general design criteria based on the objectives above, in order to use mesh elements for Soil Stabilisation. In order to achieve these objectives a series of standard laboratory tests (with the exception of the model footing apparatus), had to be carried out using standard soil-laboratory equipment and techniques such as compaction and CBR tests, triaxial tests, some long term and repeated loading and also some permeability tests. Finally certain full-scale trial testing of the bearing capacity were also performed in conjunction with the Transport and Road Research Laboratory.

1.4 GENERAL LAYOUT OF THIS THESIS

The general layout of this thesis is as follows; Chapter 1 describes the concept of soil stabilisation at every scale and briefly states past and present soil stabilisation, research and techniques. Then the technique which is the scope of this project, is introduced and is compared to the existing ones.

In Chapter 2 a literature review of soil stabilisation at the MESO-SCALE is described in detail.

A selection of apparatus , materials and general equipment that is used in this project, as well as, their detailed description and all the testing procedures are described in Chapter 3.

Chapter 4 follows with a detailed discussion and analysis of the compaction and CBR test results. In this chapter the soils were firstly tested alone and their relationships between dry density and water content, as well as, CBR values and water content were established. Secondly the same soils were tested with each type of mesh element at optimum water content and various concentrations. Thus relationships between CBR values and mesh element concentration are fully examined here for each type of mesh. The mesh types that produced high CBR values were selected for further testing. Additionally the effects of mesh elements on soils' permeability are also discussed in this chapter.

In Chapter 5 of this thesis the test program, discussion and analysis of results from a large number of triaxial tests are described in detail. Tests were carried out firstly on each soil alone and secondly on every soil mixed with mesh elements at various concentrations and cell pressures. In this way the behaviour of the composite material in terms of mesh-content and stress-level is discussed. Similarly test results related to mesh-element size optimisation, mesh orientation and strain rate variation are also examined in this chapter. Finally the elastic behaviour of the composite material and its response to creep, are also analysed, by examining the results from a series of cyclic loading and long term loading tests.

Chapter 6 examines the behaviour of soil and mesh element mixtures in a plane strain model footing apparatus. A large number of tests were carried out on soils mixed with mesh elements and they were examined for load-penetration characteristics using this apparatus.

Finally general conclusions about the behaviour of the soils mixed with mesh elements at different sizes, shapes, contents etc. are fully stated in Chapter 7.

CHAPTER TWO

LITERATURE REVIEW

CHAPTER TWO
LITERATURE REVIEW

2.1 INTRODUCTION

The concept of soil reinforcement by plant roots is a form of Soil-Stabilisation at the MESO-SCALE. As mentioned previously, roots add strength to the soil by anchoring through the soil mass and interlock with soil particles forming a composite soil-root matrix. Once the covering vegetation is removed, these roots deteriorate and much of the soil strength is lost.

This "natural" form of soil stabilisation has been the subject of research for the past twenty-five years in order to establish a mathematical study and design criteria to enable scientists to control slope stability and deforestation techniques. At the same time however scientists being inspired by this natural "root-reinforcement mechanism", they started investigating the use of man-made (artificial) fibres to simulate "roots". Hence new forms of soil stabilisation at the meso-scale developed.

2.2 SOIL STABILISATION BY PLANT ROOTS

Investigators have recognised the correlation between timber cutting and increased frequency of landslides with time after logging.

Most notable was the paper by Bishop and Stevens (1964), who wrote that the number and acreage of slides in south-east Alaska increased more than 4-5 times within 10 years after logging. They attributed in the increasing frequency of landslides following logging to root deterioration, which requires several years to exert its full impact on slope stability.

Nakano, H. (1971) reporting on the results of research in Japan, showed that the resistance of stumps to uprooting decreased with years after cutting. The decreasing root strength on a unit area basis was offset by increasing resistance to uprooting of the young trees growing on the site. The net result of timber cutting was an increasing frequency of landslides.

Actual measurements of the decrease in the tree root strength following cutting are not commonly found in literature. Swanston (1969) mentions that Alaskan measurements of shear strength perpendicular to the grain of lateral roots greater than one inch in diameter showed a very gradual decrease in strength with time after cutting. Similarly O'Loughlin (1974) studied landslides and found that a high percentage of roots of all sizes along the margins of landslides failed in tension, while a smaller percentage failed in shear.

The question remains as to how to apply quantitative data to equations used in analysing slope stability once the decay function of root systems is understood. A Japanese study (Endo and Tsuruta 1969) measured the increase in soil shear strength by making large-scale direct shear tests on soil pedestals containing live tree roots. The soil shear strength increased direclty with the bulk weight of roots per unit volume of soil. Their results showed that data on the shear strength of soil with live roots fit the equaton;

$$S = \alpha + \beta R + \sigma \tan\Phi$$

where;

S = total shear strength in Kg/m^2

α and β = empirical constants

R = weight of roots in g/m^3

σ = normal stress in Kg/m^2

Φ = angle of internal friction of the soil.

Note, however, that tree root strength is considered to be a cohesive force.

Swanston (1970) made a stability analysis of three landslides in cohesionless soils in South eastern Alaska and found that an "apparent cohesion" of 3.31 to 4.26 kN per square metre was needed to maintain stability. He concluded that the most likely source for this stabilising force was the anchoring effect of tree roots growing through the slide-prone, weathered till into the compacted till. Swanston and Dyrness (1973) considered roots to "provide continuous long-fibre cohesive binders to the soil mass proper and cross local zones of weakness within the soil mass". In their opinion the anchoring effect of roots can be extremely important on some steep, shallow soils.

The effect of roots on the ultimate shear properties of a soil was also studied by Kaul (1965). Tests were made on uncompacted sandy clay loom soil having a matrix of millet roots grown in a growth chamber using a direct double shear apparatus (See Figures 2.1 and 2.2). Kaul found that the presence of roots increased the shearing and tensile strength of the soil at all moisture levels. The increase in strength, however, was found not to conform to a superimposable pattern, mainly because of extreme variation in the homogeneity of the samples tested. The values of ϕ were found to be of a larger magnitude for samples having roots as against those with no roots. This was postulated to be due to the action of rootlets making larger virtual particles out of smaller ones, with a consequent increase in the shearing angle.

Kaul's investigation was unique, but suffered form the fact that he experimented with samples influenced by too many factors which could not be controlled during testing and hence could not be separated during the analysis of the results. These factors included variations in moisture, density, root/soil area ratio, etc., which actually govern the stress-strain behaviour of this material.

Three years later Kasiff, G. and Lopelovitz, A. (1968) managed to control some of these factors and also to separate their influence on results. They similarly studied the strength properties of soil-root systems and they conducted tests on soils at predetermined moisture and density reinforced with synthetic fibres of known quality and quantity. The strength parameters were investigated with the aid of a direct double shear apparatus, and some of their results are shown on Figures 2.5 and 2.6. They concluded that the overall strength of the composite material increases with the amount of fibres (or roots) in the soil and also cohesionless soils exhibit a "high cohesion" due to the presence of roots. Later, Haussman (1978) called this "pseudo-cohesion". Fixity of fibres in a soil, which simulate deep roots relative to the plane of cutting, also increases the strength of the composite. In sandy soils, maximum resistance to shear occurs at moisture-density conditions corresponding to optimum moisture and maximum density on a compaction curve, while in cohesive soils drier than optimum conditions govern this resistance. By energy concepts (see Figure 2.6) it was shown that during the initial stage of shear failure of the composite, the roots barely contribute to the shear strength of the composite, while its ultimate shear strength is governed by the tensile strength of the fibres.

Additionally the effect of the strength parameters on the behaviour of a cutting tool was also studied by the same people. The composite material was placed inside a model, composed of a transparent box, and was subjected to the action of a model blade in the horizontal direction (see Figures 2.3 and 2.7). The shape of the failure zone and the forces acting were measured and correlated with existing theories using the strength parameters of the composite. It was found that the failure mechanism of a soil composite under the action of a cutting tool involves initial passive rupture of the soil component followed by additional ruptures resulting from the shearing resistance of the composite. The rupture surfaces were found to assume an approximate plane shape, lacking curved portions thus complying with Rankine's theory for passive earth pressure (see Figure 2.4). When a soil without fibres is stressed laterally by a thin cutting tool, the height of the zone stressed amounts to a few times the thickness of the blade. This finding applies also to reinforced cohesionless soils. However, when a reinforced cohesive soil is concerned, the fibres act as a continuous medium in all directions in front of the cutting tool, thus increasing the height of the stressed zone and the forces acting on the tool. Hence Kassiff and Kopelovitz concluded that root-cutting can be achieved efficiently only when the roots are fixed in the soil. To increase fixity of the roots it is suggested that a device should be developed which would compact the soil and cut the roots simultaneously.

Soil stabilisation became a popular topic of research for Geotechnical Engineers in the early seventies. Gray, D.H. (1970)

studying the effects of forest clear-cutting on the stability of natural slopes, recognised four ways in which vegetation affects slopes:

(1) The root system provides mechanical reinforcement to the soil; (2) Vegetation provides a vertical slope surcharge; (3) wind in the trees causes surface shears and moments; and (4) soil moisture content and water level are modified by vegetation changes.

Brown, and Sheu, (1975) continued Gray's research on the effect of deforestation on slopes. They analysed the four vegetation features affecting slope behaviour (creep and stability) proposed by Gray and they also worked out factors of safety against instability of slopes. They idealised the ground by considering it as an infinite soil slope of angle β founded on bedrock (see Figures 2.8, 2.9, 2.10, 2.11 and 2.12). Hence the following dimensionless arrangements were concluded:

$$a = \frac{\sigma_1 + \sigma_\gamma}{\gamma_w H \cos^2\beta}$$

accounts for root reinforcement of the soil

Where σ_1 is the initial tensile strength of the soil, σ_γ is a measure of the effect of the tree root system onthe cohesion of the soil, γ_w is the unit weight of water and H is vertical thickness of soil mantle

$$b = \frac{q_o}{\gamma_w H}$$

accounts for the vertical slope surcharge

where q_o is the vertical tree surcharge on surface

$$d = \frac{H_w}{H}$$

accounts for the water level and soil moisture state

where H_w is the vertical height of ground-water table above bedrock

$$e = \frac{\gamma_s}{\gamma_w}$$

Accounts for the physical properties of the soil

where γ_s is the saturated unit weight of soil

$$f = \frac{\gamma}{\gamma_w}$$

Accounts for the water level and soil moisture state

where γ is the unit weight of soil

$$h = \frac{\tan\theta_o}{\tan\beta}$$ Accounts for the physical properties of the soil

where θ_o is the separation shear angle τ_o between the rigid and creep phases

$$h' = \frac{\tan\Phi}{\tan\beta}$$ Accounts for the physical properties of the soil

where Φ is the ultimate shear angle or effective angle of soil friction.

And finally;

$$j = \frac{T_s}{\gamma_w H \sin\beta\cos\beta}$$ Accounts for the wind in the trees

where T_S is the shear per unit area of slope surface due to wind in trees.

Hence the factor of safety against instability of slope was found to be;

$$F = \frac{\text{shear capacity}}{\text{maximum applied shear}}$$

or $$F = \frac{[a+b+f(1-d)+d(e-1)]h'}{b+f(1-d)+ed+j}$$

Sudden slope failure is frequently observed at times of heavy rainfall. The worst case will be then $H_W = H$. Then ;

$$F = \frac{(a+b+e-1)h'}{b+e+j}$$

From the whole previous analysis it was concluded that; (1) The removal of the overburden of trees deceases the creep rate; (2) the cutting and removal of trees with the consequent drop of overburden and wind loading to zero increases the slope stability; (3) the decay of the root system attenuates the soil tenacity especially in soils with low cohesion and increases the creep rate and decreases the stability; and (4) the rising of the water table occasioned by the drop in evapotranspiration increases the creep rate and decreases stability.

Similarly Waldron, (1977) studied the effect of plant roots on soil shearing resitance using a direct shear device in which a prism of soil was sheared along a plane perpendicular to the axis of the prism shown on Figure 2.14.

The predetermined failure plane was selected to study root effects at different depths in a homogeneous soils or along specific layers in a stratified soil. Alfalfa roots in the homogeneous clay loam had a larger reinforcing effect than young European alder trees. The alder roots increased shoil shear resistance ΔS at the 20 cm depth, by 8.21 kN/m^2 compared to 9.81 kN/m^2 for alfalfa at 30 cm depth. The relative strength increase $\Delta S/S_f \times 100$ was far greater for alfalfa than for the alders but barley and pine roots at 30 cm depth, gave much lower ΔS values.

Root reinforced soil may be analysed, according to Waldron, as if it were a composite material in which fibres of relatively high tensile strength, are embedded in a matrix of lower tensile strength. This is the basis of the engineering technique of reinforced earth in which true cohesion is imparted to soil by linear reinforcing elements. Force is carried from point to point within the matrix by forces tangential to the fibres producing different tensions along their length. These tangential forces may be carried by friction or by bonding between the fibres and the surrounding matrix. The elements may be randomly oriented or they may be oriented in conformity with the stresses in an earth structure. Plant roots are neither randomly oriented nor placed by design to resist stresses in the soil. However, in the soil columns of the present model study the roots had general vertical orientation normal to the shearing surface. Therefore, the simple model of the root-soil system (shown in Figure 2.14) was useful in applying fibre reinforcing concepts for root reinforcing soil by treating roots as flexible elastic reinforcing elements.

A few years later Waldron, and Dakessian, (1981) produced calculations of the increased soil shear resistance ΔS (or soil reinforcement) from root properties and compared it with measured shear resistance difference between rooted and non-rooted soils (see Figure 2.17. Their model study was based on the Coulomb equation in which soil shearing resistance S is developed by cohesive and frictional forces

$$S = c + \sigma_N \tan\phi \qquad (1)$$

where σ_N is the normal stress on the shear plane, ϕ is the soil friction angle, and c is the cohesion. For a rooted soil a few assumptions were made in modifying equation (1):

(1) roots extend vertically across a horizontal shearing zone of thickness Z, as shown in Figure 2.15 and this Z does not change during shear;

(2) roots of different diameter classes are flexible and linearly elastic with Young's Modulus E;

(3) the soil friction angle ϕ, is unaffected by roots, so that Coulomb equation for root-permeated soil becomes:

$$S = C + \Delta S + \sigma_N \tan\Phi \qquad (2)$$

Hence,

$$\Delta S = \Delta S_1 + \Delta S_2$$

ΔS_1 is an increment of root reinforcement due to stretching of roots and

ΔS_2 is the increment of reinforcement due to slipping roots.

One year later Waldron, and Dakessian, (1982) studied the effect of grass, legume and tree roots on soil shearing resistance and they calculated factors of safety (see Figures 2.13 and 2.16) of shallow planar and rotary slides using measured shear strength. They showed that plant roots can make large increases in slope stability. They also modified Brown and Sheu's factor of safety which included root strength enhancement, depth of the soil to rock, depth of free water, vegetation surcharge, slope angle, soil density and wind loading via

$$F = K_1 + K_2 a$$

where K_1 and K_2 are constants and a is the dimensionless parameter defined previously by Brown and Sheu.

2.3 SOIL STABILISATION USING MAN-MADE FIBRES

The research on slope stability and the effects of deforestation on slopes still continues today in order to improve the mathematical analyses and deforestation techniques. Scientists, inspired by the idea of root-soil reinforcement, began to research on soil stabilisation at the meso-scale by using synthetic (artificial) or natural fibres to simulate the tree roots, like Kassiff and Kopelovitz (1968).

Andersland, and Khattak, (1979) examined the behaviour of soil mixtures prepared from pulp fibres and Kaolinite using the triaxial test. Dry paper pulp (cellulose) fibres with an average length of 1.6 mm were mixed randomly with dry Kaolinite in proportions of 16 and 40 per cent of fibre by weight and water was added in amounts needed to form a slurry. Thus the samples with fibre inclusions were tested in triaxial apparatus for undrained and drained conditions. A summary of their results is shown in Figures 2.18 and 2.19. They concluded that; (1) the addition of small amounts of fibre significantly increased the peak strength of Kaolinite for undrained loading conditions. Larger amounts of fibre changed the material behaviour from brittle to plastic with strength continuing to increase up to 20 + % axial strain. (2) The shear strength of the fibre Kaolinite mixtures increased with normal stress whereas the shear strength parameter ϕ' was dependent on both the fibre content of the soil mixtures, and the test procedure. (3) Consolidated-drained triaxial tests with failure based on the peak stress or stress at 20% axial strain, gave ϕ' values which increased from 20 degrees for Kaolinite alone, up to 31 degrees for the composite material. Consolidated-undrained tests with the same failure conditions, gave values ranging from 20 degrees for Kaolinite alone, to 80 degreees for the same composite material. (4) Recomputation of the safety factor for an experimental slope failure in an excavated fibrous sludge with properties similar to the fibre/Kaolonite mixtures suggests that the shear strength parameter ϕ' based on consolidated-undrained tests, is the most suitable for a stability analysis. Use of ϕ' from consolidated-drained tests gave values for the factor of safety much less than one.

In the same year Hoare, (1979) had undertaken the study aimed at determining the feasibility of using randomly oriented discrete fibres as a soil inclusion, to improve the properties of the soil. The soil used for the tests was a dry angular crushed sandy gravel and the reinforcement materials were; (1) small strips cut up into 66 x 7 mm, ICI Terram 140 (named RM1) and (2) polypropylene fibres in the form of proprietory twisted, (5 cm) chopped staple fibre (named RM2).

Various percentages by weight of these discrete fibres were mixed with the granular material and were compacted at various levels and methods of compaction (named CM1, CM2 etc) achieving different porosities. A series of triaxial tests were then performed on the composite materials and some of the results are shown in Figures 2.20 and 2.21.

Hoare's conclusions were that; (1) reinforcement provides resistance to the compaction of the soil. A positive linear relationship exists between resulting porosity and amount of reinforcement in the soil, which appears to be independent of the compaction method adopted, but dependent on the characteristics of the reinforcing material and the soil properties and their interaction. (2) Triaxial tests performed using various compaction methods showed that the reinforcement has beneficial effects on both the strength and the ductility of the soil. For samples with different amounts of reinforcement compacted by different compaction methods to constant porosity, a substantial increase in strength results. The strength increase will not be so big (it may even be negative) when a constant amount of compactive effort is applied to a range of samples with increasing amounts of reinforcement. This is due to the increases in porosity which occur with increasing reinforcement and the inherent decrease in strength which this porosity increase causes. Ramming methods of compaction appear more beneficial to strength increase than vibration methods. (3) A linear correlation exists between the amount of reinforcement and the increase in the ductility of the mixture. This is independent of the compaction method but depends on the properties of the reinforcement. Hoare also remarked that for the technique to have practical application (as for example, a mix-in-place soil stabilisation process for low cost roads), sufficiently heavy compaction would need to be used to overcome the resistance to compaction afforded by the reinforcement.

Oriented fabric layers or geotextiles are widely used in engineering practice in a variety of reinforcement applications (Giroud, 1984 - Holtz, & Broms, 1977 - McGown, & Andrawes, 1977). Reinforcement with randomly distributed, discrete fibres has attached considerable attention in concrete technology (Namaan, Moavenzadah, and McGarry, 1974). Very little information has been reported, on the other hand, about the use of this technique for reinforcing soils.

Recently Gray, and Al-Refeai, (1985) performed triaxial compression tests to compare the stress-strain response of a sand reinforced with continuous, oriented fabric layers, as opposed to randomly distributed, discrete fibres. Both natural and synthetic fibres (such as reed-fibres and glass-fibres) were used, varying from 13-38 mm in length (l) and 0.3-1.75 mm in diameter(Ω)and were mixed at various weight properties (w) with sand. For continuous, oriented fabric inclusions, fabrics such as GEOLON, TYPAR and FIBREGLASS 196 were used , cut into discs, and placed horizontally in layers, inside the triaxial specimen's sand mass. (See Figure 2.25). Some of their results are shown in Figures 2.22, 2.23, 2.24 and 2.26. They concluded that both types of reinforcement systems increased strength and modified the stress-deformation behaviour of sand in a significant manner.

1. Continuous, oriented fabric inclusions markedly increased the ultimate strength, increased the axial strain at failure, and in most cases limited reductions in post-peak loss of strength.

2. At very low strains (less than 1 per cent) fabric inclusions produced a loss in compressive stiffness of triaxial specimens.

   The loss in stiffness was more pronounced the greater the number of layers, or the higher the tensile modulus of the fabric (see Figures 2.22 and 2.25).

3. Discrete, randomly distributed fibres increased both the ultimate strength and stiffness of reinforced sand. The decrease in stiffness at low strains, observed with fabric inclusions, did not occur with the fibres.

4. The increase in strength with fibre content varied linearly up to a fibre content of 2% by weight, and thereafter approached as an assymptotic upper limit. The rate of increase was roughly proportional to the fibre aspect ratio.

5. At the same aspect ratio, confining stress, and weight fraction, "rougher", not stiffer fibres tended to be more effective in increasing strength.

6. Fibre reinforced samples failed along a classic planar shear plane whereas fabric reinforced sand failed by bulging between layers.

Soil fabric inclusions (such as geotextiles) normally used in layers, offer the undeniable advantages of good combination between the constituent polymer fibres and the soils. This use in the form of layers makes the material thus obtained strongly anisotropic, hence the idea of trying to find an "isotropic" material, reinforced in all directions. Several researchers, such as Hoare (1979), Andersland and Khattak (1979) by using fibres dispersed in the soil mass as mentioned before, applied themselves to the problem. Similarly Leflaive (1982), Leflaive, Khay, and Blivet (1983) developed a new method of soil stabilisation at the meso-scale, by reinforcing granular materials with a continuous polyester thread, TEXSOL. The composite material thus obtained from a mixture of two constituents which possess very different moduli of deformation such as sand and thread has "isotropic" properties and an overall behaviour which depends on the mechanical characteristics and relative proportions of each of the two elements.

Texsol is a new material. The sand, fed from a hopper, is transported on a continuous conveyor to a duct, ending a retrievable mould. The continuous tensile thread, collected in a container, is injected into the duct by a pneumatic tube. The sand and thread mixture falls into the mould, which is subjected to an eccentric rotation. (See Figures 2.34, 2.35 and 2.36). Leflaive et al (1983) studied the Texsol on a triaxial apparatus at various densities, using two types of sand: the Streff 0/5 mm semi-crushed and the Perche 0/2 mm rounded, and threads made of polyester consisting of 30 staples, with a diameter of 14 μ per staple. The quantities of threads used were between 1.4 and 2.0 per cent by dry sand weight. The behaviour of the Texsol is relatively complex, but four distinct actions can be schematically identified:

a. grain-to-grain friction (internal rubbing of the sand),
b. grain-to-thread friction,
c. the "loop" effect of the threads which enclose the grains (interlock),
d. entangling of the threads and thread-to-thread friction.

Some of the triaxial testing results are shown in Figures 2.27, 2.28, 2.29, 2.30 and 2.31. Finally Leflaive et al (1983) reached the following principal conclusions; (1) Texsol in general possesses shear strength greater to that of sand alone. Its modulus remains equal or superior to that of the soil but it is not certain (as Hoare has shown) that it always increases with the percentage of thread. There exists a marginal percentage, beyond which an increase in it results in a reduction of the modulus. (2) At rupture, when there is breakage of the threads, the sand is at the marginal limit and the presence of the thread is reflected in the first instance by a slight increase in the angle of friction of the sand, due to friction between the threads and to their tortuousness, but overall by the existence of an "isotropic" cohesion, proportional to the tensile strength of the thread, to the quantity of geotextile μ in the mixture, invesely proportional to the section s of the thread, and depending on the characteristics of the sand used and of the textile (See Figures 2.32, 2.33 and 2.36):

$$c = \mu \frac{\gamma}{\gamma_t} \cdot \frac{R_t}{(1+\mu\frac{\gamma}{\gamma_t})s} \cdot \frac{\sqrt{k_p}}{2}$$

$$\text{with } (1+\mu\frac{\gamma}{\gamma_t}) \sim 1$$

where:

μ = the quantity of geotextile

γ = the volumetric weight of the sand

γ_t = the volumetric weight of the geotextile

R_t = the tensile strength of the thread (resistance to rupture)

s = the section of the thread

$k_p = \tan^2(\frac{\pi}{4}+\frac{\Phi}{2})$ = the coefficient of resistivity of the sand

Φ = the angle of internal friction of soil alone.

Similarity with the cohesion of reinforced earth can be noted:

$$C = \frac{R_t}{\Delta H} \frac{\sqrt{k_p}}{2}$$

where ΔH = spacing of the discs.

(3) A study of rupture in samples of Texsol shows a resemblence to that of sand, according to whether it is loose or dense. A bulging (barrel) rupture occurs in the case of a loose Texsol and along a plane, in the case of compact Texsol. In the latter case, the sand reaches a marginal state before that of the thread.

The tests on Texsol however remain in small-scale and it would be interesting apart from studying the variations of each of the parameters within the theoretical formula, to carry out full-scale tests.

2.4 DISCUSSION

The research in plant root-soil stabilisation at meso-scale continues today to improve prediction methods for this system which as yet are problematic. The research in soil stabilisation by man-made fibres, inspired by the same idea, remains to some extent at the small-scale laboratory test stage. The works of Andersland, Khattak (1979), Hoare (1979), Leflaive, Khay, Blivet (1983), Leflaive et al ((1983), Gray (1984) including the work mentioned in this thesis, still remain within the boundaries of laboratory work. Only recently have, full scale trials been conducted on Texsol and Polymeric mesh elements (1985-86).

Thus this particular aspect of soil stabilisation is still in its infancy.

CHAPTER THREE

TEST APPARATUS, PROCEDURES AND MATERIALS USED

CHAPTER THREE

TEST APPARATUS, PROCEDURES AND MATERIALS USED

3.1 INTRODUCTION

In order to carry out the investigations, a selection of test apparatus and equipment had to be made, any existing equipment had to be serviced, calibrated and checked for accuracy and certain modifications had to be carried out. These will now all be described in this Chapter. Secondly a selection of materials had to be made in order to cover as wide a range of soils as possible. Thirdly a wide range of mesh-elements had to be selected varying in weight, shape, filament size, strength, stiffness etc., to be tested with these soils. Finally the testing procedure had to be planned accurately to save time and effort.

3.2 TEST APPARATUS

The test apparatus will be divided into two main groups; those used to test the mesh elements alone, those used to test the soils alone and the soil/mesh element mixtures.

3.2.1 Polymer Testing Equipment

The preliminary testing of the polymer meshes involved the use of the following equipment:

3.2.2 Polymer Tensile Strength Testing Apparatus

A 2 Ton, T 22 K, Electronic tensile testing machine was used. It is made by J.J. INSTRUMENTS and fitted with an accurately calibrated (according to B.S. 1610) 5 kN capacity load-cell and an attached automatic graph plotter, as shown on PLATE 1. This machine was capable of performing strain-controlled as well as stress-controlled tests, and was fitted with a special set of clamps so that the mesh samples were adequately gripped during extension. The whole testing machine was encircled by an air conditioning device capable of controlling temperatures at various fixed levels, during the polymer testing.

3.2.3 Polymer Flexural Stiffness Index Apparatus

A special device, shown on Plate 2, was made consisting of a set of metal plates acting as clamps, five centimetres wide, which were capable of gripping a long mesh strip of same the width between them and holding it in a horizontal cantilevering position. A cylindrical shaped metal weight of 110 g was connected from the centre of its mass, to the top metal plate by a 50 mm long and 50 mm wide flexible fabric belt. This belt served as means of keeping the metal (110 g) weight a fixed distance from the top clamp plate. Hence one end of the 50 mm wide mesh-strip was placed between the two clamp plates and the 110 g weight was placed on the other end of this mesh strip, at a distance fixed by the fabric belt. Thus the 110 g weight, multiplied by the 50 mm fixed distance, was capable of applying a fixed (5.93 x 10^{-10} kNm) bending moment to every type of polymer mesh-strip. Thus the bending moment difference, called the Flexural Stiffness Index, could be easily read on a calibrated glass plate, attached to the apparatus.

3.2.4 Equipment for Testing Soils and Soil-Mesh-Element Mixtures

Preliminary tests were preformed on all granular materials in order to establish their identities. A series of sieves having standard aperture openings, according to B.S. 1377 : 1975, were employed. Similarly sedimentation procedures were carried out using the hydrometer method (B.S. 1377 : 1975, Test 7) in order to establish the particle size distribution of silts. Additionally B.S. equipment was also employed for carrying out specific gravity and density tests.

3.2.5 Compaction Equipment

All compaction tests were performed using British Standard compaction equipment in a standard CBR mould, according to B.S. 1377 : 1975, Test 12. This was 6" (152 mm) diameter and 7" (177 mm) in height. The soil alone, or soil-mesh mixture, was placed into this in three equal layers and each layer was subjected to 55 blows using a 5.5 lb (2.5 Kg) hammer dropping through a height of 12" (300 mm) onto the soil. The choice of the CBR mould instead of the B.S. compaction mould was deliberate. Firstly every sample had to be tested for CBR values directly after its compaction and therefore a common mould would be convenient. Secondly, mesh elements were more appropriately accommodated (with regards to their size) in a larger mould rather than a smaller one.

A mechanical compaction apparatus shown on PLATES 3 and 5, was sometimes preferred, eliminating much of the physical effort required in carrying out the large number of compaction tests.

3.2.6 CBR Testing Equipment

The California Bearing Ratio test, or CBR, test as it is usually called, is an empirical test for estimating the bearing value of highway sub-bases and subgrades. The test first appeared in British Standards in 1953 when it was termed the "cylinder penetration test" and was used for stabilised soils. It is referred to here as Test 16, BS 1377 : 1975, the CBR test. It is the ratio of the force required to penetrate a circular piston of 1935 mm^2 (or 3 square inches) cross-section (about 2 inches diameter) into a soil in a special container at a rate of 1 mm per minute, to that required for similar penetration into a standard sample of compacted crushed rock. The ratio is determined at penetrations of 2.5 and 5.0 mm (or 0.1 and 0.2 inches) and the higher is often used. Figure 3.1 shows a diagrammatic arrangement of the CBR aparatus.

A standard ELE, CBR testing machine was used throughout this work, as shown on PLATE 4. It consists of a baseplate containing the sample located centrally on a platen. A motorised gear-box system was used which is capable of driving the platen containing the sample, vertically up (or down) at a rate of 1 mm per minute, against the standard 1935 mm^2 circular CBR piston. This piston is in turn fixed to the crosshead of the testing machine via a proving ring with a dial guage attached onto it which measures penetration. A surcharge of 2.25 Kg weight in the form of annular steel ring was placed on the top surface of the prepared specimen. This is a common practice in CBR testing since it simulates the effect of the thickness of a 7.6 centimetres road construction overlaying the layer being tested.

3.2.7 Triaxial Testing Apparatus

A standard triaxial apparatus was used consisting of a motor-geared platen capable of moving vertically upwards (or downwards) at a constant rate of speed and pushing a cylindrical shaped soil sample against a fixed end plate a constant rate of deformation. The cylindrical soil specimen is sealed in a water-tight rubber membrane having a thickness of 0.028 mm and enclosed in a cell, in which it can be subjected to fluid pressure (see Figure 3.3). A load applied axially through a ram (piston) acting on the top cap, is σ_v used to control the normal vertical (deviator) stress. Connections to the ends of the sample permit either the drainage of water and air from the voids in the soil, or alternatively, the measurement of the pore water pressure under conditions of no drainage. Ideally the triaxial test apparatus permits independent control of the three principal streses so that generalised states of stress can be examined (see Figure 3.2). Under these conditions the axial stress is the major principal stress-σ_1. The intermediate and minor principal stesses (σ_2 and σ_3 respectively) are both equal to the cell pressure. Drainage, however, was permitted throughout the triaxial testing in this project, so that full consolidation occurred under the all-round stress, and no excess pore water pressure was set up during the application of the deviator stress. A diagrammatic layout of the triaxial cell is shown in Figure 3.4.

The axial load acting through the top cap of the apparatus in this case, was applied by means of a piston having a diameter of 38 mm, and fitted with load-cell of 22.2 mm internal flange diameter and 17.7 mm external web diameter. The load cell was made out of Beryllium Copper in a cylindrical "strut" shape and designed to have similar compressive and tensile elastic behaviour. Four strain gauges were attached onto its web and electrically wired in such a way as to form a Wheastone Bridge circuit. As the piston was compressed axially, the built-in Beryllium Copper load cell, was also compressed. The strain gauges attached onto the web of the load cell altered in length and their resistivity changed, changing also the electric current that flows through them. Hence by wiring the load cell through an accurate voltmeter, a calibration was established relating voltage and load characteristics. More details about the design, shape, electrical connections and operating principles of load-cells are given in APPENDIX A.

The soil, or soil-mesh element mixture, specimens had 155 mm diameter and 200 mm height. Since this ratio of length/diameter was smaller than 2, the ends of each triaxial sample were lubricated with membranes attached to allow free-end movement during testing. A suitable former for preparing the 155 mm diameter samples was fabricated as shown on PLATE 6 and consisted of a split mould of 155.5 mm internal diameter (which enclosed the rubber membrane) and was (di apit) clamped to the sides and base parts of the triaxial cell frame.

Additonal equipment used during triaxial testing was an efficient air vacuum system consisting of an air pump and a metal vacuum cylinder. It served as a means of providing the sample with (cukup.) sufficient strength to stand, while the cell was (dipasang.) being assembled. It was also used to perform low stress triaxial tests having cell pressures of 10 kN/m^2 and 25 kN/m^2, without the use of water pressure. PLATE 7 shows the whole triaxial apparatus and sample assembly.

3.2.8 Repeated (Cyclic) Loading Test Apparatus

The same triaxial apparatus that was described previously, was employed in this case.

3.2.9 Long Term Loading (Creep) Equipment

Similarly in this case, the same triaxial apparatus described in 3.2.7 was used with the exception that the machine was fixed, i.e. the upward moving piston plate was not used. Instead of this, a horizontal metal beam was positioned centrally on the top of the cell-cap-piston that applies the deviator stress on the soil sample, and this was fitted with two metal hangers one on either end. Thus dead weights of equal magnitude , could be placed on each hanger, applying a total vertical load to the piston via the horizontal metal beam. See Plates 8 and 9.

3.2.10 Plane Strain Model Footing Test Apparatus

The test apparatus consisted of a rigid, steel-framed and glass-sided tank 640 mm long, 300 mm deep and 75 mm wide, as shown in Figure 3.5. This model tank was firmly positioned on the triaxial apparatus driving platen and was capable of moving vertically up (or down) at a constant rate of speed, by operating the triaxial machine's motor. The model footing was made out of a smooth, 20 mm thick metal plate of 75 x 75 mm size, attached centrally to a brass rod which in turn, was attached to a fixed proving ring on the crosshead of the testing machine. The footing displacements were measured by means of a dial gauge fixed onto it. The whole model tank containing the soil, or soil and mesh mixture, moved upwards at a constant speed, was restricted by the 75 x 75 mm footing-proving ring arrangement which penetrated into the soil composite at a constant rate of penetration, the penetration force being recorded by the proving ring. Between the base of the tank and the driving platen, a horizontally cantilevering prespex-glass rod was firmly fixed and held a polaroid camera at its other end. The rod-camera arrangement was also capable of moving simultaneously with the actual tank. In this way a sequence of photographs could be taken of the model's glass side displaying the soil composite. These photographs were displaying the soil composite deformation patterns during loading and could easily be examined in pairs by any stereo-photogrammetric viewer. A typical stereo-photogrammetric pair of photographs is shown in Figures 3.6 and 3.7, whereas PLATE 10 shows the model tank-footing and rod-camera arrangement.

The compaction of the soil;, or soil and mesh composite, inside the tank, was performed by 'hand tamping", using a wooden rammer 100 mm long by 75 mm wide and 50 mm thick, weighing approximately 400 gm.

3.2.11 Permeability Testing Apparatus

The permeability tests were carried out using a Oedometer consolidation cell having 152.4 mm internal diameter and 165 mm total internal height and fitted with pressurising rubber bellows, at its top, of equal internal diameter. A loading piston is connected to the bellows, passing through the top-cap plate of the cell, and touching along with the bellows, the top of the soil sample to be tested. Two perforated plates of about 20 mm thickness each, were added, one on the top of the soil sample (below the piston-rubber bellows assembly) and the other at the bottom part of the soil, making a net soil sample height of about 100 mm. The top-cap plate of the Oedometer cell is fitted with a valve where pressurised air is passed through capable of forcing the rubber bellows and piston downwards and hence applying a uniform top pressure on the fully saturated, de-aired soil/mesh element composite. Additionally flow tube connections, piezometer nipples and connection sealing rings, piezometer tubes, were all parts of the same apparatus shown diagrammatically in Figures 3.8, 3.9 and 3.10. Finally two filter discs made of wire gauge and of the same diameter as the internal cell diameter, are placed on the top and bottom of the soil sample having an aperture not greater than the D_{85} size of granular soil sample itself. Plates 12 and 11 shows the preparation and final apparatus assembly in permeability testing.

3.3 TEST PROCEDURES

The test procedures were divided into two main groups; those performed on the polymer meshes alone, and those performed on the soils alone and soils mixed with mesh elements.

3.3.1 Test Procedures in Polymer Mesh Testing

3.3.2 Polymer Tensile Strength Testing

A number of polymer-mesh samples were cut in rectangular shapes and "pulled" until failure along their shorter dimension. Each sample was strongly gripped by the special set of clamps provided, and hence positioned in the main 2 ton, 722K, Electronic tensile testing machine. The attached automatic graph-plotter was set at "zero" reading, its graph-scale was also adjusted and the whole machine was switched on. All tests were performed strain-controlled, at various rates of strain and at a constant room temperature. Each test was repeated twice or more, to check repeatability of results, for every type of mesh and each (machine or transverse machine) manufactured direction. Thus the load-extension-time characteristics and the effect of rate of applied strain in the strength behaviour of each polymer were established.

To take this work further, a selection of temperatures was produced by using the air-conditioning device mentioned previously, around the tensile testing machine. Polymer mesh samples of the same shape and size were clamped as before, and then tested at these temperatures at a constant rate of strain. Hence the load-extension-time characteristics and the effect of temperature in the strength behaviour of each polymer were established.

3.3.3 Polymer Flexural Stiffness Testing Procedure

Every polymer mesh type was cut into four strips of 50 mm width and 100 mm length. Two were cut along its machine-manufactured direction and the other two along its transverse-machine direction. Each strip was placed between the plates of the Polymer Flexural

Stiffness Index apparatus and the 110 g weight was positioned at the end of the strip, as shown on Plate 3. Thus the Flexural Stiffness Index was read on the calibrated glass plate, attached to the apparatus, as the polymer bent downwards. Every test was repeated twice and an average value was accepted between the machine and the transverse machine-manufactured direction.

3.3.4 Testing Procedures of Soils Alone and Soils Mixed With Mesh Elements

3.3.5 Mixing, Compaction and CBR Testing Procedure

The soil to be used was mixed with water at various percentages and then was placed inside the CBR mould in three layers of equal thickness, and subjected to standard compaction energy consisting of 55 blows per layer using the 5.5 lb (2.5 Kg) hammer having a drop of 12 inches (300 mm), according to BS 1377 : 1975, Test 12. Then the soil sample was weighed and its dry density was determined. The same CBR mould then was positioned on the ELE, CBR testing machin;e. A surcharge of 2.25 Kg weight in the form of annular steel ring was placed on the top of the soil's trimmed surface and the 2 in. (50 mm) diameter, or 1935 mm^2 cross sectional area, standard CBR plunger was pushed through the annular steel ring into the soil at a constant rate of 1 mm per minute.

The California Bearing Ratio was determined at penetrations of 0.1 and 0.2 inches (2.5 and 5.0 mm) from the measured forces on the proving ring:

$$CBR = \frac{\text{Measured Force}}{\text{Standard Force}} \times 100$$

The test specimen was then turned upside down, the base of the mould removed, the annular ring positioned and the test was repeated as before on the bottom of the specimen. The relationships between penetration and load applied, were plotted and corrected where necessary for bedding errors according to Figure 3.11. The CBR values were calculated for both, top and bottom of the specimen as follows:

For 0.1 inch (2.5 mm) penetration

$$CBR = \frac{\text{Test Load}}{\begin{matrix}3000\ (\text{lb}) \\ (13.3\ \text{kN})\end{matrix}} \times 100 \text{ per cent}$$

and for 0.2 inch (5 mm) penetration:

$$CBR = \frac{\text{Test Load}}{\begin{matrix}4500\ (\text{lb}) \\ (20\ \text{kN})\end{matrix}} \times 100 \text{ per cent}$$

In this way the compaction curve of each soil was established and its maximum dry density at optimum water content. Additionally the variation of CBR values with water content was also established. Afterwards the soil was mixed with each mesh-element type, at various percentages of mesh element concentration, ranging from 0% to 0.6% (by dry soil weight) and the same procedure was repeated as before.

All mixing was performed by hand using a hand trowel and a tray.

3.3.6 Triaxial Testing Procedure

To determine the fundamental stress-strain behaviour of soil-mesh mixtures, fully dry (and so drained) triaxial test specimens were prepared in the following way;

The split mould-former was assembled and firmly mounted on the triaxial cell base. The rubber membrane was stretched inside the mould's interior wall and two lubricated sliced circular membranes were stuck, one at the bottom of the mould, and the other on the top cap of the specimen, in order to provide the specimen with "free ends" during testing. An amount of dry soil alone, or dry soil and mesh elements were weighed and mixed by hand on a tray. Some of this dry soil, or soil mixture, was placed inside the split-mould-former in three equal layers and was compacted by standard compaction according to BS 1377 : 1975, Test 12 ,using the 5.5 lb (2.5 Kg) hand hammer. After compaction, the top surface of the soil (with or without mesh elements) was trimmed, levelled and capped. The remaining soil on the tray was weighted and an assessment of the sample's dry density was made. In many instances two membranes had to be used in order to avoid any piercing of holes, by the sharp edges of the mesh elements, in the thin (0.028 mm) membrane.

The sealed and capped specimen was placed on the platen of the triaxial apparatus and suction by means of an air pump was applied to give the specimen sufficient strength to stand, while the cell was assembled. The low-stress triaxial tests, having cell pressures of

10 kN/m^2 and 25 kN/m^2, were performed without the use of water pressure; vacuum was used instead, simulating the radial pressure σ_2. This pressure was controlled using an air-bleeding valve in the vacuum cylinder and two mercury manometers, one attached at the top of the triaxial sample, and the other at the bottom. Both manometers measured the average low radial pressure σ_2 on the sample.

3.3.7 Repeated Loading Test Procedure

The same triaxial apparatus was used in this test series as was used in the (Repeated Cyclic) loading tests. The whole preparation;, therefore, in mixing, compaction, sealing of the samples containing the soil;, or soil/mesh composite, as well as, assembling the triaxial cell, was identical to the previous case (Section 3.3.6). These tests were "stress-controlled" and therefore the triaxial apparatus had to be switched from the "load" position to the "unload" positon when the stresses reached the prescribed maximum and minimum levels. Thus the whole operation required two operators, one to control the machine and the other to monitor the stress levels by calculating the piston force and the corrected sample area, at any appropriate deformation. A series of tests were carried out on each granular soil along and the same number of tests were repeated for the same soils mixed with mesh elements. The number of loading-unloading cycles performed was restricted to six for convenience. The rate of strain was fixed at 0.05%/minute and all the tests were carried out fully drained (dry).

3.3.8 Long Term Loading Test Procedure

In the case of long term (creep) loading, the soil and soil/mesh element composite samples were prepared, mixed, compacted and sealed in an identical way to triaxial testing, since the same apparatus was used in this case. Similarly all tests were performed fully drained (dry). The composite samples were incrementally loaded by "dead" weights which were placed on each end of the hanger. The incremental loading procedure was completed in twenty four hours for each test. It was thought that this was sufficient time to allow for consolidation of the dry composites. Afterwards, the "dead" weights were left for about forty to fifty days for each test;, and a reading on the deformation gauge was taken once a day.

3.3.9 Model Footing Tests

The soil was placed inside the model tank in thin layers of 25.5 mm each using a small hand shovel. Each layer was compacted by hand using the 100 x 75 mm compaction area of 50 mm thickness wood-rammer, and tamped 20 times throughout its surface area. The mesh elements were randomly distributed inside the model tank being mixed randomly with the soil with which it was being placed. The mesh element concentration was always kept constant at 66 m^2/m^3 of dry soil, throughout this testing. Mesh elements were mixed with the soil in layers of (1/2)B, B, 2B, 3B and 4B (where B was the width of

the footing).
The compactive effort adopted throughout this testing was standard but arbitrary in character. The densities achieved however, were not far from those obtained in the CBR and triaxial testing where B.S. compaction took place.

When the tank was full of soil mixed with the mesh elements, it was elevated until the model footing fixed to the proving ring, touched its surface. Then a photograph was taken by the camera attached to the tank. Then the testing began and the footing started penetrating the soil mass (with or without mesh elements) at a standard (convenient) rate of penetration of 1 mm per minute. At various increments of penetration, photographs were taken until a maximum penetration was reached. The forces required to obtain these penetrations were recorded by the proving ring, whereas the net penetration was registered by a dial gauge attached to the apparatus. When maximum penetration was reached the machine was switched onto the "unload" position and the model tank began lowering itself at the same rate of 1 mm per minute. In this way the force-penetration-recovery was also recorded.

At the end of each test the whole apparatus carrying the soil composite, was weighed on a reasonably accurate large weighing scale and an assessment of bulk density was made.

3.3.10 Permeability Testing Procedure

For soils of high permeability constant head permeability testing took place with flow running upwards, as shown diagrammaticaly in Figure 3.9. The soil was placed inside the Oedometer cell fitted with the perforated plate and the filter disc, and was compacted in three equal layers according to BS 1377 : 1975 Test 12, having the blows adjusted and using the 5.5 lb (2.5 Kg) hammer with a drop of 12 inches (300 mm). The top filter disc and perforated plate were then positioned and the cell was capped and sealed. All tube atachments were then made to the water reservoirs and a vacuum was applied by means of an air pump, through the top cap of the cell in order to de-air the soil and tube system. When water began to flow through the pump the vacuum was disconnected. At this stage, the soil sample was fully saturated. After allowing a few hours for the water to reach a steady state, testing began.

Tests were carried out at various hydraulic heads and the water discharge was measured at fixed time increments using a measuring container and a stop clock. Six readings were taken for every change of piezometric head difference (between the reservoirs) and an average was accepted. The range of confining pressures tested in the cell were; 0 kN/m^2, 25 kN/m^2, 50 kN/m^2, 150 kN/m^2 and 300 kN/m^2. These pressures were obtained by compressed air supplied inside the rubber bellows of the cell. Similarly readings of flows were taken at every confining pressure for every piezometric difference. The whole procedure was repeated for each soil mixed with mesh elements. The hand-mixing, compaction, sealing and connecting to the water reservoirs was made in identical way to the case when soil was tested alone.

For soils of low permeability the same cell was used. The soil alone, (or mixed with mesh elements) was placed and compacted inside the cell the same way as before. All tubes were removed from the cell and connection valves were shut with only one vertical pipe attached to the cell's top cap, and an outlet pipe at its bottom. Water was supplied through the top pipe flowing through the mass of the fully saturated sample, and was discharged through the outlet pipe. Hence the above apparatus was converted into a Falling Head Permeameter, as shown in Figure 3.10. The same range of confining pressures were also selected in this case and a number of tests were performed for each one of them at different piezometric differences.

3.4 MESH TYPES

3.4.1 General

The meshes used in this study were manufactured from poly-propylene using the NETLON extrusion process. This process produced integrally extruded mesh. Polymer is extruded from two (or more) counter rotating at the same speed, cylindrical concentric dies with slots cut in the matting surfaces. As the matting surfaces move past each other, the slots move in and out of register. When two slots move into register an integrally extruded joint is formed. Moving out of register the slots produce two separate filaments. Thus a tube of net is built up as shown in Figure 3.12.

To make the cast net of mesh elements, the outer die is fixed, and the inner die is rotated and the resulting tube is cut on a 45^0 helix to give a flat sheet with square meshes. The cast sheet is biaxially oriented to increase the tensile strength of the filaments, and is finally slit and cut to produce mesh elements. (See Figure 3.13).

The polymer itself has been tested by the manufacturers against chemical attack in order to simulate the worst possible acidic ground. It was found very resistant (ICI 1971). Additionally, being made out of polypropylene, it has a fair resistance to dry heat, moist heat, insects and vermin, (Cannon 1976). The effects of ultra violet radiation from the sun on the polymer itself, can be very destructive. This will be eliminated however when the mesh elements are buried in the ground. Thus site storage under the sun should be restricted to a few days.

3.4.2 Types of Mesh Elements

Twelve types of mesh elements were supplied in total, by the manufacturers for testing. They varied in mesh pitch size, interface properties, weight per unit mesh area, flexural stiffness and tensile strength. The various types of mesh elements are shown in Figure 3.14, whereas Tables 3.1 and 3.2 set out their various properties as identified in Figure 3.15.

The first three types; 5345, 5340 and 8630 were the first that were available for testing. While they were being tested, Types, 1, 2, 3, 4, 5 and 6 arrived from the manufacturers for similar analysis. Type 5345 produced the highest CBR values amongst all those above eleven types, and therefore it was requested that more of this type (5345) to be provided in order to proceed with further testing. Netlon Ltd. however supplied Type 7 which they stated, was an improved version of Type 5345, in terms of interface area and strength. Thus the Type 7 mesh element was established as the "standard" polymeric mesh element in this study.

Type 10 arrived much later during this project as a substitute for Type 7, since the manufacturers were researching for larger interface area by keeping (almost) the same weight per unit area, and enlarging the mesh pitch size. This Type 10 is now the commercial form of Netlon's mesh element. The Type 7 mesh element however, was mainly adopted for all types of testing in this work since it proved to be superior when mixed with the soils available, in terms of strength and interlocking capacity.

3.4.3 Polymer Mechanical Properties

a. STRENGTH-STRAIN BEHAVIOUR AT VARIOUS RATES OF STRAIN AND AT CONSTANT TEMPERATURE

A number of polymer mesh samples were cut to 100 x 200 mm and pulled along their 100 mm dimension until failure. Half of the samples were tested having their 100 mm dimension along the machine-manufactured direction and the remaining along the transverse machine direction. They were all tested in the 2 ton, T 22 K, Electronic tensile testing machine, strain-controlled, for tensile strength behaviour at various rates of strain and at a constant room temperature of 20^0 C. The range of rates of strain selected for testing were;

15%/minute, 2%/minute, 0.5%/minute, 0.045%/minute and 0.005%/minute.

The scope of this investigation was firstly to assess an average tensile strength between the machine direction and the transverse machine direction of the appropriate polymer. Secondly to investigate the effect of rate of applied strain in the strength behaviour of the mesh, at constant temperature. Only the mesh types that gave high CBR values were tested in these ways. The other types were omitted after they produced inferior results in the CBR testing. It may be assumed, however, that they would behave with respect to rate of strain and temperature the same way as the tested ones.

Figure 3.16 shows the load-extension-time behaviour pattern of mesh Type 7 at the above stated rates of strain. Similarly Figure 3.19 shows a similar to Type 7, load-extension-time behaviour pattern of mesh Type 10, at a different range of rates of strain. From these figures it could be concluded that the polymer behaviour shows high maximum strengths at high rates of strain, but low failure strains. At low rates of strain, much larger failure strains take place and the strength capacities are smaller:

(b) <u>STRENGTH-STRAIN BEHAVIOUR AT VARIOUS TEMPERATURES AND AT CONSTANT RATE OF STRAIN</u>

Similarly a number of polymer mesh samples were cut and tested in a similar way as before, at a fixed rate of strain of 2%/minute and at various temperatures. The range of temperatures were;

5^oC, 10^oC, 20^oC and 40^oC

Similarly in this case only the mesh types that gave high CBR values were tested in these ways;, such as Type 7. It may be assumed, however, that all the other types would behave with respect to temperature, the same way as the tested one.

The scope of this investigation was to examine the polymer's strength behaviour at different temperatures, at the same rate of applied strain. Figure 3.17 shows the behaviour of mesh Type 7 at the stated range of temperatures, at 2%/minute, constant rate of strain. It may be observed here that high temperatures make the polymer softer and increase its ductility causing an overall reduction in its strength, and increase in extensibility. The opposite occurs a low temperatures.

(c) STRENGTH-STRAIN BEHAVIOUR AT AN INDEX CONDITION

All twelve types of polymer-meshes were tested in this case the same way as previously. Each type of mesh was tested in both (machine, transverse machine) directions, at constant rate of strain of 2%/minute and at room temperature of about 20^{0}C. The scope of this investigation was to assess the maximum tensile strength of each type of mesh element individually. The results of this work are shown on Table 3.2 and Figure 3.18. To take this work further all twelve types of mesh elements were tested for flexural stiffness index and the results are also shown in Table 3.2.

3.5 TYPES OF SOILS USED

Seven different soils were considered throughout this project; six granular and one cohesive. A well graded gravelly sand was thoroughly tested with every type of mesh element. A uniformly graded sand was tested thoroughly with Type 7 mesh, whereas the remaining granular soils were tested in a more restricted manner with Type 7 mesh elements. A very limited study was also made on a cohesive soil using this same mesh.

The cohesionless materials were carefully chosen to cover a wide range of granular soils occurring in nature. They were also chosen to have no cementing properties since they were going to be mixed, and tested with a new soil-stabilising material and any kind of cementing properties in the soil might confuse the results of this research.

3.5.1 Mid-Ross Sand

The sand used here was a locally available processed fluvio-glacial soil having a mixed composition with larger fraction consisting of Highland schists, vien quartz and dolerite. It is known as Mid-Ross sand and often used in concrete making. Mid-Ross sand has been used in Strathclyde University as standard granular soil for calibration testing and research. Plate 13 shows a magnification of the sand's particles. It has sub-angular particles ranging in size from 0.05 mm to 7.0 mm diameter with a uniformity coefficient of 6. The particle size distribution of this sand is shown in Figure 3.20. Its specific gravity was found to be 2.69.

This sand was compacted by standard (BS) compaction energy (using the CBR mould) according to BS 1377 : 1975, Test 12, at various percentages of water content. Its maximum dry density for this level and method of compaction, was approximately 1818 Kg/m^3 at an optimum water content of about 7.5% as shown on Figure 3.26. Its variation of CBR values with water content are also shown on Figure 3.27. For this level and method of compaction Mid-Ross sand was found to be at a dense state having a relative density of 78% and an average porosity of 32% with a void ratio of about 48%. Its maximum achieved dry density was about 1886 Kg/m^3 and its minimum 1620 Kg/m^3. The tests employed to locate these values were performed according to Kolbuszewski's (1948) procedures. These were; the BS 1377 : 1975, Test 14, using the vibrating hammer method with the soil submerged under the water for determination of maximum dry density and also the "standard" Kolbuszewski test for minimum dry density consisting of the "shaking" of a cylinder containing the dry sand sample. (Head, 1981, page 138).

A moisture content of 9.3 per cent was chosen for use in all subsequent compaction and CBR tests on Mid-Ross sand alone and Mid-Ross sand-Mesh mixtures. This was chosen at it ensured that the air void space available for occupation by the mesh filaments, without causing an increase in void space was at a minimum. Thus tests at this moisture content provided a "worst case" condition.

The mean CBR values of the Mid-Ross sand at 9.3 per cent water content after a series of repetitive tests were;

At 0.1" (or 2.5 mm) penetration

2.3 ± 0.9% TOP
3.6 ± 0.8% BOTTOM

At 0.2" (or 5.0 mm) penetration

3.1 ± 0.8% TOP
4.0 ± 0.4% BOTTOM

3.5.2 Leighton Buzzard Sand

The material used here was Leighton Buzzard sand which is a processed alluvial material consisting mainly of quartz. This is a uniform sand having rounded particles (as shown on Plate 14) ranging from 2.0 to 0.03 mm. Its uniformity coefficient is 1.2 and its particle size distribution curve is shown on Figure 3.21. Additionally its specific gravity was found to be 2.65. Leighton Buzzard sand has been used by many soil laboratories in research and calibration testing.

This sand was similarly compacted according to BS 1377 : 1975, Test 12 (using the CBR mould) at various percentages of water content and its compaction curve is shown on Figure 3.28. This curve indicates that at standard (BS) compaction energy, the optimum moisture content which produces maximum dry density for this uniform sand, is very small. Thus due to the undefinable nature of this uniform sand's compacton behaviour, a value of zero% optimum moisture content was accepted for convenience. This behaviour is quite normal for certain types of smooth, fine uniform sands, or clays having very high or very low plasticities, according to Lee and Suedkamp (1972), Transportation Research Record No. 381 (see Figure 3.29). The degrees of saturation against void ratio and water content of this sand are also shown in Figure 3.30. Leighton Buzzard sand's maximum average dry density at BS compaction was found to be 1618 Kg/m^3 having a void ratio of 63.7 per cent and a porosity of 38.8 per cent. For this type and amount of compactive effort, Leighton Buzzard sand was found to be at a medium dense state having a relative density of 53 per cent. Its maximum and minimum dry densities were obtained using Kolbuszewski's procedures as before. The maximum dry density was 1780 Kg/m^3 and the minimum 1465 Kg/m^3.

The mean CBR values of Leighton Buzzard sand after a series of repetitive tests at zero per cent water content were;

At 0.1" (or 2.5 mm) penetration

3.50% TOP

6.03% BOTTOM

At 0.2" (or 5.0 mm) penetration

3.40% TOP

4.80% BOTTOM

3.5.3 West Highland Moraine

This is a silty soil of glacial and detrital (psammitic) origin with well-rounded (windblown) grains showing a frosted surface (See Plate 15) and ranging from 2.0 to 0.02 mm. Its main composition is quartz with small amounts of feldspar. This soil's origin is Argyllshire (Scotland) and is called West Highland Moraine. It has a particle size distribution curve shown in Figure 3.22, and a uniformity coefficient of 2.93. Its specific gravity was found to be 2.68. West Highland Moraine like many other silts, being a "marginal" soil and susceptible to moisture content, has been widely tested by many researchers (McGown et al 1975)

This soil was also compacted by standard (BS) compaction (using the CBR mould) according to BS 1377 : 1975 Test 12, at various percentages of water content. Its maximum dry density for this level and method of compaction was found to be 1740 Kg /m^3, having a porosity of 36 per cent and a void ratio of 56 per cent and an optimum water content of 14 per cent. Figure 3.31 shows the compaction curve of this soil.

When West Highland Moraine was tested in CBR, at 14 per cent water content its mean CBR values were;

<u>At 0.1" (or 2.5 mm) penetration</u>

6.0% TOP

11.3% BOTTOM

<u>At 0.2" (or 5.0 mm) penetration</u>

7.0% TOP

12.5% BOTTOM

3.5.4 <u>Colliery Spoil</u>

The colliery spoil used in this project was a supply of three different types known as:

(a) Cardowan black,

(b) Cardowan black blaes

and (c) Twechar black blaes

(All the three types are shown on Plate 18).

The Cardowan black and Twechar black blaes are unburnt types of spoil whereas the Cardowan red is the burnt product of Cardowan black. The Twechar black blaes although it is an unburnt type , has been processed mechanically by the National Coal Board and has become "burnt". All the three types can equally be used for fills and other constructional purposes bearing in mind that the unburnt ones can sometimes catch fire. The National Coal Board never supplies burnt spoil mixed with unburnt ones. In this project however three equal amounts of the three types above were mixed together to make a kind of "general colliery spoil" of burnt or unburnt nature, bearing in mind that this is not the kind of "representative spoil" supplied by the NCB in the construction industry but has the same general geotechnical behaviour.

The particle size distribution of the above mixed general spoil specimen is shown in Figure 3.23. Its particle sizes range from 60 to 0.02 mm and its uniformity coefficient is 14.5m whereas Plate 17 shows a magnification of the colliery spoil's particles. A limited study was made in this material. Only a few triaxial tests were performed using mesh elements.

3.5.5 Pulverised Fuel Ash

Pulverised Fuel Ash is the residual product of the pulverised coal burnt in the furnaces of most modern electricity power stations. It is marketed as PFA by the Central Electricity Generating Board and is known internationally as fly ash.

The type of PFA used in this project is known as "base grout" and is a mixture of Cyclone and Lagoon types. In its dry state, as supplied, it can often closely resemble cement in colour and texture. The individual particles are extremely fine and spherical (rounded) in shape. They are composed principally of glass in the size and range of 5 to 0.01 mm (as shown by its particle size distribution curve on Figure 3.24) having a uniformity coefficient of 3. Its predominant elements are silicon, aluminium and iron;, the oxides of which account for 75% to 95% of the material.

The specific gravity of the Pulverised Fuel Ash used in this project was found to be 1.71. Additonally Plate 16 shows a magnification of the PFA's particle. This material was compacted by standard (BS) compaction and its compaction curve was established as shown in Figure 3.32. Its maximum dry density was found to be 1260 Kg/m^3 at an optimum water content of about 25.0 per cent. As with the previous case, an incomplete study was made with this material which consisted of a few triaxial tests using mesh elements.

3.5.6 Type 1 Aggregate

Type one aggregate consisting of crushed limestone from a Somerset quarry, was selected in order to examine the behaviour of a typical sub-base material mixed with mesh elements. Its particle size distribution curve is shown on Figure 3.25 having a particle size range from 30 to 0.03 mm and a uniformity coefficient of about 38. Its specific gravity was found to be 2.7. Only plate load bearing tests were performed on this material at the Transport and Road Research Laboratory. The results are stated in APPENDIX E.

3.5.7 London Clay

The London clay used in this study had a Liquid Limit of 78% and a Plastic Limit of 24% with a Plasticity Index of 54%. Its moisture content was 39.6%.

As with the previous three soils an incomplete study was made with this material. This particular soil was tested along with the Type one aggregate for load bearing capacity, at the Transport and

Road Research Laboratory. The results are also stated in APPENDIX E.

CHAPTER FOUR

COMPACTION, CBR AND PERMEABILITY TESTING

CHAPTER FOUR

COMPACTION, CBR AND PERMEABILITY TESTING

4.1 GENERAL

As mentioned in 3.4 not all mesh types, shown in Figure 3.14, were supplied simultaneously for testing, Types 5340, 8630 and 5345 were originally provided by the manufacturers, Netlon Ltd., for investigation. At the same time, they were researching in their own laboratories for mesh improvement in terms of weight, quality and interlocking properties. Therefore each mesh-type arriving afterwards was "an improvement" of a previous one. Thus a "mesh-element optimisation" took place in this chapter, based on compaction, CBR and permeability testing and a selection was made of the mesh element types that produced superior results, for further testing.

4.2 COMPACTION AND CBR TESTS ON MID-ROSS SAND MIXED WITH VARIOUS TYPES OF MESH ELEMENTS OF 50 x 50 mm SIZE

The 50 x 50 mm mesh-element size was the one introduced originally by Netlon Ltd. when this research started. The manufacturers' idea was to make mesh elements small enough to be handled conveniently by Engineering plant (such as air hoses, etc) or randomly "sowed" into the ground like grass, or plant seed and generally be treated like any other conventional soil stabiliser. At the same time, however, each element should be sufficiently large to form an adequate interlocking-anchorage bond with the surrounding soil particles. Hence the 50 x 50 mm size was thought to be ideal. The 50 x 100 mm mesh-element size came much later in this work and became the optimum size after certain investigation described in Chapter 5.

4.2.1 Test Program

The first set of tests was conducted on Mid-Ross sand alone in order to estabish its compaction curve at maximum density and optimum water content and consequently its CBR value range. The second set of tests was carried out on this sand mixed with each type of mesh element individually, at various percentages by dry soil weight. The compaction and CBR testing in this case was not performed at optimum water content which was about 7.5 per cent, but instead at an arbitrary value of 9.3 per cent. This moisture content was chosen as it ensured that air voids were at a minimum yet sufficient were available for occupation by the mesh elements, without causing an increase in void ratio. Thus tests at this water content provided a "worst case" condition.

All compaction tests were performed according to B.S. 1377 : 1975, Test 12, using the CBR mould. The program of this test series is shown in Table 4.1.

4.2.2 TEST RESULTS, ANALYSIS AND DISCUSSION

a) SAND ALONE

As mentioned previously (Chapter 3) the optimum moisture content of Mid-Ross sand alone, at standard compaction was about 7.5%. Its maximum average dry density for this method and level of compaction was 1818 kg/m^3 having an average porosity of 32% and a void ratio of about 48%. Mid-Ross sand at this level and method of compaction was found to be dense having a relative density of 78%. Its dry density at 9.3% water content was about 1810 Kg/m^3 (see Figure 3.26). Its CBR values at this condition were:

0.1" (2.5 mm) Penetration

2.30 ± 0.9% Top

3.60 ± 0.8% Bottom

0.2" (5.0 mm) Penetration

3.1 ± 0.8% Top

3.9 ± 0.4% Bottom

b) TYPE 5340

Mid-Ross sand was mixed with 50 x 50 mm size, Type 5340 mesh elements at various percentages varying from 0.1 to 0.9%. Type 5340 has a flexural stiffness index of 4 and maximum tensile strength 2.49 kN/m. The maximum average CBR value between 0.1" and 0.2" penetration, at the bottom of the sample-mixture, was 12.5%, at about 0.35% (by dry soil weight) mesh-element content (see Figure 4.1). The total area of mesh Type 5340 required per cubic metre of soil, to reach this maximum CBR value was about 310 m^2, as shown in summary Table 4.2.

Figures 4.1, 4.3, 4.5, 4.7, 4.9, 4.11, 4.13, 4.15, 4.17, 4.19, 4.21, 4.23, 4.25 and 4.27 show the effect of mixing various percentages of various mesh types with Mid-Ross and Leighton Buzzard sands, in terms of CBR values at both 0.1 inch and 0.2 inch penetration. They also show data from both the top and bottom side of the test specimens. It must be pointed out, however, that the results obtained using the top of the specimen should be discounted due to unavoidable disturbance of the specimen during trimming of the surface of the soil-mesh mixture. The results of the bottom of the test specimen were undisturbed and are therefore much more indicative of the CBR value of the mixtures.

(c) <u>DRY DENSITY BEHAVIOUR</u>

The variation of dry density of Mid-Ross sand with mesh-element content for each individual type of mesh element is shown in Figures, 4.2, 4.4, 4.6, 4.8, 4.10, 4.12, 4.14, 4.16, 4.18 and 4.20. From these figures it can be observed that the dry density of the soil shows a slight increase with the increase in mesh-element content up to about 0.3% concentration by dry weight. After this percentage, an overall "rapid" decrease takes place in dry density. This means that up to a certain mesh-element content the mixture remains dense, or becomes slightly denser as the void ratio decreases due to the presence of the polymer. After a certain amount of mesh concentration, the void ratio increases as the polymer replaces the soil and consequently decreases the bulk density. It was discovered however, that after 0.6% mesh-element content, the hand mixing and batching of soil/mesh mixtures becomes difficult causing a further reduction in their bulk density. This was due to the mesh elements tending to "bundle" together and consequently disturb the homogeneity of the mixture. This may be overcome when more efficient mixing-batching mechanical equipment is used on construction sites.

(d) <u>TYPE 8630</u>

When Mid-Ross sand was mixed with various percentages of Type 8630 mesh element, which has a low flexural stiffness index of 1 and maximum tensile strength of 1.3 kN/m, its maximum average CBR value (between 0.1" and 0.2" penetration at the bottom of the sample) was 11%. This occurred at a mesh-element content of about 0.3% as shown in Figure 4.3. The total area of Type 8630 mesh required per cubic metre of sand, to reach maximum CBR value was about 353 m^2 (see summary Table 4.2).

(e) <u>TYPE 5345</u>

When Mid-Ross sand was mixed at various proportions, with Type 5345 mesh element having a flexural stiffness index of 5 and maximum tensile strength of 2.78 kN/m, its maximum (average between 0.1" and 0.2" penetration at the bottom) CBR value was 21.5%. This occured at a mesh-element content of about 0.64% as shown in Figure 4.5. The total mesh area required in this case to reach maximum CBR value, was 290 m^2 per cubic metre of soil.

(f) TYPE 1

Similarly when Mid-Ross sand was mixed, at various percentages, with Type 1 mesh element having flexural stiffness index of 65 and a maximum tensile strength of 5.71 kN/m (from Table 3.2) its maximum average CBR value between 0.1" and 0.2" penetration (at bottom) was 7.4%. This occured at mesh-element content of about 0.25% as shown in Figure 4.7. The total mesh area required to reach maximum CBR value in this case was, 9.88 m^2 per cubic metre of sand.

(g) TYPE 2

When Mid-Ross sand was mixed, at various percentages, with Type 2 mesh element having flexural stiffness index 12 and maximum tensile strength 3.32 kN/m, its maximum (at bottom) average CBR value between 0.1" and 0.2" penetration, was about 12%. This occured at approximately 0.2% mesh-element content, as shown in Figure 4.9. The total mesh area required per cubic metre of soil, to reach maximum CBR value, was 28.8 m^2.

(h) TYPE 3

Mid-Ross sand was similarly mixed at various percentages with mesh-element Type 3, having a flexural stiffness index of 40 and a maximum tensile strength of 6.04 kN/m according to Table 3.2. The maximum (at bottom) average CBR value of the mixture at between 0.1" and 0.2" penetration was found to be about 8.5% at an average of 0.175% mesh-element content, as shown in Figure 4.11. The total mesh Type 3 area per cubic metre of sand required to reach maximum CBR value, was 6.3 m^2.

(i) TYPE 4

Mid-Ross sand was also mixed with mesh elements Type 4 at various percentages, having a flexural stiffness index of 4 and a maximum tensile strength of 3.09 kN/m. The maximum (at bottom) average CBR value of the mixture at between 0.1" and 0.2" penetration was about 24%, occurring at an average mesh-element content of about 0.4% as shown in Figure 4.13. The total area required of this type of mesh to achieve maximum CBR value was 252 m^2 per cubic metre of sand (see summary Table 4.3).

(j) TYPE 5

When Mid-Ross sand was mixed at various proportions, with Type 5 mesh element having a flexural stiffness index of 9 and a maximum tensile strength of 4.31 kN/m, its maximum CBR value (average between 0.1" and 0.2" penetration at the bottom of the sample) was

16.1%. This occured at a mesh-element concentration of about 0.52% as shown in Figure 4.15. The total area Type 5 required to reach the maximum CBR value was 231 m^2 per cubic metre of soil.

(k) TYPE 6

Similarly when Mid-Ross sand was mixed at various proportions, with mesh element Type 6 having flexural stiffness index 7 and maximum tensile strength 2.51 kN/m, its maximum CBR value (average between 0.1" and 0.2" penetration at bottom of sample) was 11.5%. This occurred at a mesh-element content of about 0.46% as shown in Figure 4.17. The total mesh area required in this case to achieve the maximum CBR value was 260 m^2 per cubic metre of sand.

(l) TYPE 7

Finally, when Mid-Ross sand was mixed with Type 7 mesh elements at various percentages, having a flexural stiffness index of about 10 and a maximum tensile strength of 3.76 kN/m, its maximimum CBR value was 16.6% (average between 0.1" and 0.2" penetration at the bottom of the specimen) This occurred at about 0.52% mesh-element content, as shown in Figure 4.19. The total mesh Type 7, area required to reach the maximum CBR value, was 158 m^2 per cubic metre of soil (see summary Table 4.2).

The results from the bottom the CBR test specimens show that there was a steady improvement in CBR values as the percentage of mesh was increased up to almost 0.6 per cent, where some 400 percent improvement over soil alone was found. According to these results (see Summary Table 4.2 and Figures 4.1, 4.3,....to 4.19) the mesh types that produced highest CBR values at a minimum possible required area per unit volume of soil, were Type 5345, Type 4 and Type 7. As mentioned previously however, the supply of Type 5345 was stopped by Netlon Ltd. in favour of Type 7 which was an improved version of 5345. Thus only Type 4 and Type 7 were proceeded with for further testing,. Mesh-element Types 1, 2 and 3 produced relatively low CBR values and were rejected. It seems that these types had a relatively high flexural stiffness which made them incapable of forming dense aggrgations with the surrounding soil particles and consequently resulted in high void ratios and overall reduction in the mixture's strength. Mesh-element Types 5, 6, 8630 and 5340 also produced relatively lower (maximum) CBR values in comparison to Types 4, 5345 and 7, due to probally "poorer" interlocking capacity with the sand's particles. Thus only Types 4 and 7 were selected for further testing.

From summary Table 4.2 it can be observed that the only mesh-types that reached maximum CBR values at the lowest possible percentage, were Types 1, 2 and 3. They produced maximum CBR values at an average percentage (by dry soil weight) of about 0.2%. Thus all other mesh-element types were based on this percentage for comparison. Consequently it was found that the approximate area per unit volume of all other mesh types, at this percentage varied from 63 to 69 m^2/m^3. Hence an average area (per unit volume) of 66 m^2/m^3 was established as "optimum" in terms of mesh-area economy. This value was kept as a kind of "Datum line" to which all mesh-element contents were referred, for comparison.

At between 0.1" and 0.2" penetration the maximum CBR value of Mid-Ross sand mixed with 66 m^2/m^3, 50 x 50 mm size, Type 4 mesh elements, increased from 3.8% to 7.7%. This was an improvement of almost 103%. When Mid-Ross sand was mixed with 66 m^2/m^3, 50 x 50 mm size, Type 7 mesh elements, its average CBR value (between 0.1" and 0.2" penetration at bottom of the sample) similarly increased from 3.8% to 8.1%. This was also an improvement of almost 113% (see Figures 4.13 and 4.19).

4.3 COMPACTION AND CBR TESTS ON MID-ROSS SAND MIXED WITH TYPE 7, 50 x 100 mm SIZE MESH ELEMENTS

4.3.1 Test Program

Since the fundamental behaviour of Mid-Ross sand mixed with 50 x 50 mm size mesh elements was almost established, the work was carried a stage further to investigate whether any improvement would occur when large size mesh elements were used. Thus compaction and CBR testing continued using 50 x 100 mm size, Mesh Type 7, at the same mesh-element content. The test program in this case is shown in Table 4.3. The mixing method, compaction energy, water content (at 9.3%) and testing procedures were kept the same as in the previous case (4.2).

4.3.2 Test Results, Analysis and Discussion

The results are shown in Figures 4.21 and 4.22. The maximum CBR value at the bottom of the sample, average betweenn 0.1" and 0.2" penetration;, was about 19%. This occurred at a mesh-element content of about 0.6%. Observing the results shown in Figure 4.21 however, it can be seen that there is no definite maximum CBR value in this case. The composite mixture seems to show a continuous slight increase in CBR values with mesh-element content passing the 0.6% value. The testing however, had to stop at 0.6% content since hand-mixing and handling of this amount and size became difficult. The total mesh area required in this case to reach maximum CBR value, was also 158 m^2 per cubic metre of sand.

The maximum CBR value of Mid-Ross sand mixed with 66 m^2/m^3 (or 0.18% by dry soil weight) Type 7, 50 x 100 mm size mesh elements at the bottom of the sample, average between 0.1" and 0.2" penetration, increased from 3.8% to almost 9.0%. This was an improvement of about 137% (see Figure 4.21). The behaviour of dry density in this case shown in Figure 4.22, was similar to the case when the 50 x 50 mm size was used. A small amount of increase in the dry density, took place up to 0.3% mesh-element content, and then a rapid decrease when the 0.6% content was exceeded.

4.4 COMPACTION AND CBR TESTS ON LEIGHTON BUZZARD SAND MIXED WITH 50 x 50 mm AND 50 x 100 mm SIZE, TYPE 7, MESH ELEMENTS

A series of compaction and CBR tests were performed in this section using a uniform sand, whose properties are given in Chapter 3, and Type 7 mesh elements only.

4.4.1 Test Program

Leighton Buzzard sand along was compacted in the CBR mould and its maximum dry density and optimum water content were established. Then CBR testing took place at optimum water content. Afterwards the same sand was mixed with Type 7 mesh elements of 50 x 50 mm size, at various percentages and was compacted in the CBR mould with the same compactive effort as before, at optimum water content. Then CBR testing took place for every individual mesh-element content. The same procedure was then repeated for Leighton Buzzard sand mixed with 66 m^2/m^3, 50 x 100 mm size Type 7, mesh elements. The test program is shown in Table 4.4.

4.4.2 Test Results, Analysis and Discussion

When Leighton Buzzard sand was compacted alone (according to B.S. 1377: 1975, Test 12) its maximum dry density was found to be 1618 Kg/m^3, having a void ratio of 63.7% and a porosity of 38.8%. Since no definite moisture content, or compaction curve pattern was established however for this sand (shown in Figure 3.28) a convenient value of 0% was accepted as optimum moisture content in this case.

This sand at standard (BS) compaction energy was found to be medium dense having a relative density of about 53%, as mentioned in Chapter 3. The mean CBR values of Leighton Buzzard sand alone at zero per cent water content, were:

<u>At 0.1" penetration</u>
3.50% TOP
6.03% BOTTOM

<u>At 0.2" penetration</u>
3.40% TOP
4.80% BOTTOM

(a) <u>50 x 50 mm Size</u>

When the same sand was mixed with Type 7, 50 x 50 mm size, mesh elements at various percentages, its maximum average CBR value between 0.1" and 0.2" penetration at the bottom of the sample, became 34%. This occurred at a mesh-element content between 0.4 to 0.6%, as shown in Figure 4.23. From this figure it can be seen that there is no definite maximum CBR value at the bottom of the sample containing the mixture. There is a tendency for further increase in CBR values after 0.6% mesh content, but testing had to stop at this percentage since the handling and mixing of mesh elements became difficult. At 66 m^2/m^3 (or 0.2% by dry sand weight) mesh-element content however, the average CBR value between 0.1" and 0.2" penetration at the bottom of the sample, increased from 5.4% of sand alone to about 12%. This was an improvement of 122%.

(b) 50 x 100 mm Size

When Leighton Buzzard sand was mixed with 50 x 100 mm size, Type 7 mesh elements at various proportions, the maximum average CBR value between 0.1" and 0.2" penetration at the bottom of the sample, became 46%. This also occurred at a mesh-element content of 0.6% with the tendency for further increase, as shown in figure 4.25. At 66 m^2/m^3 mesh content however, the average CBR value between 0.1" and 0.2" penetration at the bottom of the sample, increased from 5.4% to about 17%. This was an improvement of about 214%

The variation of dry density with mesh-element content of Leighton Buzzard sand for both sizes of Type 7 mesh elements, is shown in Figures 4.24 and 4.26 respectively. From these figures it can be observed that a small increase in density takes place with the increase of mesh percentage, up to 0.4%, since mesh elements increase the density of the soil mixture. After 0.4% content the dry density "levels off" and begins to decrease at about 0.6%. This behaviour is almost similar to the one of Mid-Ross sand mixed with the same Type of mesh elements (see Figures 4.20 and 4.22).

Finally comparing the CBR results of the two sands mixed with each size of Type 7 mesh element, in this chapter, it can be observed that both sands produced almost similar improvements with the exception of Leighton Buzzard sand when mixed with 50 x 100 mm size, 66 m^2/m^3 content. This mixture produced slightly higher CBR values than the corresponding Mid-Ross sand mixture. It seems therefore that Leighton Buzzard sand forms a better interlock bond with Type 7 mesh, in this case.

4.5 COMPACTION AND CBR TESTING OF WEST HIGHLAND MORAINE MIXED WITH 50 x 100 mm SIZE, TYPE 7 MESH ELEMENTS

The West Highland Moraine's physical properties are described in Chapter 3. This soil was only tested with 50 x 100 mm size, Type 7 mesh elements since this size was considered to be the "optimum" from the triaxial testing that is reported in Chapter 5. Only a limited number of tests, however, were performed on this soil.

4.5.1 Test Program

A series of compaction tests were firstly carried out according to BS 1377: 1975, Test 12, on West Highland Moraine alone in order to establish its optimum water content versus maximum dry density (compaction) curve, shown in Figure 3.31. Then a series of CBR tests took place on the soil alone at optimum water content. Secondly West Highland Moraine was mixed with 50 x 100 mm size, Type 7 mesh elements at various percentages and the mixture samples were compacted inside the CBR mould with similar compactive effort as before. Hence CBR testing was performed at optimum water content.

The test program of this section is shown on Table 4.5.

4.5.2 Test Results, Analysis and Discussion

The maximum dry density achieved on West Highland Moraine by using B.S. compaction was about 1740 Kg/m^3 having a porosity of 36 % and its optimum water content was found to be 14% (shown in Figure 3.31). When this soil alone was tested for CBR values at 14% moisture content, the mean results were:

At 0.1" penetration

6.0% TOP

11.30% BOTTOM

At 0.2" penetration

7.0% TOP

12.5% BOTTOM

Thus an average penetration at the bottom of the sample between 0.1" and 0.2", a mean CBR value of 11.5% is accepted.

When West Highland Moraine was mixed with 50 x 100 mm size, Type 7 mesh elements at various percentages up to 0.3% by dry weight (for convenience) the maximum CBR value of the mixture became about 16.5%. This occured at a mesh-element content of 0.3%, as shown in Figure 4.27. The CBR value of the mixture however, at 66 m^2/m^3 (or 0.21% by dry weight) increased from 11.5% to 15.2%. This is only a small improvement of about 32.2% compared to the previous soils. It seems therefore that the benefit gained by mixing this soil with Type 7 elements, at 66 m^2/m^3 content, is not all that significant. Another type of mesh having a smaller grid-aperture, such as Type 8630 (shown in Figure 3.14) would probably have been more beneficial in this case, as it would form a better interlock-bond with the soil's particles. Any other type of mesh having a greater depth of rib, or larger interface area, than Type 7 would be more appropriate for mixing with this soil since it would form a greater passive resistance with the West Highland Moraine"s fine particles during tensioning.

The behaviour of dry density versus mesh-element content of West Highland Moraine mixtures, was similar to the one of previous soil-mesh mixtures shown in Figure 4.28. Mesh elements slightly increased the density up to about 0.2% concentration. When this percentage was exceeded the density decreased significantly.

4.6 PERMEABILITY TESTING

Permeability tests were carried out in order to investigate any change in the Coefficient of Permeability (k) of some of the soils used in this project, when mixed with randomly distributed polymeric mesh elements. For soils of high permeability, a constant head permeameter, which was a modified oedometer apparatus was used, whereas for low permeability soils, the same apparatus converted into a falling head permeameter was used, as described in Chapter 3.

4.6.1 Test Program

1. Mid-Ross sand was firstly tested alone compacted by B.S. (according to B.S. 1377 : 1975 test 12) compaction energy at various hydraulic head-differences and confining pressures and a constant water-flow temperature of 19 ^{0}C. The apparatus used in this case was the constant head permeameter.

2. Secondly Mid-Ross sand was mixed with 66 m^2/m^3 (or 0.18% by dry weight) 50 x 100 mm size, Type 7 mesh elements and was placed compacted (by the same compactive effort) inside the constant head permeameter. Then it was tested for permeability, under the same range of hydraulic head-differences and confining pressures as before and a water-flow temperature varying between 21 and 22.5 ^{0}C.

3. Similarly West Highland Moraine was tested alone, compacted by B.S. (according to B.S. 1377 : 1975, Test 12) compactive effort at various confining pressures and a water-flow temperature varying between 18 to 19 ^{0}C. The falling head permeameter was used in this case.

4. Finally the same soil was mixed with 66 m^2/m^3 (or 0.21% by dry weight) 50 x 100 mm size, Type 7 mesh elements and was placed compacted (by the same compactive effort) inside the falling head permeameter. Then it was tested for permeability under the same range of confining pressures

and water temperatures as before.

4.6.2 Test Results, Analysis and Discussion

(a) Mid-Ross Sand Alone

When Mid-Ross sand was tested alone at zero confining pressure and at the following range of hydraulic gradients ($\Delta h/L$);

1.71, 1.36, 4.03, 3.38 and 3.48,

its average value of the coefficient of permeability (k) was found to be 2.64×10^{-4} m/s at an average water flow temperature of 19 ^{0}C. The viscosity of water at this temperature was 1.0299×10^{-6} Poiseulle* (see Table 4.6). The viscosity of water at 20 ^{0}C standard temperature is 1.0050×10^{-6} Poiseulle. Hence the coefficient of permeability of Mid-Ross sand at standard 20 ^{0}C temperature became 2.70×10^{-4} m/s.

The average coefficient of permeability value of this sand at 25 kN/m^2 confining pressure at hydraulic gradients 2.76 and 3.59 and at a water temperature of 19 ^{0}C, was found to be 2.52×10^{-4} m/s. The Coefficient of Permeability of this sand at the same confining pressure, at 20 ^{0}C temperature, becomes 2.58×10^{-4} m/s (Table 4.6).

The Coefficient of Permeability of the same sand at 50 kN/m^2 confining pressure and at 20 ^{0}C water temperature, was found to be 2.02×10^{-4} m/s. Similarly at 20 ^{0}C water temperature and 150 and

*Poiseulle = Kgs^{-1}m^{-1}= Nsm

300 kN/m^2 confining pressures, k_{20} was 1.91×10^{-4} and 1.90×10^{-4} m/s respectively. All the results of Mid-Ross sand alone are shown in summary Table 4.7.

The average dry density of the soil sample before testing, was about 1820 Kg/m^3. The classification of soils on the basis of permeability is given in Table 4.10, which is derived from a table by Terzaghi and Peck (1948). The permeability and drainage characteristics of the main soil types, in general terms are indicated diagramatically in Figure 4.31, which includes an indication of the type of test which is most appropriate for each category. These data are shown in a different way, related to effective particle size, in Figure 4.32 (Head, 1981). According to these figures Mid-Ross sand has a "medium" permeability classification and "good" drainage characteristics. Its hydraulic gradient with water flow relationship was fairly linear, and is shown in Figure 4.29. Additonally from summary Table 4.7 it can be observed that the coefficient of permeability decreases with an increase in confining pressure. This is due to the reduction of porosity and increase in density of the sample. The selection of top confining pressures during this testing simulated real cases of water, flowing through Mid-Ross sand, at various depths.

(b) Calculation of k in Constant-Head Permeameter

The basic equation for permeability calculations is based on the assumption that the flow of water is laminar, or streamline, and not turbulent. This assumption is generally valid for soils ranging from clays to coarse sands, but may not be so for coarser materials. The relationship, discovered by Darcy (1856), concerning the flow of water in sands states that the rate of flow is proportional to the hydraulic gradient.

$$q = \frac{Q}{t} = kAi \quad , \quad i = \frac{\Delta h}{L} \qquad (4.1)$$

$$\text{or} \quad k = \frac{qL}{A\Delta h} \qquad (4.2)$$

where A is the area of cross section of the soil, k is the coefficient of permeability, $\Delta h/L$ is the hydraulic gradient and q is the rate of water flow. Thus equation (4.2) was used for the calculation of k values.

(c) Effect of Temperature

Permeability k is not constant for a given soil but is related to the dynamic viscosity of the water. Viscosity however, varies with temperature and therefore the water temperature was taken into account when performing permeability tests. It is convenient to relate permeability data to a standard temperature of 20 ^{0}C. If a permeability test carried out at T^{0}C gives a coefficient of permeability k_T, the corresponding value at 20 ^{0}C, (k_{20}) is calculated from the equation:

$$k_{20} = k_T \left(\frac{\eta_T}{\eta_{20}}\right) \qquad (4.3)$$

where $\frac{(\eta_T)}{(\eta_{20})}$ is read from Table 4.6.

(d) Mid-Ross Sand + Mesh Elements

When Mid-Ross sand was mixed with 66 m^2/m^3, 50 x 100 mm size, Type 7 mesh elements, the average value of the coefficient of permeability of the composite material (converted into 20 ^{0}C water temperature according to equation 4.3 and Table 4.6) at zero confining pressure, was 2.69×10^{-4} m/s. The values of hydraulic gradient, Ah/L at which testing range took place, were 1.69, 1.37, 4.18 and 3.26. The water temperature during testing varied between 21.5 and 22.5 ^{0}C. Additionally at 25 kN/m^2 confining pressure, the average value of the coefficient of permeability of the composite material at 20 ^{0}C, was found to be 2.28×10^{-4} m/s. Finally at 20 ^{0}C water temperature and 50, 150 and 300 kN/m^2 confining pressures, the coefficients of permeability of the composite material, were 2.19×10^{-4}, 2.11×10^{-4} and 2.04×10^{-4} m/s respectively. All the results for the composite material are shown in summary Table 4.8. The average dry density of the sand/mesh mixture samples before testing, was about 1850 Kg/m^3. The hydraulic gradient versus water flow, relationship, is shown in Figure 4.30.

Comparing the results of Mid-Ross sand alone to the ones of Mid-Ross sand mixed with Type 7 mesh elements, it can be observed that the mesh elements do not seem to affect the permeability characteristics of this sand.

(e) West Highland Moraine

West Highland Moraine was tested in the falling head permeater, due to its low permeability coefficient. The average sample dry density before testing was about 1730 Kg/m^3. Its average coefficient of permeability, converted to standard 20 ^{0}C temperature according to Table 4.6 and equation 4.3, at zero confining pressure, was 2.24 x 10^{-6} m/s. At (top) confining pressures of 25 and 50 kN/m^2, the permeability coefficients of West Highland Moraine, at 20 ^oC, were 1.92 x 10^{-6} and 1.77 x 10^{-6}m/s respectively (see summary Table 4.9).

(f) Calculation of k in Falling Head Permeameter

The notation used in the analysis is shown in Figure 3.10 as follows;

L : Length of sample

A : Cross-sectional area of sample

a : Cross-sectional area of standpipe tube

y_1,y_2 : Heights of water above datum in standpipe at times t_1, t_2 respectively

y : Height of water above datum at any intermediate time t

dy : Fall during small time increment dt

dQ : Quantity of water flowing through sample in small time increment dt

y_0 : Height of outlet level above datum

At any time t, the difference in height between the inlet and outlet levels is equal to $(y-y_0)$. The hydraulic gradient, i, at this instant is therefore equal to $(y-y_0)/L$

The quantity of water flowing through the sample in time dt is equal to the area of the standpipe multiplied by the drop in height of the water level, i.e.

$$dQ = -a\,dy$$

But from Darcy's law (equation 4.1)

$$dQ = Akidt =$$

$$dQ = Akidt = \frac{Ak\,(y - y_o)}{L}\,dt$$

Thus,

$$-ady = \frac{Ak(y - y_o)}{L}dt$$

or

$$-\frac{1}{y - y_o}dy = \frac{kA}{aL}\,dt$$

Integrating between limits $y = y_1$ to y_2 and $t = t_1$ to t_2,

$$-\int_{y_1}^{y_2}\frac{dy}{y-y_o} = \int_{t_1}^{t_2}\frac{kA}{aL}\,dt$$

or

$$-\left[\log_e(y-y_o)\right]_{y_1}^{y_2} = \left[\frac{kA}{aL}\right]_{t_1}^{t_2}$$

Hence

$$\log\frac{y_1 - y_o}{y_2 - y_o} = \frac{kA}{aL}\,(t_2-t_1) \qquad (4.4)$$

putting $y_1 - y_0 = h_1$ and $y_2 - y_0 = h_2$, equation (4.4) becomes :

$$k = \frac{aL}{A(t_2-t_1)}\log_e\left(\frac{h_1}{h_2}\right) \qquad (4.5)$$

Thus equation 4.5 was used for the calculation of the permeability coefficients in the case of falling head permeameter.

(g) West Highland Moraine + Mesh Elements

When West Highland Moraine was mixed with 66 m^2/m^3, 50 x 100 mm size, Type 7 mesh elements the average coefficient of permability of the composite material at 20 ^oC and zero confining pressure, was found to be 2.20 x 10^{-6} m/s. At 25 and 50 kN/m^2 confining pressures the coefficients of permeability were 1.75 x 10^{-6} m/s and 1.56 x 10^{-6} m/s respectively. Similarly these results are also shown in summary Table 4.9. Thus comparing the coefficient of permeability of West Highland Moraine alone, to the one when this soil was mixed with mesh elements, it can be observed that the mixing of mesh elements did not affect significantly the permeability characteristics of West Highland Moraine.

4.7 CONCLUSIONS

As stated previously, the mesh elements interlock with soil particles to form aggregations and these are in turn locked together by adjacent meshes to form a coherent matrix. Generally the requirements were that the mesh elements were evenly distributed and randomly oriented throughout the soil matrix. Preliminary tests using a wide variety of mesh types have shown that the crucial factors in achieving this, are the size and shape of the elements and their flexural stiffness and recovery. For different methods of mixing and different end uses, it is likely that the elements will vary in shape from squares to rectangles. For ease of mixing and maintenance of their geometrical stability during this stage and during subsequent stressing, the flexural stiffness and recovery of very flexible meshes would form loose bundles and large voids within the soil and not interlock as intended. At the opposite end of the stiffness range, rigid elements were found to form bridges and so void spaces within the soil, which is highly undesirable.

Thus the flexural stiffness and recovery properties of the mesh elements are important and must be carefully selected.

The results from the CBR testing showed that there is a steady improvement in CBR values as the percentage of mesh is increased up to almost 0.6 per cent for all types of soil and mesh element.

In the cases of Mid-Ross and Leighton Buzzard sands mixed with mesh element Type 7, the CBR test specimens showed that there is a large improvement in the CBR values, as the mesh concentration is increased up to almost 0.6 per cent, where some 200 per cent improvement over soil alone is discovered. It is envisaged that smaller mesh contents would be used in practice, however, it is clearly demonstrated in this testing that such smaller mesh content would still provide substantial improvement in soil properties.

The change of mesh element size from 50 x 50 mm squares to 50 , 100 mm rectangles made no significant difference in the CBR performance of both sands.

In the case of West Highland Moraine, the improvement in CBR values, as the percentage of mesh increased up to almost 0.3 per cent, was not very significant. It seems however that the interlock mechanism between Type 7 and silt particles, is weak. Another type of mesh element therefore should be attempted, having a smaller aperture (such as the Type 8630) or a greater depth of rib to produce a better interlock mechanism, or greater passive resistance with the soil particles during tensioning.

The dry density of the soil/mesh mixtures seemed to show a slight increase as the percentage of mesh increased up to almost 0.3 per cent. After that the dry density decreases. When 0.6 per cent mesh-content is exceeded, the decrease is very substantial.

The mixing of the mesh elements into the soil is easily and efficiently achieved up to 0.6% by dry weight of the soil alone. After this percentage, the mixing becomes difficult, and the composite material rapidly transforms into an elastic medium which could be undesirable in certain Civil Engineering applications.

Finally when granular soils are mixed with mesh elements, their permeability characteristics remain almost unaffected.

CHAPTER FIVE

TRIAXIAL TESTING

CHAPTER FIVE

TRIAXIAL TESTING

5.1 GENERAL

To determine the fundamental stress-strain behaviour of soil-mesh mixtures, 150 mm diameter by 200 mm height drained triaxial tests on soil samples with lubricated ends, were carried out on Mid-Ross and Leighton Buzzard sands mixed with various proportions of mesh elements. Similarly limited numbers of triaxial tests were performed on West Highland Moraine, Colliery spoil and Pulverised Fuel Ash.

All the triaxial soil specimens with, or without mesh elements were tested dry, except for the cases of West Highland Moraine and Pulverised Fuel Ash, where the samples were tested at optimum water content and treated as clayey soils, as will be described later.

For a large number of tests reported in this project, the mesh was cut into elements of 50 x 50 mm size, but for the majority of tests it was cut into 50 x 100 mm elements. Also some tests were performed with mesh cut into 50 x 150 mm strips, or 100 x 100 mm squares, with the greater length in the machine direction. Only one type of mesh was considered throughout the triaxial testing. This was the Type 7. A small amount of triaxial testing was carried out, however, on Types 10 and 12 and their results are compared to mesh Type 7, as detailed in Appendix F.

5.1.1 Correction Studies in Triaxial Testing

(a) VOLUME CHANGE CORRECTION

This work consists of a series of comparative tests between soil alone and soil mixed with mesh elements. In order to assess the values of the deviator stresses at any particular axial strain, the cross secitonal area has to be determined. This area is a direct function of the original volume of the sample plus the change in volume (ΔV) during initial consolidation of the sample and during subsequent dilation (or shearing). For dry sand however, it is very difficult to monitor both of these. Additionally, the shape of the triaxial specimens containing sand and mesh elements changes during testing (later shown in Figure 5.71 and Plates 22 and 30) so such that determination of average cross sectional area is rather complex. Thus in the calculations, the initial volume has been assumed to be constant. As the testing in this work is of a comparative nature, the error between deviator stresses for soil alone and the composite material calculated with this constant volume assumption, should be very small. From Appendix D where the absolute error of this constant volume assumption is assessed, it is apparent that a maximum error of about 5% would result for the sand, or sand and mesh elements and so the comparative error is a fraction of this.

(b) MEMBRANE CORRECTION

A membrane correction was applied and taken into account. This correction factor was necessary at low cell pressures. The procedure is described in Appendix D.

5.2 TRIAXIAL TESTING OF MID-ROSS AND LEIGHTON BUZZARD SANDS MIXED WITH 50 X 50 mm SIZE MESH ELEMENTS

At first, as described in previous chapters, the mesh-size used was about 50 x 50 mm because it was thought that the smaller the size of the mesh elements, the more convenient would be their handling in the construction industry. The concept of 50 x 100 mm mesh size came later in this project when research was devoted to mesh size optimisation. However, most of the work in this chapter was performed using the minimum (50 x 50 mm) size mesh.

5.2.1 Test Program

A series of drained triaxial tests were performed on 155 mm diameter by 200 mm height, cylindrical specimens made out of compacted Mid-Ross sand alone, having free ends, at various cell pressures. The range of cell pressures examined were:

0, 10, 25, 50, 100, 150, 200 and 300 kN/m^2

The same procedure as above, was then repeated for compacted Leighton Buzzard sand alone and the range of cell pressures examined were:

0, 10, 25, 50, 150 and 300 kN/m^2

A series of drained triaxial tests was then performed on Mid-Ross sand and Leighton Buzzard sand mixed with Type 7, 50 x 50 mm size mesh elements, at various percentages and tested at the same cell pressures as sands alone. The mesh element concentrations used in every case were:

<u>Mid-Ross sand</u>

33 m^2/m^3 of dry soil (or 0.09% by dry soil weight)
66 " " " " (or 0.18%) " " " ")
90 " " " " (or 0.24% " " " ")

<u>Leighton Buzzard sand</u>

33 m^2/m^3 of dry soil (or 0.10% by dry soil weight)
60 " " " " (or 0.20% " " " ")
90 " " " " (or 0.27% " " " ")

The composite specimens had also free ends and the same size and were compacted with the same compaction energy as the soils alone.

The rate of deformation chosen throughout the triaxial testing was: 0.1 mm/min or alternatively the rate of strain was 0.05%/min.

The triaxial specimens were tested until they reached 30 mm total deformation, or 15% total axial strain. The reason for a slow rate of strain was to include possible creep effects during loading.

The testing programme for the triaxial testing is shown on Table 5.1.

5.2.2 Test Results and Discussion

The average triaxial sample weight of Mid-Ross sand was about 6.9 Kg and its average volume 0.0038 m^3. Thus a total average density of about 1815 Kg/m^3 was achieved for the sand alone with an average porosity of 33% and a void ratio of about 50%. All Mid-Ross sand triaxial specimens were dense having a relative density of about 79%. The triaxial sample dry densities of the same sand mixed with mesh elements at various contents, varied from 1810 to 1849 Kg/m^3 having the same, or slightly greater average density and consequently porosity, to soil alone. Similarly for the Leighton Buzzard sand, the total average dry density was about 1640 Kg/m^3 having an average porosity of about 38% with an average void ratio of 60% and a weight of 6.3 Kg. At this density, Leighton Buzzard was found to be medium dense having a relative density of about 58%. When the same sand was mixed with Type 7 mesh at various proportions, the dry density of the composite material varied between 1650 to 1710 Kg/m^3. This indicates that when mesh elements are added to the soil mass, the composite material becomes slightly denser. This, same, behaviour was observed in CBR testing before.

Note: All these density values were achieved employing standard compaction energy according to BS 1377: 1975, Test 12, modified for the triaxial split former-mould size. It can be seen however, that by employing standard (B.S.) compaction energy Leighton Buzzard sand became medium dense, whereas Mid-Ross sand for the same compactive effort became dense. It seems therefore that it is not always the amount of compactive effort that governs the density state of a uniform sand, but rather the mode of compaction. A vibrating plate for example, for the same compaction energy, could possibly make Leighton Buzzard sand denser, than the rammer method.

(a) MID ROSS SAND
$33\ m^2/m^3$ CONTENT

The deviator stress of Mid-Ross sand at peak or residual stress states, when mixed with $33\ m^2/m^3$ (or 0.09% by dry soil weight) Type 7, 50 x 50 mm size mesh-elements, increased by an average factor of 1.23 at high cell pressures (50, 100, 150, 200 and 300 kN/m^2). At low cell pressures (10 and 25 kN/m^2) the deviator stress at peak or residual stress states, also increased by a factor of 1.48 compared to that of sand alone. (See Figures, 5.1, 5.2, 5.3, 5.6, 5.12, 5.15, 5.18, 5.40, 5.43 and 5.46). A list of all deviator stresses is also shown in Table 5.2.

The performance efficiency (η) for both peak and residual stress states and for the whole range of cell pressures (see Table 5.15) was found to range between 10 to 65.3%, depending on the stress level.

The performance efficiency (η) in this case is defined as;

$$\eta = \frac{(\text{d.s. of composite material} - \text{d.s. of soil alone}) \times 100}{\text{d.s. of soil alone}}$$

where d.s. = Deviator stress

The case of zero cell pressure for soils alone, has not been tested in triaxial apparatus and therefore is not included in the above values. Although there is a small value of initial radial pressure in this case, due to the pressure of the rubber membrane holding the specimen's particles together, the values of the deviator stresses at peak or residual stress states were insignificant. Therefore in the mathematical analyses following in this project they are considered as zero, for convenience.

66 m^2/m^3 CONTENT

Similarly when Mid-Ross sand was mixed with 66 m^2/m^3 (or 0.18% by dry soil weight) Type 7, 50 x 50 mm mesh-elements, the deviator stress for both peak and residual stress states, at high cell pressures, increased by a factor of 1.32 to that of sand alone. At low cell pressures, however, it increased by an average factor of 2.24. A list of all deviator stresses in this case, is given on Table 5.3. See also Figures 5.1, 5.3, 5.6, 5.7, 5.13, 5.16, 5.19, 5.41, 5.44 and 5.47. The performance efficiency (η) in this case, for the whole range of cell pressures, was found to range between 20 to 217% depending on the stress level (see Table 5.3).

90 $m^2/^3m$ CONTENT

Finally when Mid-Ross sand was mixed with 90 m^2/m^3 (or 0.24% by dry soil weight) Type 7, mesh-elements of 50 x 50 mm average size, the deviator stress at high cell pressures increased by an average factor of about 1.5. At low cell pressures however, it increased by a factor of 3.3 compared to sand alone. In this case the performance efficiency (η) for both peak and residual stress states and for the whole range of cell pressures (except when $\sigma_3 = 0$) was found to range between 30 to 290% depending on the cell pressure. (See Table 5.4 also Figures 5.1, 5.4, 5.6, 5.14, 5.17, 5.20, 5.42, 5.45 and 5.48).

(b) LEIGHTON BUZZARD SAND

$33\ m^2/m^3$ CONTENT

In the case of Leighton Buzzard sand mixed with $33\ m^2/m^3$ (or 0.10% by dry sand weight) Type 7 mesh-elements of 50 x 50 mm size, the total deviator stress for both peak and residual stress states and for high cell pressures (50, 150 and 300 kN/m^2) increased by a factor of 1.22. At low cell pressures (10 and 25 kN/m^2) however, it increased by an average factor of 1.6 compared to sand alone. The performance efficiency for each case is shown in Table 5.5. The deviator stress behaviour with strain, is also shown in Figures 5.8, 5.9, 5.21, 5.24, 5.25, 5.28, 5.31, 5.34 and 5.37. The performance efficiency for both, peak and residual stress states and for the whole range of cell pressures (high and low) was found to range between 13 to 70% depending on the stress level. The case of zero cell pressure has not been included in the calculations.

$66\ m^2/m^3$ CONTENT

Similarly when Leighton Buzzard sand was mixed with $66\ m^2/m^3$ (or 0.20% by dry soil weight) Type 7 mesh elements of 50 x 50 mm size, the average deviator stress at high cell pressure increased by a factor of 1.46. At low cel pressures, however, the deviator stress also increased by an average factor of 2.8. A list of the deviator stresses at peak and residual stress states and the performance efficencies at all stress levels, is given on Table 5.6. See also Figures 5.8, 5.10, 5.22, 5.24, 5.26, 5.28, 5.32, 5.35 and 5.38. The performance efficiency in this case, for the whole range of cell pressures (except when $\sigma_3 = 0$) was found to range between 30 to 300% depending on the stress level.

90 m^2/m^3 CONTENT

Finally when the same sand was mixed with 90 m^2/m^3 (or 0.27% by dry sand weight) Type 7 mesh-elements of the same size, the total deviator stress (peak and residual) at high cell pressures, was increased by an average factor of 1.65. At low cell pressures, however, it increased by a factor of 3.3. For this mesh content, the performance efficiencies was found to range between 44 and 500% depending on the cell pressure. All deviator stresses are shown plotted on Figures 5.8, 5.11, 5.23, 5.24, 5.27, 5.30, 5.33, 5.36 and 5.39. The list of peak and residual stress-state values of the deviator stresses and the performance efficiencies are shown on Table 5.7.

The behaviour indicated by the previous test series data, clearly demonstrate the ability of the mesh elements to generate tensile strain resistance from the beginning of the test. The tendency for the improvements to level-off at high axial strains is believed to be associated with the soil alone and soil/mesh composite material both approaching their state of constant volume. Drained triaxial tests on fully saturated samples in which volume changes can be measured, to investigate this phenomenon, could be performed, but photographic records of the forms of soil alone and soil mixed with mesh elements, test specimens when both are at large strains illustrate the changes in overall deformation characteristics that the mesh content imposes on the sand as it is drained. (See Figure 5.71 and Plates 21, 22 and 30). Thus the sand with mesh elements has both different strength and deformation characteristics than the sand alone.

Plates 21, 22 and Figure 5.71 show the overall shape of triaxial specimens at 15 per cent axial strain after testing. The sample containing the sand and mesh element mixture shows "growths" in the surrounding membrane due to "bundles" of soil and meshes slipping sideways during failure. The sample however, containing sand alone only shows a shear "slip" or a "bulging" shape depending on its density. This indicates that the overall deformation pattern of the composite material is different to that of soil alone.

Additionally from the previous data test series, mesh elements increase the deviator stress developed at all strains, even at very small strains, and the peak stresses in the sand/mesh element mixtures occur at slightly higher axial strains than for the sand alone.

Finally the performance efficiency reduces as the cell pressure increases (see Tables 5.2, 5.3, 5.4, 5.5, 5.6 and 5.7). This indicates that the 50 x 50 mm size mesh elements are more effective at low applied external pressures than high. At high pressures the two sands seem to have sufficient self-strength to resist shearing action, whereas at low external pressures the polymeric mesh elements provide large shearing resistance compared to that of sand alone.

5.2.3 Analysis of Results

The results of all four cases such as;

1. sands alone,
2. sands mixed with 33 m^2/m^3 mesh elements,
3. sands mixed with 66 m^2/m^3 mesh elements,
4. sands mixed with 90 m^2/m^3 mesh elements,

were computed using the Mohr-circle stress analysis shown in Figures 5.52, 5.53, 5.54 and 5.55. Two types of computation were employed; one graphical and one analytical. The analytical method of computation was only used for the Mid-Ross sand mixed with 50 x 50 mm mesh-elements. Due to the curvilinear nature of the composite material's Mohr envelope however, this method was found "limited" and therefore a graphical simplified bi-linear envelope analysis was chosen to represent the soil + mesh element behaviour throughout this work.

(a) ANALYTICAL METHOD

This method is based on Mohr-Coulomb failure theory;

$$\sigma_1 = \sigma_3 K_p + 2c\sqrt{K_p} \qquad -(1)$$

where; σ_3 = minor total principal stress,

σ_1 = major total principal stress

c = cohesion , Φ = angle of internal friction, and

$K_p = \tan^2(45 + \Phi/2)$ = constant — (2)

It should be noticed however, that the symbols c & Φ are <u>not</u> the "cohesion" and "the angle of internal friction" of Mid-Ross sand when mixed with mesh elements, behaving as a composite material, therefore

substituting c by I

and Φ by S,

(2) becomes; $K_p = \tan^2(45 + S/2)$

and $\sqrt{K_p} = \tan(45 + S/2)$

Equation (1) can be written as;

$$\sigma_1 = m\sigma_3 + n$$

From the triaxial-test results, σ_1 was plotted against σ_3 using regression analysis (method of Least Squares) for every percentage of axial strain and for the whole range of strains; 0.5, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 8.0, 10.0 and 12.0%, also 15.0% (residual stress state) and peak sress state (independent of axial strain). Thus the equations of the straight lines were computed from these results, and m, n were measured.

$$m = K_p, \quad n = 2\,I\sqrt{K_p}$$

Hence S was calculated from;

$$m = \tan^2(45 + S/2)$$

and then I from;

$$n = 2\,I\,\tan(45 + S/2)$$

S in this case is the computed intercept of the composite material, otherwise called "apparent cohesion" (Gray, 1985) or "pseudo cohesion" (Hausmann, 1978), and S is the angle of shearing resistance of the composite material.

This analysis was carried out for every percentage of axial strain and for both peak and residual stress conditions. Some of the results of this analysis are shown in APPENDIX B.

(b) <u>BI-LINEAR MOHR ENVELOPE ANALYSIS</u>

This graphical method of analysis was widely used throughout this work. In order to illustrate the deviator stress improvements at all strains, Mohr failure envelopes were constructed for the sands with and without mesh elements, for peak stress conditions as shown in Figures 5.52, 5.53, 5.54 and 5.55.

Equivalent envelopes, based on mobilised stresses at 1.0 and 15.0 per cent axial strains were also produced (Figure 5.56) to show the improved behaviour of the composite material at low and high strain conditions. From these and similar envelopes at different constant axial strains, the increase in shear resistance ($\Delta\tau$) was plotted against normal stress (σ) for both peak stess and constant axial strain conditions as shown in Figure 5.57 (a) and (b) respectively. The peak stress condition does not fit the pattern shown for constant axial strain conditions as it compares stresses developed at unequal axial strains (see Figure 5.57).

Representation of Strength Characteristics

In order to represent the data obtained from the previous test series a standarised means of characterising the strength of soil + mesh has been developed which is compatible with that of the soil alone.

The approach taken is to represent the soil + mesh as a "modified soil" using a Mohr envelope. Due to the curvilinear nature of the actual envelope, a simplified bi-linear envelope has been chosen to represent the soil + mesh element behaviour over the normal stress range of 0 to 500 kN/m^2. From the data obtained, as an example, from Mid-Ross sand mixed with Type 7 mesh elements, the most appropriate bi-linear envelope is determined in a manner shown in Figure 5.52. The initial highly curved part of the actual envelope, between 0 and 50 kN/m^2 normal stress (σ), is represented by the line OF and the envelope between 50 and 500 kN/m^2 is represented by the line FG. The slope of the line FG is represented as ($\Phi+\beta$) where Φ is the slope of the Mohr envelope for the soil alone. The extension of the line FG projected back to vertical axis is denoted as b and the equation of the bi-linear Mohr envelope can then be represented as:

For $\sigma =$ 0 to 50 kN/m^2 then

$$\tau = \sigma\left(\frac{b}{50}\right) + \sigma\tan(\Phi+\beta)$$

For $\sigma =$ 50 to 500 kN/m^2 then

$$\tau = b + \sigma\tan(\Phi+\beta)$$

where Φ is a characteristic of the soil alone and b and β are the characteristic modifications to the soil behaviour derived from the mesh elements.

To illustrate the application of the above approach to the results obtained from the triaxial testing, the construction of the bi-linear Mohr enveope for Mid-Ross sand mixed with 66 m^2/m^3 Type 7, 50 x 50 mm mesh elements, is shown in Figures 5.53 and 5.54. As can be seen, the bi-linear envelope always produces a conservative estimate of the soil + mesh strength throughout the stress range considered.

Using the above approach the strength characteristics of Mid-Ross sand when mixed with Type 7, 50 x 50 mm and 50 x 100 mm size mesh elements have been obtained and are as listed in Table 5.15. The bi-linear Mohr envelope analyses of this sand are shown in Figures 5.59, 5.60, 5.61, 5.62, 5.63 and 5.64.

Similarly the strength characteristic of Leighton Buzzard sand when mixed with Type 7, 50 x 50 mm and 50 x 100 mm size mesh elements are also listed in Table 5.16. The bi-linear Mohr envelope analyses of this sand are also shown in Figures 5.65, 5.66, 5.67, 5.68, 5.69 and 5.70.

5.3 TRIAXIAL TESTING OF MID-ROSS AND LEIGHTON BUZZARD SANDS MIXED WITH 50 x 100 mm MESH ELEMENTS

Further tests at various cell pressures were conducted on soil-mesh element mixtures with 33, 66 and 90 m^2/m^3 mesh content. The individual mesh elements were cut into a 50 x 100 mm size.

5.3.1 Test Program

Mid-Ross and Leighton Buzzard sands were also selected as in the previous case (5.2.1) and mixed with Type 7, 50 x 100 mm size mesh elements. Triaxial samples of 155 mm diameter by 200 mm height as before, were prepared using identical mixing method and compaction energy (BS 1377 : 1975, Test 12) as in the case of 50 x 50 mm size mesh-element mixtures. The rate of deformation selected was 0.1 mm/minute and the rate of strain 0.05%/minute. All samples were strained up to 15 per cent axial strain (or a total of 30 mm deformation) as before. The range of cell pressures selected in this case was:

0, 10, 25, 50, 150 and 300 kN/m^2

A summary of the test program is listed in Table 5.8.

5.3.2 Test Results and Discussion

The presence of the 50 x 100 mm size, Type 7 mesh elements mixed in the same proportion as before, did not seem to alter much the behaviour of the composite material's density. As the mesh content increased a very small increase in the bulk density of the composite material was observed. This indicates that the presence of mesh elements make the mixtures denser up to a limit of about 0.3 per cent by dry soil weight. Once this limit is exceeded a sudden drop in density occurs indicating that the material is becoming loose. The average sample density of Mid-Ross sand mixed with Type 7, 50 x 100 mm size mesh elements was about 1818 kN/m^2 varying from 1805 to 1850 kN/m^2. This was almost the same as the previous case when Mid-Ross sand was mixed with 50 x 50 mm size meshes. The present a void ratio of 51% and they were also dense having a relative density of about 80%, similar to that of sand alone.

The average triaxial sample density of Leighton Buzzard sand mixed with Type 7, 50 x 100 mm mesh elements, was about 1650 Kg/m^3 varying from 1640 to 1710 Kg/m^3. This was almost the same as in the case when the same sand was mixed with 50 x 50 mm size mesh elements. The average triaxial sample porosity was similarly about 37% having a void ratio of 60%. These values made the composite material samples medium dense having a relative density similar to that of Leighton Buzzard sand alone, of about 57%.

The dry density of the composite material triaxial samples increased slightly (up to 10 per cent) with the increase in mesh-element content. This increase was up to about 0.3 per cent by dry sand weight mesh-content. When this mesh-element content was exceeded the dry density of the composite material decreased rapidly since the soil/mesh samples became looser, as described in 5.2.2.

(a) MID -ROSS SAND

33 m^2/m^3 CONTENT

The deviator stress at peak and residual stress states of Mid-Ross sand mixed with 33 m^2/m^3 (or 0.09% by dry sand weight) Type 7, 50 x 100 mm size mesh elements, increased by an average factor of 1.25 at high cell pressures (50, 100, 150, 200 and 300 kN/m^2). See Figures 5.40, 5.43 and 5.46.

At low cell pressures, however (10 and 25 kN/m^2) the average

At low cell pressures, however (10 and 25 kN/m^2) the average deviator stress (peak and residual) of the composite material also increased by a factor of about 2.04 compared to that of sand alone. These stress values are listed in Table 5.9. See also Figures 5.12, 5.15 and 5.18.

The performance efficiency (η) defined in 5.2.2, for both peak and residual stress states and for the whole range of cell pressures (except the case when $\sigma_3 = 0$) was found to range between 13 to 237% depending on the stress level (see Table 5.9). The case of cell pressure equal to zero has not been tested for sands alone. Although there is always a small radial initial pressure on the triaxial specimen due to the membrane holding the particles together, for convenience, however, this pressure is considered insignificant or zero. The analysis of membrane effects on the triaxial specimen and membrane correnctions are given in Appendix D.

66 m^2/m^3 CONTENT

When Mid-Ross sand was mixed with 66 m^2/m^3 (or 0.18% by dry soil weight) Type 7, 50 x 100 mm size mesh elements, the average (peak and residual) deviator stress at high cell pressures increased by a factor of 1.4 compared to sand alone. At low cell pressures they similarly increased by a factor of about 2.97. A listing of all deviator stresses in this case is given in Table 5.10. See also Figures,5.13, 5.16, 5.19, 5.41, 5.44 and 5.47. The performance efficiency at all stress states and for the whole range of cell pressures (except in the case of σ_3 = 0 kN/m^2) ranged between 20 to 354%, depending on the stress level.

90 m^2/m^3 CONTENT

Finally when Mid-Ross sand was mixed with 90 m^2/m^3 (or 0.24% by dry sand weight) Type 7 mesh elements of 50 x 100 mm size, the total deviator stress (peak and residual) at high cell pressures increased averagely by factor of 1.58 compared with sand alone. At low cell pressures however, the same stress increased by a factor of 4.88. In this case the performance efficiency (η) for both peak and residual stress states, for the whole range of cell pressures (except when σ_3 = 0) was found to range between 32 to 800% depending on the stress level. All peak and residual values of deviator stresses are listed in Table 5.11. See also Figures 5.14, 5.17, 5.20, 5.42, 5.45 and 5.48.

(b) LEIGHTON BUZZARD SAND

33 m^2/m^3 CONTENT

In the case of Leighton Buzzard sand mixed with 33 m^2/m^3 (or 0.10% by dry soil weight) Type 7 mesh elements of 50 x 100 mm size, the deviator stress at both peak and residual stress states, at high cell pressures (50, 150 and 300 kN/m^2) increased by a factor of about 1.36. At low cell pressures however (10 and 25 kN/m^2) it increased by an average factor of 2.16 compared to that of sand alone. A listing of the performance efficiencies (η) as well as the deviator stresses for each case, is shown in Table 5.12. All stresses are also shown in Figures 5.21, 5.24, 5.25, 5.28, 5.31, 5.34 and 5.37. The performance efficiency for both peak and residual stress states, for all cases of cell pressures (high and low) was found ranging between 28 to 220% depending on the stress level.

66 m^2/m^3 CONTENT

Similarly when Leighton Buzzard sand was mixed with 66 m^2/m^3 (or 0.20% by dry soil weight) Type 7 mesh elements of 50 x 100 mm size, the deviator stress at peak and residual stress states, at high cell pressures, increased by an average factor of about 1.55. At low cell pressures however, it increased by a factor of 3.8 compared to that of sand alone. A listing of the performance efficiencies (η) is given in Table 5.13. All deviator stress patterns are also shown in Figures 5.8, 5.22, 5.24, 5.26, 5.29, 5.32, 5.35, and 5.38. The performance efficiency for both peak and residual stress states and for the whole range of cell pressures (except when $\sigma_3 = 0$ kN/m^2) was also found to range between 35 to 420% depending on the stress level.

90 m^2/m^3 CONTENT

Finally when the same sand was mixed with 90 m^2/m^3 (or 0.27% by dry soil weight) Type 7, mesh-elements of 50 x 100 mm size, the deviator stress at peak and residual stress states and at high cell pressures, was increased by an average factor of about 1.8. At low cell pressures however, it also increased by an average factor of 5.15 compared to that of sand alone. All deviator stress values at peak and residual stress states are given in Table 5.14. Similarly all deviator stress patterns are shown in Figures 5.8, 5.23, 5.24, 5.27, 5.30, 5.33, 5.36 and 5.39. For the 90 m^2/m^3 mesh element content, the total performance efficiency (η) for both peak and residual stress states and for the whole range of cell pressures (except when $\sigma_3 = 0$) was found to range between 48 to 800% depending on the stress level (high or low).

The performance efficiencies shown in Tables 5.9, 5.10, 5.11, 5.12, 5.13 and 5.14, increased with the increase in mesh element content for both sands. When the mesh element size changed from 50 x 50 mm to 50 x 100 mm, the average performance efficiency almost doubled in value. This indicates that the choice of 50 x 100 mm size mesh for mixing with both sands was justifiable, since it forms a far superior interlock bond than the 50 x 50 mm one.

Comparing the performance efficiencies between the low and high stress levels shown in previously mentioned Tables, it should be noticed that mesh elements are far more beneficial at low stress levels than high ones, as in the case of 50 x 50 mm size mixtures. This means that mesh elements could be an advantage in shallow earthworks, since soils at large depths possess sufficient strength to resist shear stresses on their own.

Additionally comparing the performance efficiencies between the 50 x 50 and 50 x 100 mm size elements, it can be observed that the soil/mesh mixtures containing 50 x 50 mm size mesh elements have performance efficiency almost twice as great at low stresses, than at high ones. Similarly mixtures containing 50 x 100 mm size mesh elements have performance efficiency almost four times greater at low stresses than high ones.

The performance efficiencies η, of both sands mixed with mesh elements were plotted against all cell pressures (except the case of $\sigma_3 = 0$) and are shown in Figures 5.50 and 5.51. The curves AA' and BB' in these figures, indicate the variation of the performance efficiency of the composite material with cell pressure at peak and residual stress states respectively. The AA' and BB' curves show that the performance efficiency gradually increases as the cell pressure decreases. These curves however are not asymptotes to the x and y axes. This means that the performance efficiency η, does not become "infinity" at zero cell pressure. There must be a small amount of "confinement" or external pressure, for the mesh elements to become effective. In the case of triaxial testing this "confinement" at zero cell pressures, was provided by the rubber membrane holding the mixture's soil particles together. If the membrane was removed, or sliced with a sharp knife, most of the sand "bled sideways" resulting in a collapse of the cylindrical specimen. In the case of full-scale trials of the bearing capacity of fill containing mesh-elements (Transport and Road Research Laboratory, 1985) this "confinement" was provided by a small surcharge applied round the circular test footing (see Appendix E). Therefore a small confinement is essential for mesh elements to become effective. The concept of "zero cell pressure" is only a "mathematical convenience".

5.3.3 Analysis of Results

The results were analysed graphically in a similar manner to the case of 50 x 50 mm size mesh elements. In order to illustrate the deviator stress improvements at all strains, Mohr failure envelopes were constructed for sands with and without mesh elements, for peak and residual stress conditions, as shown in Figures 5.59, 5.60, 5.61, 5.62, 5.63, 5.64, 5.65, 5.66, 5.67, 5.68, 5.69 and 5.70. Due to the curvilinear nature of the actual envelope, a simplified bi-linear envelope has been chosen to represent the soil + mesh element behaviour over the normal stress range of 0 to 500 kN/m^2, as in the case of 50 x 50 mm mesh elements.

The strength characteristics of sands alone and sands mixed with 50 x 100 mm size mesh elements are shown in summary in Tables 5.15 and 5.16. From both Tables can be observed that the "apparent angles of friction" ($\phi + \beta$) of the modified soils increase by 1/2 to 5 degrees maximum, compared to those of sands alone, whereas an "apparent cohesion" (or intercept) b, is developed due to the presence of polymer. This is in the order of 23 to 70 kN/m^2 depending on the mesh element content.

5.4 TRIAXIAL TESTING OF MID-ROSS SAND WITH OTHER SIZES OF MESH ELEMENTS

Since the fundamental behaviour of the Mid-Ross and Leighton

Buzzard sands mixed with 50 x 50 mm size, Type 7, mesh elements at various percentages was established, the work was carried a stage further, to investigate whether any improvement would occur when larger size elements were used. Thus triaxial testing continued using 50 x 100 mm and 50 x 150 mm strips and also 100 x 100 mm squares mixed with the same granular materials.

5.4.1 Test Program

An arbitrary cell pressure of 150 kN/m^2 was selected for comparison, and the mesh material was cut into 50 x 100 mm, 50 x 150 mm and 100 x 100 mm sizes. The granular soil used in this case was Mid-Ross sand and the percentage of mixing was selected as an average of 66 m^2/m^3 or 0.18% by dry soil weight. Two triaxial, dry an drained tests were performed for every mesh size case and the results are demonstrated in Figure 5.49.

The soil was mixed with 50 x 100 mm and 50 x 150 mm strips and similarly the 100 x 100 mm square meshes on a mixing tray using a hand tool, and was placed in the triaxial former-mould in three layers, and compacted using the 2.5 Kg drop-rammer according to BS 1377 : 1975 Test 12. Thus 200 mm height by 155 mm diameter triaxial specimens were formed. Then triaxial testing proceeded.

5.4.2 TEST RESULTS, ANALYSIS AND DISCUSSION

As can be seen from the results of Figure 5.49, all mesh sizes larger than 50 x 50 mm mesh, give greater deviator stresses than the original composite material mixed with the 50 x 50 mm size ones. This increase seems to be significant in the small axial strain region, (between 1% to 5%) and also at the residual (15%) strain region, rather than the peak. The magnitude of this increase in deviator stress, is in the order of about 150 kN/m^2. The 50 x 100 mm size meshes however, seem to be more effective up to 5% axial strain, and also between 13% and 15% strain. Hence it was decided that the optimum mesh-element size in this project should be 50 x 100 mm. When this size was exceeded, the behaviour of the composite matrial in terms of deviator stress remained the same. It seems therefore that a kind of "saturation" in mesh element size takes place when the 50 x 100 mm size is exceeded. All deviator stress lines seem to overlap in a "bundle shape" also shown in Figure 5.49.

The reduced benefit at peak and residual conditions of 50 x 150 mm elements compared to 50 x 100 mm and 100 x 100 mm elements was due to the 50 x 150 mm elements being larger than the diameter of the test specimen and therefore being in many cases "forced" to double over in the sample. This doubling over reduced the effective length of these elements.

Larger diameter triaxial tests would require to be undertaken to determine if further improvement over 50 x 100 mm and 100 x 100 mm size elements is obtainable. The marked improvement however in the intitial slope of the curves (see Figure 5.49) of all the sand mixed with mesh-elements samples, over the sand alone, was obvious.

The "mesh interlock length" (or mesh anchorage length) optimisation was also investigated by carrying out a series of "pull out" tests in a special "pull out" testing apparatus (Pradhan, G.S., 1985). The effective anchorage length of a strip mesh being pulled out of a compacted and compressed soil mass, in relation to the externally applied compressive stress (on the soil mass) was examined. The results of these tests were in agreement with the present project, in that they indicated a optimum mesh size of 50 x 100 mm. A brief summary of these results is given in Appendix C.

5.5 TRIAXIAL TESTING OF MID-ROSS SAND REINFORCED BY 155 mm DIAMETER MESH TYPE 7, DISCS

So far the random distribution of polymeric mesh elements has been examined. An attempt however was made to investigate the case when mesh is placed inside the soil in the form of horizontal sheets, along the direction of the principal planes.

The whole idea was to "reconstruct" the concept of "polymer grid reinforcement" using instead of geogrids or geotextiles, mesh Type 7 material. Hence a comparison could then be drawn between the concepts of random distribution of polymeric mesh and non-random distribution.

5.5.1 Test Program

Mesh Type 7 was cut into circular sheets - disc shape - of similar diameter to the triaxial specimens (155 mm). These discs were placed inside the soil into horizontal layers, spaced approximately 15 mm apart and forming a 200 mm composite column consisting of soil and layers of mesh-sheets alternatively. No mixing took place in this case. The soil-mesh-disc composite was constructed inside the triaxial mould-former containing the rubber membrane in its inner-wall surface. The compactive effort was identical to the previous cases when mesh elements were used with the same soils, achieving an average density of 1840 Kg/m^3. The soil used in this case was Mid-Ross sand. Thirteen layers of 155 mm diameter, Type 7, were inserted into the soil, for every triaxial specimen, making a total of about 66 m^2 per cubic meter of dry soil, or 0.18% by weight. Only the 66 m^2/m^3 mesh-content was examined in this case and the following tests were carried out as shown on Table 5.17.

5.5.2 Test Results and Analysis

The triaxial testing results were plotted and are shown in Figures 5.72, 5.73, 5.74, 5.75, 5.76 and 5.77. From these figures it is noticed that the deviator stresses show much greater improvement when mesh discs are placed along the planes of principal tensile strains (especially at small to medium strains). This could be analogous to the case where all mesh elements are placed along the principal tensile strain-directions and are not distributed at random. In this instance, all polymer strength seems to be utilised since it all lies inside the tensile zones of the triaxial specimen.

Mohr circles were drawn and Mohr bi-linear envelope analysis was carried out, as shown in Figures 5.61 and 5.62. The results listed in Table 5.15 show a very small change in the angle Φ of the reinforced material, but a very large intercept b, of about 70 kN/m^2.

Comparing the results of Table 5.19, where Mid-Ross sand was reinforced by discs, to those of Table 5.10 where Mid-Ross sand was mixed with 66 m^2/m^3, Type 7, 50 x 100 mm mesh elements, it should be noticed that the reinforcing Type 7 mesh adds approximately an extra 40% performance efficiency over the other case. This seems to happen however, only at low stress levels. At high stress levels, very little difference between the discs and the 50 x 100 mm size mesh can be observed. The deviator stresses of the disc/soil arrangement, at high cell pressures and large strains show a rapid decline after 12 per cent axial strain. They seem to "join" the deviator stresses of sand mixed with 50 x 100 mm size mesh elements as shown in Figures 5.72, 5.73, 5.74, 5.75, 5.76 and 5.77. This could probably be explained by the sudden rupture of some discs after a certain sample deformation has taken place. (See Plates 27 and 28). When the triaxial testing was completed, samples were stripped and the polymer discs were examined. Two to three discs (out of thirteen) were often ruptured, as shown on Plates 27 and 28.

5.6 THE EFFECT OF RATE OF STRAIN IN THE COMPOSITE MATERIAL

The stress-strain behaviour of polymeric meshes, at constant temperature, depends on the rate of applied strain. This is shown in Figures 3.16 and 3.19. When meshes are mixed with soil however, a composite material is formed. The stress-strain behaviour of this composite material was investigated at various rates of strain, and at constant (ambient) temperature, in triaxial testing. The idea behind this investigation was to form a relationship between the stress-strain behaviour of the polymer itself (since this is well established) and the stress-strain behaviour of the composite material. If this relationship became apparent then "a new avenue" could possibly open in "designing with mesh-elements", using only the stress-strain characterestics of the polymer itself.

5.6.1 Test Program

At first, Mid-Ross and Leighton Buzzard sands were tested in triaxial on their own, at three different rates of strain: 0.05, 0.5 and 2 percent/minute. The results are shown in Figures 5.78 and 5.79. The cell pressure adopted throughout this work was 150 kN/m^2. Secondly triaxial specimens were made out of Mid-Ross sand and thirteen, mesh Type 7, 155 mm diameter discs placed horizontally inside the 200 mm height by 155 mm diameter soil-sample mass, spaced about 15 mm apart. Thus a number of tests were also performed using 150 kN/m^2 cell pressure, at the above three basic rates of strain. The results are shown in Figure 5.80. Thirdly a number of tests were performed using Type 7 mesh elements cut into 50 x 100 mm size and mixed randomly with Mid-Ross sand at 0.18% by weight, and with the Leighton Buzzard sand at 0.20% by dry weight (or a total of 66 m^2 per cubic meter of dry soil). These triaxial specimens were tested at 0.005, 0.01, 0.05, 0.5 and 2.0 per cent per minute, rates of strain. The results are shown in Figures 5.81 and 5.82. Al triaxial samples were compacted identically using standard compaction effort, according to BS 1377 : 1975, Test 12, and they were tested at the same cell pressure. A test program of

5.6.2 Test Results, Analysis and Discussion

Figures 5.78 and 5.79 show the behaviour of the granular soil alone at various rates of strain. According to Cassagrande (1948) the speed of triaxial testing should not affect the deviator stress of a dry granular material. From these figures, one could say that the deviator stresses of both sands at various rates of strain, show an identical behaviour pattern within the limits of experimental error.

When thirteen sheets (155 mm diameter discs) of mesh Type 7 were placed along the principal tensile strain directions inside the granular soil-sample's mass, forming a total of 66 m^2 per cubic metre of dry soil, the behaviour pattern of the reinforced soil's deviator stresses altered. See Figure 5.80. The "fast" tests (2.0%/minute strain) showed a much higher deviator stress pattern at peak and residual stress states (unless the mesh ruptured as shown on Plates 27 and 28). The "slow" tests (0.5, 0.1 and 0.05%/minute strain) showed a lower deviator stress pattern. This phenomenon is analogous to the stress-strain behaviour of the mesh polymer alone, shown in Figures 3.16 and 3.17. At "fast" rates of deformation, no creep is allowed in the system and therefore the polymer shows much higher strength, but ruptures earlier. In the case of "slow" rates of deformation, an allowance is made for creep in the system and therefore lower strength is achieved, but larger extensibility.

When geogrids or geotextiles, are used in reinforcing granular (or cohesive) materials, the concept of the stress-strain behaviour of the polymer itself, is a predominant design factor. The case, however, of cohesive) materials, the concept of the stress-strain behaviour of the polymer itself, is a predominant design factor. The case, however, of material Type 7 cut into 50 x 100 mm size elements and mixed with the soil, at random orientation, showed different results. (See Figures 5.81 and 5.82). The composite material containing sand and 66 m^2/m^3 mesh elements does not seem to follow a similar pattern to the stress-strain behaviour of the polymer alone. Figures 5.81 and 5.82 show an average of two to three repetitions of each test. No definite pattern in the deviator stress behaviour can be distinguished. It seems that the composite material is independent of any rate of deformation and does not relate to the stress-strain behaviour of the polymer itself. Creep is not a significant factor in this case and the composite material, consisting of granular soil and mesh elements, complies more with Cassagrande's theory.

5.7 TRIAXIAL TESTING OF COLLIERY SPOIL MIXED WITH MESH ELEMENTS

As mentioned previously in Chapter 3, a limited number of triaxial tests were carried out mixing Colliery spoil with mesh elements to investigate any improvement in the stress-strain characteristics of the mixture.

5.7.1 Test Program

The type of colliery spoil used here was a mixture of three different kinds of spoils, mentioned also in Chapter 3.

Two dry, 155 mm diameter by 200 mm height, triaxial specimens were prepared inside the split-mould fitted with surrounding rubber membrand and lubricated circular membranes, to produce free ends at its top and bottom. Colliery spoil was mixed with 66 m^2/m^3 (or 0.22% by dry spoil weight) 50 x 100 mm size mesh elements and placed inside the mould in three layers and subjected to standard (BS 1377 : 1975, Test 12) compaction.

Similarly the above procedure was repeated for two dry 155 mm diameter by 200 mm height, triaxial specimens of Colliery spoil alone, without mesh elements.

All triaxial testing was conducted under fully drained and dry conditions at an arbitrary cell pressure of 150 kN/m^2. The samples were tested up to 20% strain (or 40 mm total deformation) at a constant rate of deformation of 0.25 mm/minute.

5.7.2 Test Results, Analysis and Discussion

The average results of the triaxial testing of Colliery spoil, alone and mixed with mesh elements, are shown in Figures 5.83, 5.84 and 5.85. The colliery spoil alone at standard compaction, became dense, to medium dense in state having an average density of 1620 Kg/m^3. Its angle of internal friction at peak stress state (from one Mohr circle) was about 50^0.

From Figures 5.83, 5.84 and 5.85, it can be seen that mesh elements improved significantly the deviator stress of Colliery spoil.

The deviator stress of the composite material at peak stress state increased by a factor of about 1.3 compared to Colliery spoil alone, with a performance efficiency of 25%. At residual stress state, however, the deviator stress of the composite material increased by a factor of about 1.56, with a performance efficiency of 47%. Colliery spoil alone failed at about 10% axial strain whereas the mixture failed at twice that.

From the same figures it was observed that both triaxial specimens behaved in a similar way up to about 4.0% axial strain. Afterwards the spoil/mesh-element mixture started becoming stronger.

5.8 TRIAXIAL TESTS ON WEST HIGHLAND MORAINE MIXED WITH MESH ELEMENTS

West Highland Moraine was the only natural silt chosen to be mixed with Type 7 mesh elements and tested triaxially. Its properties are given in Chapter 3. Only a limited number of tests however, were performed on this material, due to shortage of time.

5.8.1 Test Program

Two tests were carried out in this section. The first one was performed on West Highland Moraine alone and the second, on the same soil mixed with 66 m^2/m^3 (or 0.20 per cent by dry soil weight) 50 x 100 mm size, Type 7 mesh elements. An arbitrary cell pressure of 150 kN/m^2 was chosen throughout the triaxial testing of this section.

Like all previous cases West Highland Moraine had to be tested either fully dry and drained, or fully saturated and drained in order to avoid any possible pore-water pressure build-up in the system. Technically, however, it was very difficult for West Highland Moraine to be compacted fully dry, inside the split mould, since a lot dust was created. It was similarly difficult for the above soil to be compacted fully saturated and also to be kept "upright" (into shape) by lowering an attached water burette and create negative pressure in order to release the split mould and insert the rubber membranes. Hence it was decided to perform triaxial testing with the soil partially saturated and compacted at optimum moisture content and maximum dry density. The following conditions, however, had to be satisfied:

(a) Firstly the West Highland Moraine, compacted at optimum water content, had to be treated as clay and be subjected to consolidation.

(b) Secondly the consolidation-curve-analysis should provide a suitable rate of deformation (or rate of axial strain) so that no pore water pressures were built-up in the sample during the fully drained triaxial testing.

(c) Finally the required rate of strain derived from the consolidation analysis, should lie between the rate of strain-limits of the mesh-polymer/soil composite materials tested in the previous section 5.6. These limits were:

0.005%/minute, (lowest rate of strain)

and 2%/minute, (highest rate of strain).

West Highland Moraine alone or mixed with mesh elements, was compacted by standard compacting (according to BS 1377 : 1975, Test 12) at 14% optimum water content. Hence 200 mm height by 155 mm diameter triaxial samples were prepared.

CONSOLIDATION PROCEDURE

The sample was pressurised at 150 kN/m^2 cell pressure and its outlet drain valve was closed and connected to a volume calibrated burette. When the outlet drain valve was released, consolidation began immediately. the water started rising in the burette and its volume was recorded at the following time intervals:

0, 15 seconds, 30 seconds, 1 minute, 2 minutes, 4 minutes, 8 minutes, 15 minutes, 30 minutes, 1 hour, 2 hours, 4 hours, 8 hours and 24 hours. Thus the cumulative change in volume ΔV was plotted against the square root of time (in minutes) and the curve shown in Figure 5.86 was produced.

5.8.2 Test Results and Analysis

The following analysis was carried out;

It is found that the average degree of dissipation at failure $\bar{U}_f$, may be expressed in the form :

$$\bar{U}_f = 1 - \frac{h^2}{\eta C_v t_f}$$

where $h = 1/2$ the height of the sample

C_v = coefficient of consolidation

t_f = time to failure

η = a factor depending upon drainage conditions at the sample boundaries

In our case $\eta = 3.0$ for drainage from both ends (Bishop and Henkel).

A comparison between this theory of consolidation (Gibson and Henkel, 1954) and the results of many drained tests shows that a theoretical degree of dissipation of 95% is sufficient to ensure a negligible error in the measured strength. The requisite time to failure for a test may then be written as:

(2)- $t_f = \dfrac{h^2}{0.05\eta C_v} = \dfrac{20h^2}{\eta C_v}$ (Bishop & Henkel, 1962)

The consolidation stage, prior to the shear test (volume change against $\sqrt{t}$) was used and the time intercept of the assymptotes at 100 % consolidation from Figure 5.86 was found to be:

(3) — $\sqrt{t_{100}}$ = 8.7,or t_{100} = 75.69 mins.

The coefficient of consolidation for triaxial testing is given by the expression:

(4) — $C_v = \dfrac{\pi h^2}{4\,t_{100}}$ (Bishop & Henkel, 1962).

Substituting (3) into (4),

$$C_v = \frac{\pi\,100^2}{4 \times 75.69} = 103.71 \text{ mm}^2/\text{min}$$

Substituting the value of C_v in (2) by taking

η = 3.0 for drainage at both ends (Bishop & Henkel, 1962).

Hence equation (2) becomes:

$$t_f = \frac{20 \times (100)^2}{3.0 \times 103.71} = 642.8 \text{ mins.}$$

This is the total requisite time to failure.

Assuming that the sample failed at about 10% axial strain, or 20 mm of the 200 mm height sample, then the rate of strain required should be:

$$\frac{20}{642.8} \sim 0.031 \text{ mm/min,or } 0.01\ \%/\text{min.}$$

If the sample however had failed at 5% axial strain, then the rate of strain required should be:

$$0.01\ ^{mm}/min\text{ , or }\ 0.005\%/min.$$

Thus the above two rates of strain calculated from the triaxial consolidation analysis, lay well within the limits: 0.005%/minute and 2%/minute, mentioned previously in section 5.6. Hence the two samples, one containing only West Highland Moraine at 14% water content, and the other containing the same soil and water content mixed with 66 m^2/m^3 mesh elements, were tested at a rate of strain of 0.005%/min, or a 0.01 mm/minute deformation rate. The results of the above triaxial testing are shown in Figure 5.87. The angle of internal friction of this soil (from one Mohr circle) was found to be about 36^0.

5.8.3 Discussion of Results

There was very little improvement in the deviator stress of West Highland Moraine when mixed with Type 7 mesh elements, as shown from Figure 5.87. The performance efficiency of the soil/mesh mixture was in the order of 18% to 20%. It seems in this case, that the "interlock mechanism" of Type 7 mesh element with this fine (powdery) soil is not very efficient. It was suggested therefore that, since West Highalnd Moraine is susceptible to water content, the mixing of Type 7 mesh elements with a combination of a small concentration of cement, or bitumen, or any other type of conventional soil-stabiliser, would probably provide additional improvement in its strength behaviour but with loss in ductility and permeability. Alternatively, another Type of mesh element could be used for mixing with this soil, such as Type 5340, or Type 8630 (see Figure 3.14).

These two types would probably form a better "interlock mechanism" with the soil's fine particles, than the one of Type 7, because they have a much smaller aperture size grid. Alternatively any other Type of mesh having a greater depth of rib, or larger interface area than Type 7, would be more appropriate for mixing with this soil, since it would form a greater passive resistance with West Highland Moraine's fine-size particles, during tensioning.

5.9 TRIAXIAL TESTING OF PULVERISED FUEL ASH MIXED WITH MESH ELEMENTS

The P.F.A. described in Chapter 3 was also examined by triaxial testing. This was the only c-ϕ material tested with mesh elements in this project and due to a shortage of time only limited testing was performed on it.

5.9.1 Test Program

Pulverised Fuel Ash, for the same reasons as with West Highland Moraine, was treated as a clayey soil and was tested partially saturated. Only two tests were performed in this case as follows:

1. P.F.A. alone, was mixed with water at optimum (25%) moisture content and compacted by standard (BS) compacton energy according to BS 1377 : 1975, Test 12. Thus a 155 mm diameter by 200 mm height, triaxial specimen as before, was prepared. The specimen was then consolidated at 150 kN/m^2 cell pressure for 24 hours and afterwards was tested under undrained

conditions with measurement of pore water pressure, at the same cell pressure. The rate of deformation applied was 0.02 mm per minute and the test stopped when the specimen reached 40 mm total deformation, after 24 hours.

2. P.F.A. again was mixed with 66 m^2/m^3 (or 0.27% by dry weight) Type 7, 50 x 100 mm size mesh elements, at 25% optimum water content. The mixing and compaction was identical to the previous case and in the same way, a 155 mm diameter by 200 mm height triaxial sample was formed. This was also consolidated in the triaxial cell at 150 kN/m^2 cell pressure and 24 hours later was tested at the same cell pressure, with a rate of deformation similar to the case when no mesh elements were involved.

5.9.2 Test Results, Analysis and Discussion

The volume of water ΔV rising in the burette during consolidation was recorded and plotted against the square root of time shown in Figure 5.88. From this figure it can be observed that the value of t_{100} (Bishop & Henkel, 1962, p.126) was 272.25 mins. This value indicates that P.F.A., being almost an impervious material and having a coefficient of permeability k in the order of 0.005 x 10^{-6} to 0.08 x 10^{-6} m/s (S.G.E.B. 1984) would take a very long time to be tested under drained conditions, without pore water pressures being built up during testing. Thus it was decided to perform testing under undrained conditions using a sensitive pressure transducer to measure the pore pressures. Consequently effective stresses were assessed by subtracting pore water pressures. Since P.F.A. when mixed with water acquires cementitious (pozzolanic) properties, the deviator stress levels versus axial strain shown in Figure 5.89, would be much higher in 7, 28, or 365 days than at 1 day which was the time of testing. (Sutherland, & Finlay, 1968). See also Figure 3.33.

When P.F.A was tested mixed with 66 m^2/m^3, Type 7 mesh elements under the same conditions, the deviator stress showed an increase in performance efficiency of about 13% at peak stess state. At residual stress state however, the performance efficiency also increased by about 30% (see Figure 5.89). These improvement compared to the Mid-Ross and Leighton Buzzard sand mixtures were relatively smaller. It seems therefore that the benefit gained in mixing P.F.A. with Type 7 mesh elements is not all that significant. Another type of mesh, having a smaller grid-aperture, such as Type 8630 (shown in Figure 3.14) would probably have been more beneficial in this case, since it would form a better interlock-bond with the fine particles of the Pulverised Fuel Ash used in this project. Alternatively any other type of mesh having a greater depth of rib, or larger interface area than Type 7, would be more appropriate in mixing with PFA since it would form a greater passive resistance with the ash's fine particles, during tensioning.

5.10 REPEATED LOADING TESTS ON MID-ROSS AND LEIGHTON BUZZARD SANDS MIXED WITH RANDOMLY ORIENTED MESH ELEMENTS OR REINFORCED BY 155 mm DIAMETER MESH DISCS

The testing of soils mixed with randomly distributed mesh elements, or reinforced by horizontally oriented mesh discs, has so far been restricted to static tests of one form or another. However, if such elements are to be introduced into a layer of road construction it becomes vital to investigate the effects of repeated loading, in simulation of stresses induced by moving vehicles. A limited study on repeated (or cyclic) loading took place in this project. Due to the limitations in technical facilities and equipment, however, the greatest part of this study was sub-contracted by the manufacturers, to Nottingham University (Brown & Thom, 1985).

5.10.1 Test Program

A total of nine tests were performed here.The rate of deformation was kept, as in section 5.2 at 0.1 mm/minute (or rate of strain: 0.05% /minute) for all cases.

The cell pressure was also kept at 25 kN/m^2 for all nine tests.

The test program was as follows;

1. A Mid-Ross sand alone, 155 mm diam. by 200 mm height sample, compacted at standard (BS 1377 : 1975, Test 12) compaction, was prepared and tested in the triaxial apparatus, fully drained at both ends and dry, for six "loading-unloading" cycles.

2. The same procedure was repeated again to check the consistency of results.

3. A Leighton Buzzard sand alone, 155 mm diameter by 200 mm height sample, compacted at standard (BS 1377 : 1975, Test 12) compaction, was also prepared and tested in the triaxial apparatus

drained at both ends and fully dry, for six "loading-unloading" cycles.

4. The same procedure was repeated again.

5. Mid-Ross sand was then mixed with 66 m^2 per cubic metre of dry soil (or 0.18% per dry soil weight) 50 x 100 mm size mesh elements of Type 7. A 155 mm diameter by 200 mm height sample, compacted with standard compactive energy, as above, was prepared and tested in the triaxial apparatus, drained at both ends, and fully dry, for six "loading-unloading" cycles.

6. The same test was repeated again.

7. Similarly Leighton Buzzard sand was mixed with 66 m^2 per cubic metre of dry soil (or 0.20% by dry soil weight) 50 x 100 mm size mesh elements Type 7. A 155 mm diameter by 200 mm height sample, compacted by standard compaction, was also prepared and tested in the triaxial apparatus fully dry and drained at both ends, for six "loading-unloading" cycles.

8. The same test was repeated.

9. Finally Mid-Ross sand was reinforced by thirteen horizontally orientated, 155 mm diameter discs of mesh Type 7, making a total of 66 m^2 mesh per cubic metre of dry soil. Hence a 155 mm diameter by 200 mm height sample, compacted by B.S. standard

compaction, was prepared as before. This sample was tested in the triaxial apparatus fully dry anddrained at both ends, for six "loading-unloading" cycles.

5.10.2 TEST RESULTS, ANALYSIS AND DISCUSSION

Two stress levels were selected as "borders" for the stress fluctuation during each "loading-unloading" cycle : one "upper" and one "lower".

The "upper" stress level chosen was 140 kN/m^2. This is the aproximate deviator stress in which Mid-Ross and Leighton Buzzard sands begin to fail when they are tested, on their own, at 25 kN/m^2 cell pressure, without mesh elements. The "lower" stress level selected was 40 kN/m^2. This again is approximately the minimum deviator stress that the soil/mesh element mixtures begin to show substantial difference over the ones of soils alone, when tested in triaxial, at 25 kN/m^2 cell pressure (see Figures 5.90 and 5.91).

The results of Mid-Ross sand mixed with 66 m^2/m^3, Type 7, 50 x 100 mm size mesh elements and Mid-Ross sand alone, are shown in Figure 5.90. These rsults indicate that for the six "loading-unloading" cycles, the composite material containing Mid-Ross sand mixed with elements, showed a far superior and resilient stress-strain behaviour to the one of soil alone.

When a number of stress cycles is applied to an element of granular material, it will very quickly settle down to a repeatable non-linear elastic stress-strain relationship, unless the peak stress is at, or close to, failure. There will be in both cases an irrecoverable strain associated with each load cycle. This elastic relationship is known as the 'resilient stress-strain behaviour' and is important in any consideration of repeated loading. The relationship is a complex one and many forms have been devised in which to express it, none being completely adequate (according to Mayhew, 1983, TRRL LR 1088).

In the case of Mid-Ross sand alone (Figure 5.90 B) the six cycles covered a range of axial strains from 1.60% to almost 2.25%, leaving a difference of about 0.65% strain irrecoverable. Thus a permanent deformation is obvious here. In the case of Mid-Ross sand mixed with mesh elements (Figure 5.90 A) the six cycles covered a range of axial strains from 0.8% to 1.0%, leaving a difference of almost 0.2% strain irrecoverable. Thus the permanent deformation in this case, was about a third of the previous one. It is therefore obvious that the soil/mesh element composite material retains its elastic properties.

Similarly in the case of Leighton Buzzard sand alone (Figure 5.91 B), the six cycles covered an axial strain range of 2.3% to almost 3.1%, leaving a difference of 0.8% strain irrecoverable. Thus a permanent deformation is obvious here too. The Leighton Buzzard sand mixed with mesh elements (Figure 5.91 A) however, during the six "load-unload" cycles, covered a range of axial strains from 0.9% to 1.2%, leaving a difference of almost 0.3% strain irrecoverable. The permanent deformation therefore in this case was about a third of that of the soil alone. Thus, it is obvious that the soil/mesh element mixture is much more superior in retaining its elastic properties.

Additonally an attempt was made to place Type 7 mesh polymer horizontally along the triaxial soil-specimen's principal axes, where in this case all the polymeric mesh will be utilised in tension, since it all lies along the direction of tensile strains. In this way an ideal condition was going to be investigated where mesh elements are not randomly distributed, and the amount of resilient stress-strain behaviour to be benefitted by the use of this system. Hence Mid-Ross sand was reinforced by thirteen Type 7, mesh-material discs, making a total of about 66 m^2 per cubic metre of dry soil, and spaced about 15 mm apart.

In the case of Mid-Ross sand alone (see Figures 5.90B and 5.92 B the six cycles covered an axial strain range of 1.60% to almost 2.25%, leaving a difference of 0.65% strain irrecoverable, as mentioned previously. When the Mid-Ross sand was reinforced by the thirteen discs (Figure 5.92A) the six cycles covered an axial strain ranging from 0.9% to 1.10%, leaving a difference of 0.2% strain irrecoverable. Thus the permanent deformation in this case, was slightly less than one third of the unreinforced sand one. This shows that the difference between the non-randomly orientated mesh and the randomly orientated one, in resilient stress-strain behaviour, during repeated loading, and for the chosen stress levels (40 and 140 kN/m^2) is not very significant.

The stress/strain relationship in cyclic loading that can be substituted in analytical and design procedures requiring a "modulus of elasticity", is called Resilient Modulus (ASTM. D. 18.09.06) and is expressed by

$$M_r = \sigma_d / \varepsilon_r$$

where: σ_d = the repeated deviator stress, and

ε_r = the recoverable (resilient) axial strain

The Resilient Modulus of Mid-Ross sand alone at 140 kN/m^2 repeated deviator stress and 2.25% recoverable strain was:

$$M_r = 140/2.25 = 62.2$$

In the case of Mid-Ross sand mixed with mesh elements, the recoverable axial strain was 1.0% at repeated deviator stress of 140 kN/m^2,

$$M_r = 140/1.0 = 140$$

Similarly for Leighton Buzzard sand alone,

$$M_r = 140/3.1 = 45.1$$

and for the same sand mixed with mesh elements,

$$M_r = 140/1.2 = 116.6$$

Thus in both cases the sand/mesh-mixtures possess much higher Resilient Moduli than sands alone.

5.11 LONG TERM LOADING TESTS ON MID-ROSS SAND MIXED WITH MESH ELEMENTS

The rate of settlement due to creep was another factor that had to be examined in order to assess the behaviour of sand/mesh element mixtures over a long period of time.

5.11.1 Test Program

Three main tests were carried out in this section, as follows;

1. Mid-Ross sand was mixed with 66 m^2/m^3 (or 0.18% by dry weight) 50 x 100 mm size, Type 7 mesh elements. The mixture was compacted by standard (BS) compaction, modified for the triaxial former-mould, according to BS 1377 : 1975, Test 12, and placed into it, in three layers. Hence a 155 mm by 200 mm height triaxial specimen was prepared and assembled, with a cell pressure of 10 kN/m^2 applied to it, by means of a hydrostatic column of water. At the same time a dead load of 142.5 Kg (or 73.6 kN/m^2 pressure) was applied vertically into the triaxial cell's piston by means of of dead weights attached into the hanger, as described in 3.2.9 and shown on Plates 8 and 9. This dead load was left for about 45 days.

2. The same test was repeated again for the same soil and mesh element size, Type, content and density, but the applied cell pressure was 25 kN/m^2 this time. Similarly a dead load of 258.2 Kg (or 134 kN/m^2 pressure) was aplied via the piston and left for about 51 days.

3. Similarly the same test was repeated for the same soil + mesh element size, Type, content and density, but the applied cell pressure was 50 kN/m^2. A dead load of 377 Kg (or 195.5 kN/m^2 pressure) was applied vertically via the piston and kept for about 51 days.

4. Finally, in order to determine the creep parameters A, a and m (described in 5.11.2) from the log strain-rates versus stress intensity and log time, two simple triaxial specimens were

prepared containing Mid-Ross sand and $66 m^2/m^3$, Type 7 mesh elements, at the same density as before. Each specimen was loaded by a different dead load; one analogous to a third 0.308 kN (or 31.4 Kg) and the other analogous to a quarter 0.23 kN (or 23.5 Kg) of the total sample's failure load 0.992 kN (or 94 Kg). Both samples were tested for about ten days, at zero cell pressure, and their rates of settlement were recorded.

The dead load in all cases was applied incrementally over a period of 24 hours and remained for the specified period during which, a reading was recorded, on the deformation gauge once a day.

5.11.2 Test Results

In the case of $10 kN/m^2$ cell pressure, Mid-Ross sand, alone, failed at $69 kN/m^2$ deviator stress and about 3 per cent axial strain, with an axial maximum piston load of 1.347 kN (or 137.3 Kg) as shown in Figure 5.16 and Table 5.10. When this sand was mixed with $66 m^2/m^3$ Type 7, 50 x 100 mm size mesh elements, it failed at $211 kN/m^2$ deviator stress at about 4.5 per cent axial strain, with an axial maximum piston load of 4.183 kN (or 426.4 Kg) also shown in Figure 5.16 and Table 5.10. Thus by using a factor of safety of 3, which is common in Foundation Engineering, it was decided to apply one third of this total maximum (4.183 kN) failure load, i.e. 1.394 kN (or about 142.5 Kg) in the long-term loading test.

The 142.5 Kg load was applied incrementally over a period of 24 hours to simulate an initial time-elapse when initial consolidation settlement takes place in reality, during which time, a structure is being built. Since one third of the failure load of the sand/mesh element mixture was greater than the total failure load of the sand itself, no "control" testing took place in this work.

At a cell pressure of 10 kN/m^2, the total settlement of the specimen containing Mid-Ross sand mixed with 50 x 100 mm size, 66m^2/m^3 mesh elements, forty five days after the total dead load was applied, was 0.35 mm (or 0.17% strain). This makes an average rate of settlement-strain of:

$$\dot{\varepsilon} = 3.88 \times 10^{-3}\%/\text{day}$$

The results of this test are shown in Figure 5.93.

Similarly, at 25 kN/m^2 cell pressure Mid-Ross sand alone, failed (in triaxial testing) at a deviator stress of 154.2 kN/m^2 and at an axial strain of 3.5 per cent (see Figure 5.19). This was a total piston load of 3.03 kN (or 308.8 Kg). When the sand was mixed with 66 m^2/m^3, Type 7, 50 x 100 mm size mesh elements, it failed at a deviator stress of 381 kN/m^2 and at 5.0 per cent axial strain (see Figure 5.19 and Table 5.10). The total piston load in this case was (2.57kN) 7.601 kN (or 774.8 Kg). One third of this load, being about 259 Kg, was applied to the specimen containing the same soil and sand mixture, as a dead load, for long term testing.

At this cell pressure, the total settlement of the triaxial sample composite, fifty one days after the total dead load was applied, was 0.36 mm (or 0.18% total strain). This makes an average rate of settlement-strain of:

$$\dot{\varepsilon} = 3.53 \times 10^{-3}\%/\text{day}$$

The results of this test are also shown in Figure 5.94.

Finally at 50 kN/m^2 cell pressure, Mid-Ross sand alone, failed at a deviator stress of 314 kN/m^2, and an axial strain of about 4.7% (see Figure 5.41). The maximum piston force was 6.23 kN (or 635 Kg). When the same sand was mixed with 66 m^2/m^3 (or 0.18% by weight) Type 7, 50 x 100 mm size, mesh elements, it failed at almost the same axial strain and at a deviator stress of 558 kN/m^2 (see Figure 5.41 and Table 5.10). The piston force at this stress, was 11.1 kN (or 1,131 Kg). Thus one third of this force (i.e. about 377 Kg) was used as a dead load, in the long term loading test.

The total settlement of the triaxial "composite" sample, fifty one days after the whole load was placed, at this cell pressure, was about 0.48 mm (or 0.24% total strain). This makes an average rate of settlement-strain of:

$$\dot{\varepsilon} = 4.71 \times 10^{-3}\%/\text{day}$$

The results of this test are shown in Figure 5.95. The total average density of all the long term loaded samples was about 1830 Kg/m^3 which was similar to all previous triaxial specimens.

5.11.3 Analysis of Results and Discussion

Because all the long term loaded samples were fully drained and dry with water flow and permeability not playing an important role, secondary consolidation, or plastic lag (creep) was pedominant in this case. Terzaghi's consolidation theory (Taylor's $\sqrt{\text{time}}$, Cassagrande's log time) deals only with the first factors and presents a good understanding of the hydrodynamic lag. Plastic lag however is complex and not yet fully understood. A theoretical comparison however had to be made between the previous experimental rates of settlement and theoretical predicting ones, studied by Mitchell, (1976).

According to Mitchell, rheological models could be developed in an effort to duplicate the stress-strain-time rsponse of a soil in terms of various arrangements of springs, dashpots and sliders. Phenomenological relationships developed by this method and previous theories, are empirical curve-fitting techniques, that do not necessarily imply anything about the mechanism underlying the deformation process. Such relationships however, are useful in practice as a basis for organisation of data for different soils and characterisation of creep. An example of a general stress-strain-time function is given by the expression:

$$\dot{\varepsilon} = A \cdot e^{a \cdot D} (t_1 / t)^m \qquad \text{(Mitchell, 1976)}$$

where; $\dot{\varepsilon}$: is the strain rate at unit time,

m : is the absolute value of the slope of the straight line on the log strain rate versus log time plot,

t_i : is a reference unit, for example 1 day,

a : is the slope of the linear part of the log strain rate versus stress plot,

D : is taken as the stress intensity under the influence of creep load,

t : is the time after start of creep,

A : is a parameter reflecting an order of magnitude for the creep rate under a given set of conditions, it is in a sense, a soil property. A is calculated by plotting log strain rate versus deviator stress at different values of times (See Figures 5.96 and 5.97).

This simple three parameter relationship (Mitchel, 1976) has been found suitable for the description of the creep rate behaviour of wide variety of soils. A minimum of two creep tests are needed to establish the values of A, a & m for a soil. If identical specimens are tested using different creep stress intensities, a plot of log strain rate versus log time yields the value of m, and a plot of log strain rate versus stess for different values of time can be used to find A and a, from the intercept at unit time and the slope respectively (see Figures 5.97, 5.98, 5.99 and 5.100).

Two tests were performed on Mid-Ross sand containing 66 m^2/m^3 mesh elements at zero cel pressure, for about 10 days. One sample was loaded by a 31.4 Kg dead load and other by a 23.5 Kg. The strain rate $\dot{\varepsilon}$ decreased with time, but increased with stress intensity.

Determination of m

The log strain-rate versus log time was plotted from the specimen loaded with the 31.4 Kg dead load, having a stress intensity of 16.33 kN/m^2, as shown in Figure 5.101. Thus the slope of the strainght line provided m.

Determination of A and a

Similarly the log strain-rate versus stress intensity was plotted for both specimens as shown in Figure 5.102. The intercept of the straight line (at theoretical D = 0) produced the parameter A. The slope of the same line also gave the parameter a.

Hence from Figures 5.102 and 5.101 m = 0.79543, A = 0.00365 and a = 0.04366. Substituting these values in equation:

$$\dot{\varepsilon} = A \cdot e^{a \cdot D} (t_1/t)^m$$

for t_1 being unit time = day, t being 45 days and stress intensity D being 73.6 kN/m^2,

$$\dot{\varepsilon} = 0.00365\, e^{0.04366 \times 73.6} [1/45]^{0.79543}$$

$$\dot{\varepsilon} = 0.00439\ \%/\text{day}$$

The experimental result of the same sample under the same stress intensity was;

$$\dot{\varepsilon} = 0.00388\ \%/\text{day}$$

Similarly at stress intensity D of 134 kN/m^2 and t being 51 days:

$$\dot{\varepsilon} = 0.055\ \%/\text{day},$$

whereas the experimental result was 0.00353 %/day. Finally at stress intensity D of 195.5 kN/^{2} and t being 51 days, the theoretical rate of settlement-strain was found,

$$\dot{\varepsilon} = 0.00365\, e^{0.04366 \times 195.5} [1/45]^{0.79543}$$

$$\dot{\varepsilon} = 0.814\ \%/\text{day}$$

whereas the experimental result was 0.00471 %/day.

Thus comparing experimental to theoretical results can be observed that in the theoretical ones, there is a distinct increase in $\dot{\varepsilon}$, as the stress intensity D increases. This, however, was not very distinct in the case of the experimental results. The reason was probably because their average rate of settlement was taken over a period of 50 days when it had already "levelled-off" with time; whereas in the case of theoretical results, the parameters D, and m were taken over a period of ten-day-testing, where creep settlement, at this initial stage, was faster. Additionally the "Three-Parameter Creep Settlement" theory is semi-empirical based more on consolidated clayey soils (where water pore pressure is still present) rather than granular dry ones, or stabilised composite materials. As mentioned previously, no satisfactory analysis has, so far, been invented for secondary (or tertiary) consolidation-settlement, or creep settlement. The above analysis has only been used to provide an approximate "information" and comparison to the actual results.

Judging finally, the long term loading test results from Figures 5.93, 5.94 and 5.95, the total settlements were only noticeable the first week of each test. Afterwards they became insignificant and the settlement rate "levelled-off" with time.

5.12 CONCLUSIONS

1. The use of randomly distributed-polymeric mesh elements in Mid-Ross sand, Leighton Buzzard sand and Colliery spoil have been shown to greatly improve their strength and beneficially alter their deformation properties. The behaviours indicated by all the previous data clearly demonstrate their ability to generate tensile strain resistance from the beginning of the test.

2. When the soil is mixed with mesh elements, the generated composite material gains an initial shear stress capacity increment (b, from Mohr's bi-linear envelope analysis), and a new angle of internal friction, which is almost the same in value to the one of soil alone, varying by 1^0 to 4^0. Increasing the percentage of mesh elements present, increased b both at peak and at residual stress conditions.

3. The 50 x 100 mm size mesh elements shows much higher performance when mixed with the above soils than the 50 x 50 mm one.

4. For any given percentage of mesh-element content, the 50 x 100 mm size increases b both at peak and residual stress conditions over that of the 50 x 50 mmm size.

For any given percentage mesh-element content, the change in angle of internal friction β for the 50 x 100 mm size is not significantly greater than the 50 x 50 mm size at peak stresses, but it may slightly increase at residual stresses.

5. The performance efficiency of the soil-mesh composite is much greater at low stress levels than at higher levels.

6. Mesh elements when mixed with granular soils, significantly reduce permanent deformations and irrecoverable axial strains, by almost a third, during repeated loading and generally improve their resilient stress-strain behaviour.

7. Long-Term-Loading tests showed that settlemens due to creep, of the same soils mixed with mesh elements, are significantly reduced.

8. West Highland Moraine and Pulverised Fuel Ash mixed with Type 7 mesh elements, show much less improvement than sands and Colliery spoils. This is probably due to a inferior interlock-bond between mesh Type 7 and their particles. A smaller grid-aperture type of mesh is therefore suggested or another type of mesh having greater rib-depth, or larger interface area than Type 7, in order to form greater passive resistance with the fine particles, during tensioning.

9. The rate of applied strain (or deformation) does not seem to affect the stress-strain characteristics of granular soils mixed with randomly oriented mesh elements.

CHAPTER SIX

MODEL FOOTING TESTS

CHAPTER SIX
MODEL FOOTING TESTS

6.1 GENERAL

In order to ensure that the previously described improvements in soil strength behaviour (when mesh elements were used) were not only limited to CBR and triaxial tests and that they could be measured in a simple soil-mesh system, model footing tests were undertaken.

6.2 TEST PROGRAM

A selection of the mesh-element Types that produced high CBR values, such as Types 5340, 2, 4, 5, 6 and 7, was made in this Chapter, and they were all tested-mixed with Leighton Buzzard sand in the model footing tank. A further selecton of the mesh element Type that produced the highest bearing stress was also made, and this was also tested mixed with Mid-Ross sand.

The model footing apparatus, compaction equipment and testing procedure, are all described in Chapter 3. The test program in this section was as follows:

1. Leighton Buzzard sand was compacted alone in layers of 25.5 mm by the wood-rammer described in Chapter 3. A series of tests were carried out in this way and the applied pressure due to the model footing plate, versus the percentage of penetration was established. After the sand's failure, the stress recovery versus

penetration during unloading was recorded in each case.

2. Leighton Buzzard sand was then mixed with Type 5340, 50 x 50 mm size, mesh elements, at 66 m^2/m^3 mesh content. The mixture was placed inside the model tank and compacted as before, forming a layer of 0.5B thickness on top of sand alone. (B in this case was the width of the model footing). Hence plate loading testing proceeded until the sand/mesh mixture and sand alone failed.

3. Similarly the same sand was mixed with 66 m^2/m^3, Type 5340, mesh elements and was placed inside the model tank, of thickness B. Hence loading proceeded the usual way, until sand and mixture failed.

4. The same procedure was repeated for the same sand and mesh element content, placed in thicknesses of 2B, 3B and 4B in turn.

5. Leighton Buzzard sand was mixed with 66 m^2/m^3, Type 2 mesh elements and was placed on top of sand alone in a layer having a thickness of 0.5B. Hence testing took place. The same testing was then repeated for B, 2B, 3B and 4B thicknesses in turn.

6. The same procedure was repeated for the same thicknesses of Leighton Buzzard sand mixed with 66 m^2/m^3 mesh element content, using mesh-element Types 4, 5, 6 and 7 in turn.

7. Similarly Mid-Ross sand was compacted alone in layers of 25.5 mm and after a series of repetitive tests, its behaviour of applied

stress versus penetration due to loading of the model footing, was established. After the sand's failure, stress recovery versus penetration was recorded during unloading.

8. Additonally the same sand was mixed with 66 m^2 per cubic metre of dry soil, Type 7 mesh elements and was placed inside the model tank, on the top of the sand alone, forming a layer of 0.5B thickness. Hence testing proceeded.

9. Finally the same testing procedure was repeated for the same soil mixed with 66 m^2/m^3, Type 7 mesh-content, placed inside the model tank in layers having thicknesses B, 2B, 3B and 4B respectively.

The stress recovery versus penetration after failure, was only recorded for the cases of Mid-Ross or Leighton Buzzard sands mixed with mesh elements at 2B thicknesses. The whole testing program is well listed in Tables 6.1 and 6.2. Due to the narrow width of the model tank, only the 50 x 50 mm size mesh elements were attempted.

6.3 TEST RESULTS AND ANALYSIS

The average overall density of Leighton Buzzard sand alone, in the model tank, was 1560 Kg/m^3 and its average porosity 41.1%. This made the sand loose to medium dense, having a relative density of about 34.5%. The overall average density of the sand/mesh-element mixture was similarly about 1575 Kg/m^3. This similarly indicates a small increase in the density of the mixture due to the presence of mesh elements, as was experienced in the previous chapters. These values however are slightly lower than the ones achieved in CBR and triaxial testing. This was probably due to the non B.S. compactive effort employed in the model footing tests.

Similarly in the case of Mid-Ross sand alone, its average overall density was found to be about 1765 Kg/m^3 having an average porosity value of 34.3%. This made the sand medium dense to dense, having a relative density of 58.2%. When this sand was mixed with mesh elements the overall average density of the mixture became about 1780 kN/m^2 showing evidence of a slight increase, due to the presence of the polymer as before. These values were slightly lower than the ones achieved in CBR and triaxial testing due to the non B.S. compactive effort involved.

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6.3.1 Model Footing Tests on Leighton Buzzard Sand Alone and Mixed with Various Types of Mesh Elements

(a) Leighton Buzzard sand alone

The ultimate bearing capacity of Leighton Buzzard sand alone, using Terzaghi's formula for a *strip* footing was:

$$q_{ult.} = cNc + \gamma Z\, Nq + 0.5\gamma B\, N_\gamma$$

since c = 0 and Z = 0,

$$q_{ult.} = 0.5\gamma B\, N_\gamma$$

where: Φ at failure (from triaxial testing) $\sim 43^o$

γ = unit weight of soil = 15.31 kN/m^3

B = width of footing = 0.075 m

Taking Feda's coefficient N_γ = 466.3 (Foundation Engineering Handbook by Winterkorn & Fang, 1975)

$$q_{ult.} = 0.5 \times 15.31 \times 0.075 \times 466.3$$

Hence $q_{ult.}$ = 267.72 kN/m^2

The experimental value of the maximum bearing capacity at peak stress in this case, was found to be 316 kN/m^2 (see Figure 6.1). This occurred at a penetration percentage (δ/B) of 9.0%. At residual stress state however (when δ/B = 20%) the ultimate stress was 92 kNlm^2 as shown in the same figure and summary Table 6.1.

(b) <u>Leighton Buzzard sand/Type 5340 mixture</u>

When Leighton Buzzard sand was mixed with 66 m^2/m^3 Type 5340, mesh elements forming a stabilised layer of thickness 0.5B the applied pressure at peak, became 400 kN/m^2 and at residual stress state, 217 kN/m^2. The improvement ratios in both cases were 1.26 and 2.36 respectively. The percentag-penetration (δ /B) that the sand composite failed at in this case was 10.6%. When the thickness of the stabilised layer became B the maximum applied pressure reached 546.5 kN/m^2 and at residual stress state, 281 kN/m^2. The improvement ratios in these cases were 1.73 and 3.06 respectively (see Figure 6.3 and summary Table 6.1) and the percentage penetration at failure was 13.5%.

By increasing the volume of the stabilished layer and consequently its depth to 2B, keeping the same area of mesh elements per unit volume, the applied maximum pressure at peak became 732 kN/m^2 and at residual stress state, 485 kN/m^2, as shown in Figure 6.3. The improvement ratios in this case were 2.31 and 5.26 and the performance efficiencies 131.6% and 427% respectively. The percentage penetration (δ/B) at failure was 15%.

Increasing the depth of the stabilised layer further to 3B, while keeping the same (66 m^2/m^3) mesh content, the maximum applied pressure at failure became 626 kN/m^2, and the pressure at residual stress state 574 kN/m^2. The improvement ratios and performance efficiencies in this case were 1.98 (or η= 98.1%) and 6.24 (or η = 524%) respectively. The percentage penetration at failure (δ/B) was 15.5%.

Finally making the thickness of the stabilised layer 4B (which almost covered the total height of the model tank) the total stress at peak became 710 kN/m^2 corresponding to an improvement ratio of 2.25 and a performance efficiency of 125%. The percentage penetration at this stage was 17.0%. At residual stress state the applied pressure became 680 kN/m^2, corresponding to an improvement ratio of 7.39 and a performance efficiency of 639% as shown in Figure 6.3 and summary Table 6.1.

(c) Leighton Buzzard sand/Type 2 mixture

When Leighton Buzzard sand was mixed with 66 m^2/m^3, Type 2 (shown in Figure 3.14) mesh elements forming a composite layer (or stabilised layer) of 0.5B thickness, over the sand alone, the peak stress at failure was 578 kN/m^2 and the stress at residual state 325 kN/m^2, as shown in Figure 6.4. The improvement ratios in this case were 1.83 and 3.53 respectively and the percentage penetration at failure 13.3%. By making the thickness of the stabilised layer B, the maximum stress at faiure became 677 kN/m^2 at a percentage penetration of 13.1%. The pressure at residual stress state also became 430 kN/m^2. The improvement ratios in both stress states were 2.14 and 4.67 respectively.

When the composite layer was increased to 2B thickness, the peak and residual stesses were 760 and 650 kN/m^2, corresponding to improvement ratios of 2.40 and 7.06 respectively. The percentage penetration at peak in this case, was 12.5%. By making the composite layer 3B, the peak and residual stresses however, became 680 and 530 kN/m^2, with improvement ratios of 2.15 and 5.80 respectively. The percentage penetration at peak in this case, was about 11%. Finally, increasing the stabilised layer and making it 4B thick, the peak and residual stresses became 757 and 630 kN/m^2, resulting in improvement ratios of 2.4 and 6.85, as shown in Figure 6.4 and summary Table 6.1. The penetration percentage at failue in this case was 12.2%.

(d) <u>Leighton Buzzard sand/Type 4 mixture</u>

Similarly Leighton Buzzard sand was mixed with 66 m^2/m^3, Type 4 mesh elements and the mixture was placed compacted inside the model tank over the sand alone, at a tickness 0.5B. the maximum applied stress at failure was 475 kN/m^2 occurring at a penetration δ /B of 14%. The applied pressure at residual stress state in this case, was 293 kN/m^2 shown in Figure 6.5. The improvement ratios in both cases, were 1.50 and 3.16 respectively. By increasing the thickness of the stabilised layer to B, the peak and residual stresses became 594 and 509 kN/m^2 corresponding to improvemet ratios of 1.87 and 5.52. The δ /B penetration in this case, at failure was 14.6%. Increasing further the thickness of the stabilised layer and making it 2B, keeping the same mesh element content, the peak and residual stresses became 650 and 557 kN/m^2 respectively. The δ /B penetration at peak, here was 14.3% and the corresonding improvement ratios at both stress states, were 2.05 and 6.05 respectively.

Making the thickness of the stabilised layer 3B, the stresses at peak and residual stress-states were 585 and 424 kN/m^2 resulting in improvement ratios of 1.84 and 4.61 respectively. The (δ /B) percentage penetration at failure in this case was 12.0%. Finally, increasing the stabilised layer to 4B, the stresses at peak and residual stress-states became 590 and 450 kN/m^2, resulting in improvement ratios of 1.86 and 4.90 respectively, as shown in Figure 6.5 and summary Table 6.1. The percentage penetration at failure in this case was 14.3%.

(e) Leighton Buzzard sand/Type 5 mixture

Similarly Leighton Buzzard sand was mixed with 66 m^2/m^3, Type 5 mesh elements and the mixture was placed compacted inside the tank over the sand alone at a thickness of 0.5B. The maximum applied stress at failure was 448 kN/m^2 occurring at a percentage penetration, δ/B, of 9.6%. The residual "stress state" pressure in this case was 172 kN/m^2 resulting in improvement ratios, at peak and residual, of 1.42 and 1.87 respectively (see Figure 4.6). By increasing the thickness of the composite to B, the peak and residual stresses became 594 and 509 kN/m^2 resulting in improvement ratios of 1.87 and 4.73 respectively, over those of sand alone. The percentage penetration at which failure took place in this case was 14.6%. Increasing the thickness of the stabilised layer further and making it 2B, the peak and residual stresses became 650 and 557 kN/m^2 resulting in improvement ratios of 2.05 and 6.74 respectively. The percentage penetration at which peak stress took place was about 14.3%. Increasing further the thickness of the composite layer, to 3B, the peak and residual stresses became 582 and 424 kN/m^2 resulting in improvement ratios of 1.85 and 5.98 respectively. The penetration δ/B that the peak stress occurred in this case was about 12.0% as shown in Figure 6.6 and summary Table 6.1.

Finally making the thickness of the stabilised layer 4B (i.e. covering the whole tank) the pressures at peak and residual stress states became 780 and 704 kN/m^2. The improvement ratios in this case were 2.47 and 7.65 respectively. The penetration that maximum stress occurred at was 13.5%.

(f) Leighton Buzzard sand/Type 6 mixture

The same sand was then mixed with 66 m^2/m^3, Type 6 mesh elements and the mixtures was placed, compacted, inside the model tank forming a layer of thickness 0.5B, over the sand alone. The peak stress in this case was 472 kN/m^2 at a penetration of 9.56% and the residual was 278 kN/m^2. The improvement ratios in both cases were 1.49 and 3.02 respectively. Increasing the stabilised layer and making it have a thickness of B, the stresses (peak and residual) also increased and became 545 and 428 kN/m^2 resulting in improvement ratios of 1.72 and 4.65 respectively. The penetration that maximum pressure took place at in this case, was about 11.4% as shown in Figure 6.7 and Table 6.1.

Further increase in the thickness of the composite layer (i.e. 2B) produced peak and residual stresses 716 and 690 kN/m^2 respectively. The improvement ratios in this case were 2.26 and 7.50 and the percentage penetration at which peak stress took place was 12.3%. Increasing further the composite layer thickness to 3B, the peak and residual stresses became 580 and 520 kN/m^2 resulting in improvement ratios of 1.83 and 5.65 respectively. Finally making the composite layer 4B, resulted in peak and residual stresses of 780 and 715 kN/m^2 and improvement ratios of 2.47 and 7.77 respectively. The percentage penetration that maximum pressure occurred was 12.2%.

(g) Leighton Buzzardtsand/Type 7 mixture

Mixing the same sand with 66 m^2/m^3, Type 7, mesh elements and placing it compacted, inside the tank, in a layer of thickness 0.5B, over sand alone, resulted in peak and residual stresses of 636 and 437 kN/m^2 respectively. The improvement ratios in this case, became 2.01 and 4.75. The percentage penetration for peak stess was 11.9% as shown in Figure 6.8 (see also summary Table 6.1). By increasing the stabilised layer and making it B, the peak and residual stresses became 693 and 382 $kN/^2$ resulting in improvement ratios of 2.20 and 4.21 respectively.

Further increase in the thickness of the composite layer to 2B resulted in an increase of both peak and residual stresses which became 963 kN/m^2. The maximum applied stress in this case, took place at 20% penetration, showing a tendency for further increase (see Figure 6.8). The improvement ratios in this case were 3.05 and 10.46. By increasing the stabilised layer further, and making it 3B the peak, or residual stress, took place at almost 20% (showing a tendency for further increase) and became 888 kN/m^2 as shown in Figure 6.8. The improvement ratios on this occasion were 2.81 and 9.65 respectively. Increasing finally the thickness of the composite layer and making it 4B, resulted in a peak and a "residual stress" of 853 kN/m^2 at 20% penetration, since the stabilised layer did not show signs of failure. The improvement ratios in this case were 2.70 and 9.27 respectively.

6.3.2 Discussion of Results

Comparing all the results of the previous cases when Leighton

Buzzard sand was mixed with the various Types of mesh element at various layer-thicknesses; it can be observed that there was a gradual increase in improvement ratio up to a layer thickness of 2B. After that the improvement ratios did not seem to increase any further. Thus a kind of "saturation zone" takes place after 2B where no more benefit is gained out of the mesh elements. The relationship between improvement ratio and penetration at various depths of stabilised layer is shown in Figure 6.14.

Similarly comparing the results of the same sand mixed with various mesh types at 2B layer-thickness (see Figure 6.9) one can observe that the mesh type that produced the highest stresses was Type 7. This Type was the most beneficial mesh for this type of sand.

Photographs were taken (see Figure 3.6) at various penetrations during each test and were examined in pairs in a stereo-photogrammetric viewer. The results showed that the soil/mesh element mixtures formed a typical Prandtl-failure-pattern at peak or residual stress levels.

6.3.3 Effect of Mesh Element on the Unloading Characteristics

When unloading the footing tests, a further significant difference was observed between the behaviour of the sand and the sand-mesh mixture. As indicated in Figures 6.10, 6.11 and 6.12, where a layer of the sand-mesh Type 7 mixture was present, almost 20 per cent of the imposed vertical settlement was recovered, which was 4 times that for the soil alone. Similarly in all other mesh Types

(Figure 6.10) recovery was about 2 to 3.5 times that for the soil alone. This was probably due to the partial recovery of the strains in the mesh elements and this improved elasticity system could well prove to be a very important property of soil-mesh mixtures, particularly where repeated loading is involved.

This behaviour was also similar to the case of Mid-Ross sand when mixed with Type 7 mesh elements as shown in Figure 6.12. When unloading the foting tests, where a layer of Mid-Ross sand-mesh mixture was present, almost 20 per cent of the imposed vertical settlement was recovered, which again was about 4 times that for the soil alone.

6.4 MODEL FOOTING TESTS ON MID-ROSS SAND ALONE AND MIXED WITH TYPE 7 MESH ELEMENTS

Since Type 7, 50 x 50 mm size, mesh elements showed superior results than all other types when they were mixed with Leighton Buzzard sand, an attempt was made to proceed with further testing this time, mixing Type 7 with Mid-Ross sand.

6.4.1 Test Results and Analysis

(a) Mid-Ross sand alone

The ultimate bearing capacity of Mid-Ross sand alone using Terzaghi's formula for a strip footing was:

$$q_{ult.} = cN_c + \gamma Z N_q + 0.5\gamma B N_\gamma$$

since $c = 0$ and $Z = 0$,

$$q_{ult.} = 0.5\gamma B N_\gamma$$

Φ at failure (from triaxial testing) $\sim 45^0$

γ = unit weight of soil = 16.48 kN/m

B = width of footing = 0.075 m

Taking Feda's coefficient N_γ = 768.8 (Foundation Engineering Handbook by Winterkorn & Fang, 1975)

$$q_{ult.} = 0.5 \times 16.48 \times 0.075 \times 768.8$$

Hence $\quad q_{ult.} = 475.1$ kN/m^2

The experimental value of the maximum bearing capacity in this case was found to be 470 kN/m^2 (see Figure 6.2) and this occurred at a penetration δ/B of 9.5%. At residual stress state however (when δ/B = 20.0%) the ultimate stress was 270 kN/m^2 as shown in the same Figure and summary Table 6.2.

(b) Mid-Ross sand/Type 7 mixture

Mid-Ross sand was mixed with 66 m^2/m^3, Type 7 mesh elements of 50 x 50 mm size, and the mixture was placed compacted, inside the model tank at a layer thickness of 0.5B over the sand alone. The peak stress at failure in this case after testing , became 950 kN/m^2 showing a tendency towards further increase (see Figure 6.13). The residual stress state in this case was taken to be the same as the peak with a penetration δ/B of 20.0%. The improvement ratios in both cases were 2.02 and 3.25 respectively.

When the stabilised layer increased to thickness B, keeping always the same mesh element content of 66 m^2/m^3, the peak stress became 960 kN/m^2 and the residual 930 kN/m^2. The percentage penetration that maximum stress took place at was 17.0%. The improvement ratios in this case were 2.04 and 3.44 respectively (see summary Table 6.2). When the thickness of the stabilised layer increased to 2B, the peak stress at failure became 1133.7 kN/m^2 and the residual 1062.1 kN/m^2 resulting in improvement ratios of 2.41 and 3.93 respectively. The percentage penetration at which peak stress took place was about 16.0%. By increasing the thickness of the stabilised layer and making it 3B, the stress at peak became the same as the residual, at 1221 kN/m^2 (see Figure 6.13). From this figure it can be seen that the sand/mesh element mixture has not fully failed but still has a tendency to increase its bearing capacity further.

When finally the thickness of the stabilised layer increased to 4B, the maximum applied pressure of the footing at failure, was 1164.4 kN/m^2 and took place at a percentage penetration of 15.3%. At the residual stress state, the pressure was 1104.2 kN/m^2. The improvement ratios in both peak and residual stress states, were 2.48 and 4.09 respectively.

6.4.2 Discussion of Results

Comparing the results of Mid-Ross sand alone to those of the same sand mixed with 66 m^2/m^3 (or 0.18% by soil weight) Type 7 mesh elements, very large improvements were obtained at all strain levels by their use. These improvements were indeed very similar to those measured in the triaxial tests in terms of both strength and deformation characteristics. The ultimate bearing stresses in the case of Mid-Ross sand/mesh element mixtures, showed a significant improvement up to an increase in depth of the stabilised layer of twice the breadth of the footing. After this depth was exceeded, no significant improvement in maximum applied stresses was observed. As in the case of Leighton Buzzard sand mixed with mesh elements, the depth of 2B appears to be a kind of "saturation zone" in terms of mesh-element strengthening benefit (see Figures 6.8 and 6.13).

The improvement ratios of Mid-Ross sand and mesh Type 7 mixtures, increased with the depth of the composite layer at all strain levels as shown in Figure 6.15. The same increase in improvement ratios was observed in the case of Leighton Buzzard sand mixed with Type 7 mesh elements as shown in Figure 6.14. Mid-Ross sand-mixtures however, showed more initial increase than the later ones. This could probably be due to the denser state of Mid-Ross compared to Leighton Buzzard sand under the same compactive effort, or due to the coarser and sub-angular nature of Mid-Ross sand's particles forming a better interock mechanism with mesh Type 7. This phenomenon,however, of Mid-Ross sand showing superior results to Leighton Buzzard when both were mixed with Type 7 mesh elements, at the same content, was not predominant in the cases of CBR and triaxial testing. This again could probably be due to the irregular compactive nature of Leighton Buzzard sand, being a uniform material, and the fact that both sands were then compacted by British Standard compaction. Additionally it must be borne in mind that the plane strain model footing tank apparatus is limited in accuracy since it suffers from extensive side-wall friction and its limited (75 mm) width in comparison to the average individual size of a 50 x 50 mm mesh element probably affects the isotropy of the mixture. A large model footing tank however has been constructed (Kenny, M. 1985) using lubricated membranes at the side-walls, in order to eliminate these errors.

6.5 CONCLUSIONS

The behaviours indicated by all the previous results clearly demonstrate the ability of the randomly distributed polymeric mesh elements in Leighton Buzzard or Mid-Ross sand, to greatly improve their strength and beneficially alter their deformation properties by generating tensile strain resistance from the beginning of the test.

The improvements obtained at all strain levels, from the model footing tests, are very similar to those measured in the triaxial tests in terms of both strength and deformation characteristics, and confirm that the improvements measured in the triaxial testing are not specific to that test.

From the whole range of types of mesh elements tested in the model footing apparatus, Type 7 proved to be the most beneficial in terms of bearing stresses, as it probably formed a superior interlock mechanism with both sands. This also is very similar to the results of the CBR testing in terms of mesh Type choice.

When unloading the footing tests of sand-mesh mixtures, the imposed vertical settlement recovered is between two to four times that of the soil alone. This is probably due to the partial recovery fo the strains in the mesh elements. This improved elasticity could well prove to be a very important property of soil-mesh mixtures, particularly where repeated loading is involved.

Additionally from the same model footing test results, it can be said that when mesh elements are mixed with a granular soil, at 66 m^2/m^3 content, and placed under a square footing, the effective depth of the stabilised layer should be approximately twice the breadth of the footing in order to receive maximum strength benefit from them.

CHAPTER SEVEN

GENERAL CONCLUSIONS

CHAPTER SEVEN

GENERAL CONCLUSIONS

7.1 GENERAL

The use of randomly distributed polymeric mesh elements in Mid-Ross and Leighton Buzzard sands has been shown to greatly improve their strength and beneficially alter their deformation properties. Triaxial tests and full-scale bearing capacity trials using other soil types mixed with mesh elements, show that similar levels of improvement are obtained. The behaviours indicated by all the previous data clearly demonstrate their ability to generate tensile strain resistance from the beginning of the test and increase the stress resistant properties of soils without any reduction in soil density or ductility.

7.2 THE EFFECT OF MESH TYPE

Tests using different types of mesh show that the tensile load-strain behaviour, flexural stiffness, rib shapes and sizes and opening sizes of the mesh, are all important factors influencing the behaviour of the soil-mesh mixture. Mesh Type 7 proves to be more beneficial mixed with Mid-Ross sand, Leighton Buzzard sand and Colliery spoils whereas another type of mesh having a greater depth of rib and smaller aperture size would be more beneficial mixed with West Highland Moraine, Pulverised Fuel Ash and any other fine-particle soil.

7.3 THE EFFECT OF MESH ELEMENT SIZE

The size of mesh elements present in any mixture is also an important factor influencing the behaviour of the soil-mesh composite. The 50 x 100 mm size mesh shows much higher performance when mixed with soils than the 50 x 50 mm square one due to its higher anchorage length capacity in interlocking with soil particles.

7.4 THE EFFECT OF MESH ELEMENT CONTENT

The mesh element content is another important factor influencing the behaviour of the soil-mesh mixture. As the mesh content increases, the soil/mesh composite strength also increases. At percentages of mesh content in excess of about 0.6% however, the dry density of the mixture rapidly decreases, indicating that the meshes are forming additional void space in the soil which is not desirable. This can cause excessive elastic surface effects and significant reduction in strength.

7.5 THE EFFECT OF MESH ELEMENTS ON THE SOIL'S STRENGTH PARAMETERS AND STRENGTH CHARACTERISTICS

1. The basic responses of the mixture in the various tests, are essentially similar to those of the soil alone. When the soil is mixed with mesh elements, the composite material gains an initial shear stress increment b, from the Mohr envelope analysis, which increases with the increase in percentage of mesh element content. Additionally

a new angle of internal friction is produced which is almost the same in value to the one of soil alone varying only by 1^{0} to 4^{0}. Hence a new design technology will not need to be employed. Conventional design methods using modified soil parameters should be adequate.

2. The performance efficiency of the soil mesh-element composites is much higher at low stress levels, than at high ones. An elementary initial stress however is needed in the form of either a surcharge, or lateral confinement, in order to make the mesh elements effective.

3. Soil reinforcement using sheets of mesh material placed along the directions of the principal tensile strains proved more efficient at low stress levels than soil strengthening using randomly distributed mesh elements. The difference was not all that distinct at high stress levels.

4. During repeated loading mesh elements mixed with a soil, greatly improve its resilient stress-strain behaviour by reducing permanent deformations and irrecoverable axial strains.

5. When a granular soil is mixed with randomly orientated polymeric mesh elements, the rate of applied strain does not seem to affect its strength characteristics.

6. During long term loading the settlements of soil-mesh mixtures due to creep are very small.

7. The bearing capacity increases for any given settlement when mesh elements are mixed with a soil. Where a layer of soil-mesh mixture is present, a large percentage of the imposed vertical settlement is recovered during unloading and this improved elasticity system is a very important property of soil-mesh mixtures, particularly where cyclic loading is involved.

7.6 THE EFFECT OF MESH ELEMENTS ON THE PERMEABILITY OF SOILS

According to all the test results obtained during this project applied to several different soil types, several conclusions can be drawn. As regards the flow of water through the mesh/soil mixtures, it can be stated that the inclusion of polymeric mesh elements has no detrimental effect whatsoever on the permeability of the soil.

RECOMMENDATIONS FOR FUTURE WORK

RECOMMENDATIONS FOR FUTURE WORK

Further laboratory work should be carried out on West Highland Moraine and Pulverised Fuel Ash in order to assess the most appropriate type of mesh element suitable for their fine particles in order to achieve similar performance efficiencies to the ones of Mid-Ross or Leighton Buzzard sands when they mixed with mesh Type 7.

Additonal laboratory work should be performed on clays and generally clayey soils, mixed with Type 7 mesh elements, or alternative meshes possessing greater depth of rib and consequently larger interface area, forming a greater passive resistance during tensioning.

An attempt should be made to increase the compactive effort far beyond British Standards in order to detect any significant increase in benefit of mesh elements, as Hoare (1979) suggested as a result of his own research.

Further model footing tests should be attempted on soils mixed with mesh elements in a model tank of considerable width using lubricated interior walls in order to eliminate side friction.

The effect should also be investigated of the temperature of the soil/mesh composite material, by performing tests at different laboratory temperatures in order to examine any reduction in strength. This would enable design engineers to assess the strength behaviour of soil/mesh mixture used at very superficial ground levels, in arid climates.

Since mesh-elements require a small amount of confinement (or surcharge) in order to become effective, it would be interesting to research this matter and assess numerically these maximum and minimum confinement pressure-levels.

Additionally research should be carried out in mixing larger (than 90 m^2/m^3) amounts of mesh elements in soil by using mechanical concrete mixers or other industrial means. Some ideas about the mixing, batching and uses of mesh elements on a commercial scale, are given in Appendix G.

Finally the economics of this product should be assessed in terms of production cost, mixing and batching in comparison to other conventional soil stabilisers.

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