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Developments and innovations in reinforced soil technology

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1 INTRODUCTION

A number of innovations and studies in reinforced soil technology have been introduced in recent years. Improved reinforcing materials have been introduced which include new forms of conventional geogrid reinforcements, composite materials providing combined drainage and reinforcement, and electrically conductive reinforcements. Research has resulted in a better understanding of the residual creep strength of geosynthetic materials and the influence of a seismic event on the long term strength of geosynthetic reinforcement. In addition the mechanism of reinforcement to provide primary stage construction platforms over super soft soil is now understood. Finally the use of reinforced soil in construction is being promoted by the introduction of new National Design Codes of Practice.

This report provides details of these developments together with examples of recent applications.

2 DEVELOPMENTS IN REINFORCING MATERIALS

2.1 *New Forms of Geogrids*

Traditional geosynthetic grid materials used for reinforcement are either formed as woven structures formed from highly orientated polyester fibres enclosed in a protective coating or drawn from a perforated sheet. The drawing/stretching process results in orientation of the molecules creating a torsionally stiff geogrid with integral joints. The orientation of the molecules increases the strength of the material where stretching has occurred. Full orientation does not take place at the joints resulting in a lower strength in these areas. Uniaxial reinforcing materials are produced by stretching the base material in one direction (longitudinal or machine direction); by stretching the material in the cross-machine direction a triaxial grid is formed. The materials used to form this type of grid are high density polyethylene (HDPE) or polypropylene (PP). Both materials produce high strength, rigid structures. The major difference between woven geogrids and the orientated structures produced by the drawing technique is the integrated joints produced by the latter technique. This permits the transverse elements of the grids to act as individual anchors without fear of damage occurring to the grid structure. Woven geogrids do not have integral joints and distortion of the grid junctions can result in rupture of the protective coating, exposing the reinforcing fibres to aggressive environments.

Geogrids can also be formed from geocomposite strip materials laid orthagonally and joined by spot welding or gluing of the outer casings. This technique can produce very high strength materials, but the load carrying capacity is limited to the longitudinal and transverse properties of the material. The joints are not integral to the strengthening elements and junction strength is very low in comparison to the strength of the longitudinal or transverse elements.

Recent advances in manufacturing technology have resulted in improved geogrid reinforcing products. Bishop and Horstmann (2000) have described a new form of geogrid reinforcement manufactured in Germany using solid flat bars as the structural elements. The construction process is to form the geogrid by laying the reinforcing elements on each other in the form of a grid and welding them together over the whole of the overlapping area of contact between the bars. The reinforcing bars are made with prestressed, molecular orientated polyester (PET). Welded geogrids made from solid flat bars are stiffer than woven/knitted grids, with junction strength depending upon the welding procedures. The material can be manufactured as a biaxial or uniaxial geogrid producing strengths up to 800 kN/m short-term tensile strength.. A feature of the new geogrid is that the elongation at break is significantly less than most polyester geogrid materials. In addition the material is robust with damage reduction factors being equivalent to other integral joint geogrids.

A similar manufacturing process in Holland has been described by Voskamp (2000) to produce a new range of geogrid products formed from highly orientated reinforcing elements in the form of straps. The geogrids are made up of separate straps welded together at the connections using a laser welding technique. Laser welding permits the use of highly stretched and molecularly orientated straps to be connected without losing the orientation of the molecules in the welded area. The new geogrid combines the strength properties of stiff grids with the advantages of flexible geogrids and can be made of various polymers. The welding process results in rigid (integral) connections between the reinforcing straps. Polyester (PET) grids are used for long term applications such as reinforced soil, and polypropylene (PP) grids are used for short term strength requirements such as road reinforcement.

The new Dutch geogrid is formed from black reinforcing straps orientated in the machine direction and two transparent weft straps. One weft strap is positioned above and one below the longitudinal straps, thus at every connection/junction there are two transparent straps. This structure is used to increase the bearing resistance of the geogrid in the soil and produces a symmetrical joint structure, Figure 1.

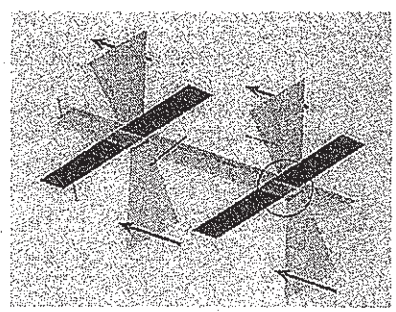


Figure 1 : Arrangement of the straps for welding

In forming the grid the laser light passes through the transparent strap and heats a thin layer of the black strap, Figure 2. This thin layer is important as this prevents distortion of the stretched molecules in the straps. The polymer materials of the transparent transverse straps and the longitudinal black straps mix and form the connection, Figure 3. The welding technique results in low losses in the tensile strength of the welded strap in comparison to the base product. Typically the losses are reported to be in the range 5-10 per cent. As the straps have better stress-rupture behaviour than yarns, the long term rupture strength is higher than equivalent geogrids formed from yarns.

The damage performance characteristics of the new Dutch geogrid are similar to the new German material.

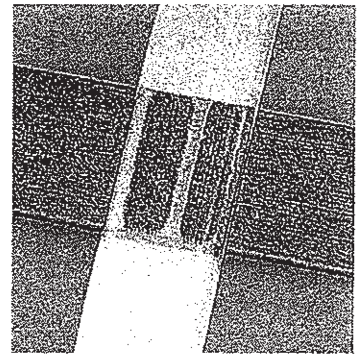


Figure 2 : Black straps in warp direction, transparent in weft direction

2.2 *Combined Reinforcement and Drainage*

The design of reinforced slopes is concerned with the provision of an adequate factor of safety on strength and the control of settlements to acceptable limits. Conventional design and construction methods have used granular materials due to their high shear strength and good drainage properties. Recent research (Zornburg and Mitchell, 1994) and long term case histories (Inada *et al*, 1978 and Fukuoka, 1998) have indicated that cohesive soils can be used in the construction of reinforced slopes if an adequate drainage system is provided in the structure of the slope.

When low permeability fills are loaded excess pore water pressures can be generated. This can result in a reduction in the available shear strength of the cohesive fill and also a reduction in the soil reinforcement bond requiring more reinforcement to provide an adequate bond length. The dissipation of excess pore water pressures results in consolidation and settlement of the reinforced structure, which can continue over time resulting in unacceptable face deflections.

The magnitude of excess pore water pressure present in a slope is a function both of the applied load and also the ability of the drainage system to dissipate the excess pore water pressure. At the base of a slope with no drainage provided large excess pore water pressures can develop. However if drainage is provided and complete dissipation of excess pore water pressure occurs before construction of the next layer the excess pore water pressure in the completed structure would only be a fraction of that otherwise present.

The *ideal* reinforcing material for cohesive soils requires the drainage characteristics of a nonwoven geotextile and the strength of stiffer/stronger reinforcing geosynthetics. Alternatively it is possible to combine existing materials (e.g. using a nonwoven drainage geotextile with a geogrid reinforcement).

Heshmati (1993) studied the effects of combin-

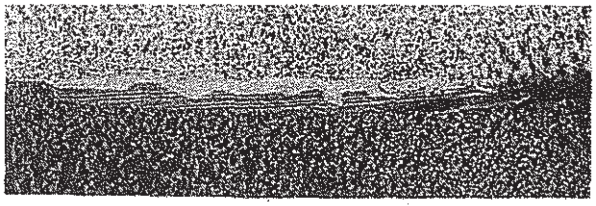


Figure 3 : Cross section of a welded area

ing a drainage material with grid reinforcement in clay soil. He concluded that the drainage and the reinforcement function were equally important in producing a stable and efficient structure. An important observation was that the method used to combine the drainage and reinforcing functions is critical. Simply placing a geotextile drain in conjunction with geogrid reinforcement can result in a *reduction in strength* as the presence of the drainage layer can lubricate the surface of the reinforcement. An essential requirement is that the combined functions of reinforcement and drainage have to be made *integral.*

A new innovative geosynthetic material has been introduced which conforms with Heshmati's (1993) findings relating to a reinforcing material which also provides drainage. The new geosynthetic consists of high tenacity polyester encased in a polyethylene sheath. The sheath both protects the load carrying elements and maintains the shape of the product which is profiled to provide a drainage channel on one side. The profiled strap has a thermally bonded nonwoven geotextile strip bonded on the shoulders of the drainage channel. The geotextile allows excess pore water pressure to dissipate while retaining the cohesive soil, Figure 4.

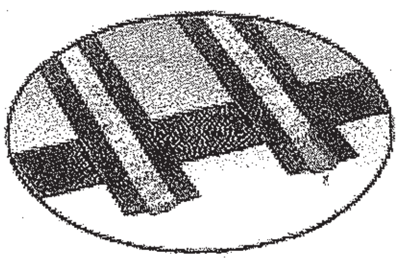


Figure 4 : Integral reinforcement and drain

Confirmation of the performance of the combined geogrid reinforcement and drainage material has been provided in a research programme conducted at the University of Newcastle, Kempton *et al* (2000). The programme had the following objectives:

1. To evaluate the effectiveness of a new combined drainage and reinforcement geosynthetic in dissipating excess pore water pressures under various confining stresses.
2. To evaluate the pullout resistance of the new geosynthetic when compared with a conventional geogrid of similar construction but with no drainage component.
3. To evaluate the horizontal flow characteristics of the material under various hydraulic gradients and confining pressures.
4. To determine suitable parameters for use in design for constructing steep slopes using cohesive fills.

The research was undertaken using modified Rowe cells and kaolin (English China Clay) which was chosen due to its low permeability and consistent properties. The apparatus was instrumented with pore water pressure probes located at various offsets above and below the composite drainage/reinforcement material. Displacement and volume transducers were also used to measure the displacement of the Rowe cells during application of load and volume of water leaving the cell through the geosynthetic during dissipation.

Two principle types of tests were carried out:

1. Dissipation tests at confining pressures of 50 and 100 kPa with continuous measurement of the excess pore water pressure in the cell and the volume of water leaving the cell.
2. Pullout testing after full and partial dissipation of excess pore water pressure at different confining pressures.

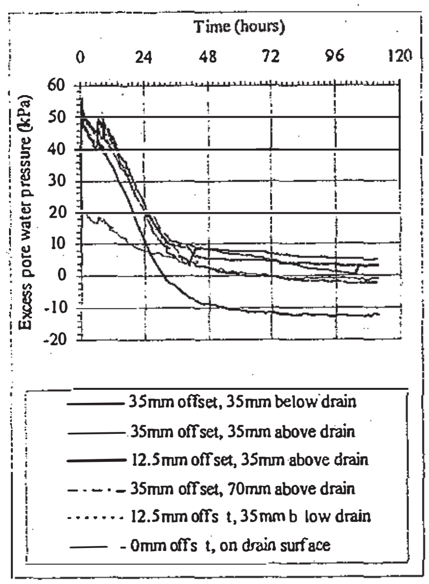


Figure 5 : Dissipation of excess pore water pressure for a confining pressure of 50 kPa

Dissipation tests were performed in the unit cell by only allowing drainage through the new geosynthetic. A confining pressure was applied and the excess pore water pressure generated was allowed to dissipate while continuous readings of the pore wa-

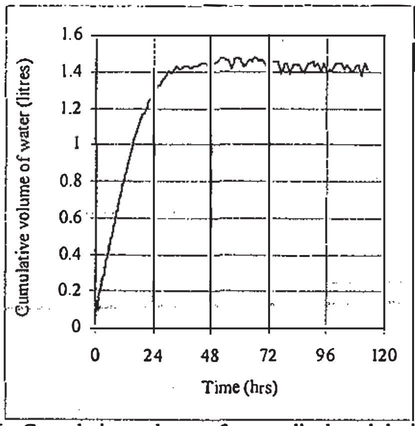


Figure 6 : Cumulative volume of water displaced during dissipation of excess pore water pressure under confining pressure of 50 kPa

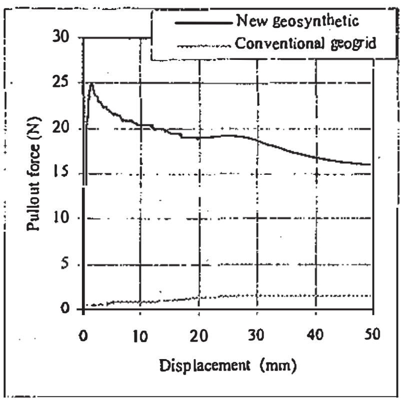


Figure 7 : Pullout results for new geosynthetic and conventional geogrid with no drainage component after dissipation of excess pore water pressure for 12 hours

ter pressure, the volume change and the displacement of the Rowe cells were taken. On the surface of the test specimen the excess pore water pressure reached approximately 40 percent and 30 percent of the applied load at confining pressures of 50 and 100 kPa respectively. The test results show that the pore water pressure reduces from the applied pressure to 20 percent of the applied pressure in a 36 to 42 hour period at confining pressures of both 50 and 100 kPa. No noticeable difference was observed in pore water pressure values measured above and below the test specimen even though the drainage channel was only on one side of the new geosynthetic. The test results for a confining pressure of 50 kPa are shown in Figure 5.

The volume of water displaced is a reliable indicator of the end of consolidation. Consolidation at confining pressures of 50 and 100 kPa occurs in a 36 to 42 hour period and is consistent with the pore water pressure measurements. The volume of water leaving the cell for a confining pressure of 50 kPa is shown in Figure 6.

Pullout testing on the new geosynthetic and on a similar geogrid with no drainage component was conducted after partial and full dissipation of excess pore water pressure in the unit cell. The pullout tests after partial dissipation were carried out after dissipating the excess pore water pressure for 12 hours under a confining pressure of 50 kPa. The new geosynthetic was found to have a much enhanced pullout resistance after partial dissipation, Figure 7. This is explained by the excess pore water pressure in the immediate vicinity of the new geosynthetic dissipating quickly thus allowing early development of bond between the reinforcement and the soil. The pullout resistance of the new geosynthetic peaks at a small displacement, 2.0 mm, where the conventional geogrid does not appear to reach a peak pullout load.

Full dissipation of excess pore water pressure was assumed when the pore water pressure valves reached 10 percent of the applied confining pressure in the immediate vicinity of the geosynthetic. Pullout tests after full dissipation were conducted at confining pressures of 50 kPa and 100 kPa. A substantial increase in pullout resistance of the new geosynthetic over a similar conventional geogrid with no drainage component was observed with the peak pullout resistance increased by approximately 33 percent while at smaller strains a 500 percent increase in pullout resistance was observed, Figure 8. The peak pullout resistance for the new geosynthetic was reached at a displacement of 3.5 mm whereas the conventional geogrid reached a peak pullout load at 7 mm.

The water flow or transmissivity of the new geosynthetic is of importance in removing the dissipated water from the soil. The ability of a drainage material to remove water from a soil structure is a function of the confining pressure, length of drainage channel, permeability of soil and magnitude of excess pore water pressure in the soil. In a reinforced slope the hydraulic gradient along any drainage path will vary according to its proximity to the face of the

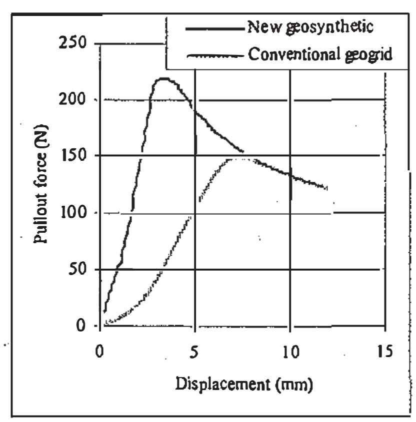


Figure 8 : Pullout results for new geosynthetic and conventional geogrid with no drainage component after full dissipation of excess pore water pressure

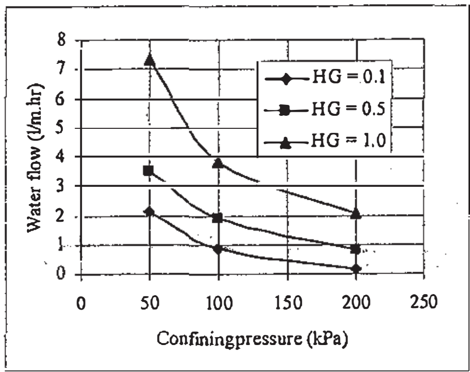


Figure 9 : Water flow through new geosynthetic under various confining pressures and hydraulic gradients


Figure 10(a)


Figure 10(b)

slope and the magnitude of the excess pore water pressure present.

The transmissivity of the new geosynthetic was measured over a 1 m long strip under confining pressures of 50, 100 and 200 kPa and at hydraulic gradients (HG) of 0.1, 0.5 and 1.0. The test apparatus consisted of a channel that held a strip of the new geosynthetic. A confining pressure could be applied over the full length of the strip and water introduced under different hydraulic gradients. The test results are presented in Figure 9.

Samples of the new geosynthetic removed from the test apparatus after consolidation testing showed minimal signs of clogging and very little wash through of fines into the drain.

An example of the combined reinforcement and drainage material is shown in Figure 10(a) and (b).

2.3 *Electrically Conductive Reinforcement*

It has been found that new uses and applications of geosynthetics can be created by incorporating electro-kinetic phenomena with the existing traditional functions of geosynthetic materials. This entails the creation of a new range of geosynthetic materials (or geocomposites) that are electrically conductive. An electrically conductive geosynthetic is referred to as an electro-kinetic geosynthetic (EKG).

The use of EKG reinforcement represents an extension of current reinforced soil technology which has important implications with respect to the type of fill which can be used with these structures.

Reinforced soil is usually constructed with good quality cohesionless fill; however, this may not always be available. The use of cohesive fill can be accommodated by the use of reinforcement which includes a drainage element as described in Section 2.2. However, the performance of the reinforcement can be further enhanced by making it electrically conductive thereby introducing electro-osmotic consolidation to reinforced soil technology. This has the potential benefit of:

1. increasing the rate of dissipation of positive pore pressures in the cohesive fill in excess of that which can be achieved using permeable reinforcement;
2. inducing additional consolidation (and associated increase in shear strength) to that obtained by the self weight of the fill material alone;
3. dissipating positive pore pressure at the soil/reinforcement interface even with impermeable reinforcement, thereby increasing reinforcement/soil bond.

EKG materials can take the form of single materials which are electrically conductive or composite materials in which at least one element is electrically conductive. A geocomposite which is both electrically and hydraulically conductive allows a single element to be used as an electrode, and at the same time removing the need to install a separate filter/drain as with metal electrodes. The principal conductive materials which have been considered for EKG use include: carbon fibre materials, conductively filled polymers and composites formed from conductive/non-conductive elements. An EKG material used for soil reinforcement consists of a

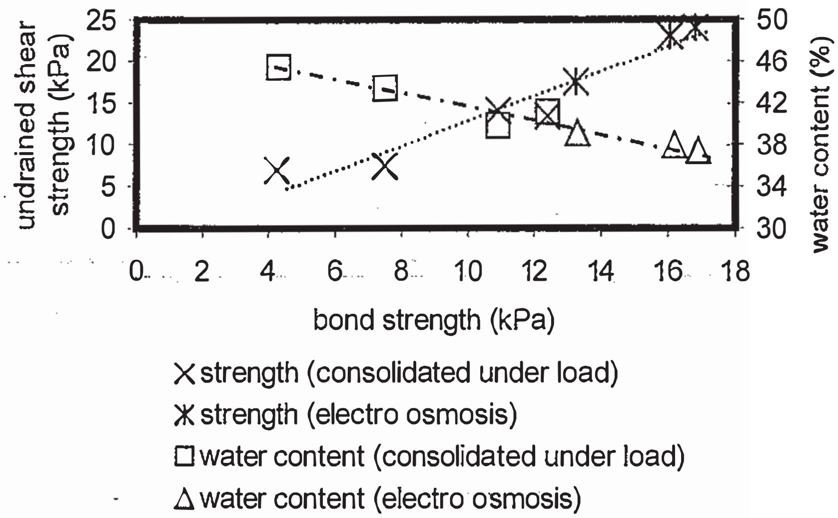


Figure 11 : Increase in shear strength and bond strength achieved with electro-osmotic consolidation using EKG reinforcement

conductor and a reinforcing element(s) which may be permeable or impermeable.

The benefits of using EKG reinforcement with fine-grained fill have been demonstrated in pullout tests (Jones *et al* 1996). Jones *et al* (1996) showed that the application of a potential gradient of 30 volts in addition to the surcharge loading perpendicular to an EKG reinforcing element caused a reduction in the water content of the soil ranging from 5.5 percent for soil consolidated at 350 kPa to 14 percent for soil consolidated at 110 kPa, together with a related increase in the shear strength. Similarly the increase in bond strength resulting from the electro-osmotic consolidation ranged between 54-210 percent, Figure 11. The results show that the increase in bond strength is the same irrespective of whether the soil was consolidated by external pressure or by electro-osmotic induced pressure. However, the application of a external pressure, or surcharge, is not possible during the construction of a reinforced soil structure, and an internally induced pressure, such as that generated using EKG reinforcement, is unique in producing a significant improvement in soil/reinforcement bond.

In order to demonstrate the effectiveness of EKG reinforcement in improving reinforced fill properties a full scale trial of electrically enhanced very soft cohesive reinforced soil using EKG geosynthetics has been undertaken, Jones and Pugh (2001).

The wall was built using a "wrap-around" design, using sandbags for the front face to temporarily retain the liquid fill. The ends of the trial wall were retained using conventional reinforced soil blocks, and the wall was raised using a staged construction technique. Clay slurry was prepared in a pit next to the wall (Figure 12) and poured in 300 mm layers (Figure 13).

Geosynthetic strips – the EKG electrodes – sandwiched each clay layer at 90° to the long axis of the wall, with the bottom layer acting as the anode and the top as the cathode. These were connected up to a generator and a direct current applied to induce dewatering by electro-osmosis.

Once a lift had been successfully treated (Figure

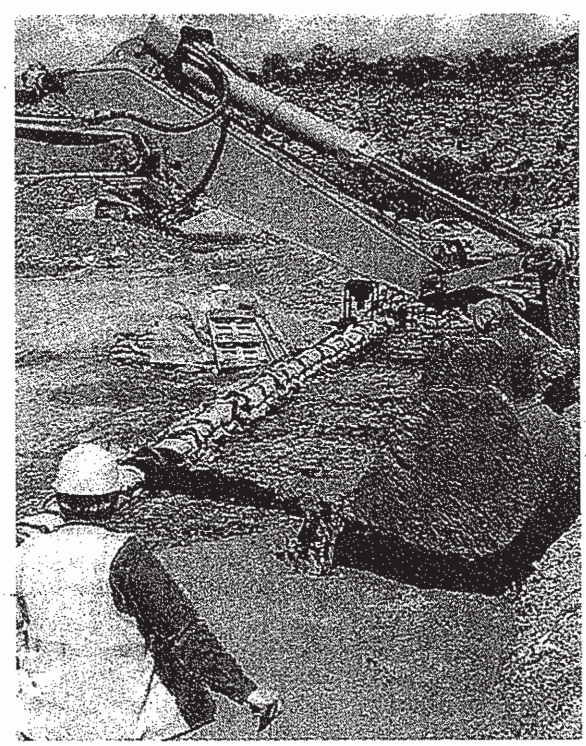


Figure 12

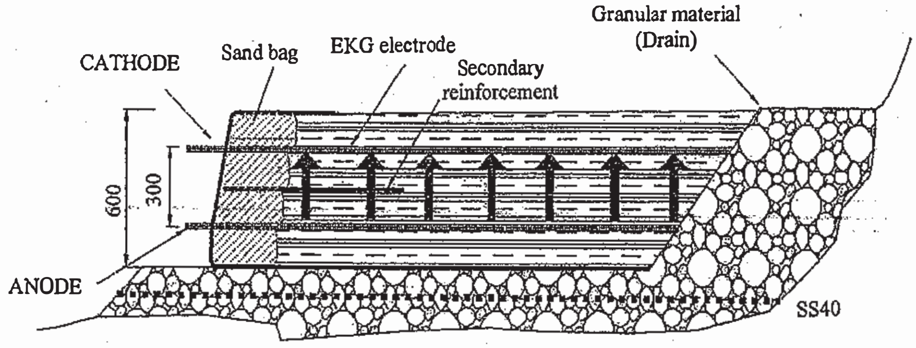
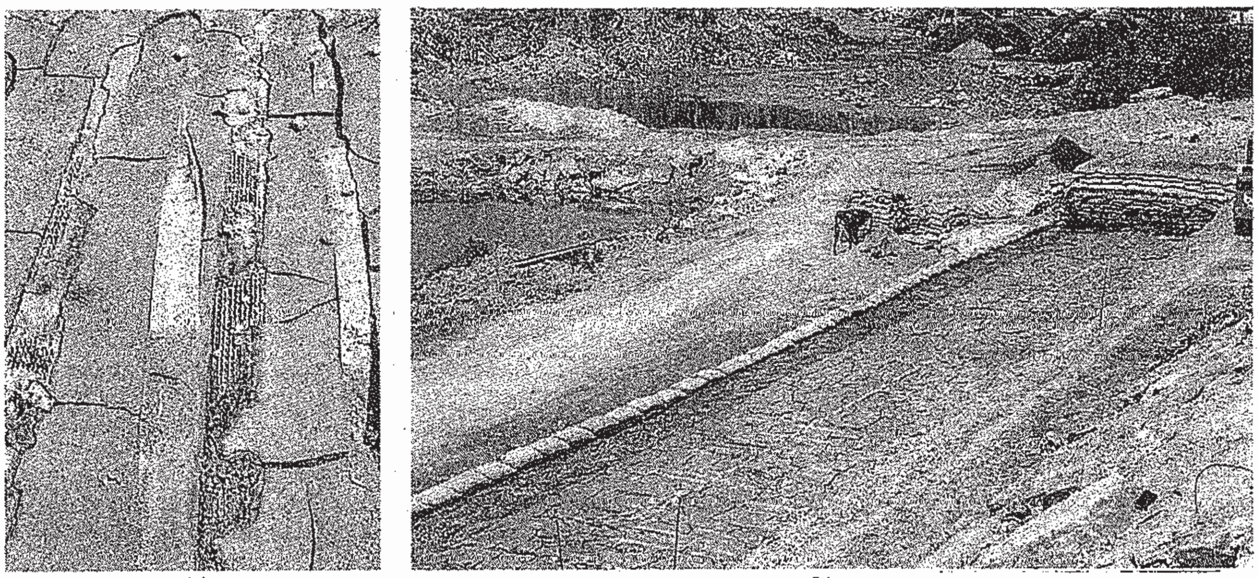


Figure 13 : Reinforced soil wall structure using EKG



(a) (b)

Figure 14 : Surface of completed lift after electro-osmotic dewatering

14), then the next lift was constructed, the original cathode now becoming the anode, and the original anode was disconnected from the electrical supply and thus adopted a reinforcing role.

The result of the trial showed that the shear strength of a wet slurry fill could be increased to permit safe construction of a vertical reinforced soil wall. Another finding was that the reinforcement/soil bond increased in proportion to the increase in shear strength.

2.4 *High Strength– Low Strain Reinforcement*

The development of subsurface voids can result from human activity such as mining or trenching, or can occur naturally by evaporite solution of underlying strata. If the void migrates to the surface major disruption can occur. The major complication of subsurface voids is their random occurrence both with respect to the time of the occurrence and their location. The logical approach to counteracting the development of surface voids is to avoid the problem by relocating to an area not affected. Unfortunately with many infrastructure systems such as highways or rail networks this is not possible, and structural precautions are required.

One of the first applications of geosynthetic reinforcement to support structures over sinkholes was in 1987-89 in Scotland, Cook (1989). This application used four layers of high tenacity polyester reinforcement of 1,000 kN/m strength to bridge sinkholes up to 12 m in diameter. Since this application the use of reinforcement over voids has increased based upon research findings and trials and importantly on the inclusion of the technique in BS 8006 (1995), Lawson *et al* (1996), Kempton *et al* (1996), Giroud *et al* (1990), Gourc *et al* (1999) and Alexiew (1997).

The usual criteria for design is to guard against the development of an ultimate limit state (collapse) and also provide support to maintain serviceability. Serviceability is usually defined in terms of a maximum differential surface deformation (d_s/D_s) which in BS 8006 is set at ≤ 1% for principal roads. Differential settlement has been shown to be influenced by the strength and stiffness of the reinforcement, as well as the size of the void and the overlying soil, Lawson *et al* (1996).

High strength polyester reinforcement (1,200 kN/M) with a modulus of 12,000 kN/m has been used to support highways over voids up to 14 m in diameter. In this case the function of the reinforcement was to provide a short period of safety before collapse. Some transportation systems require very low differential settlements (< 1 in 500) if they are to remain serviceable. In these cases high strength-high modulus material is required.

Ast *et al* (2001) have described the use of high strength aramid reinforcement of 12,000 kN/m capacity and a modulus of 56,900 kN/m placed under a 4 m high cement stabilised embankment to provide support for a high speed rail system where voids of 4 m diameter are expected. The reinforcement was prestressed when laid to ensure maximum stiffness of the system, Figure 15.

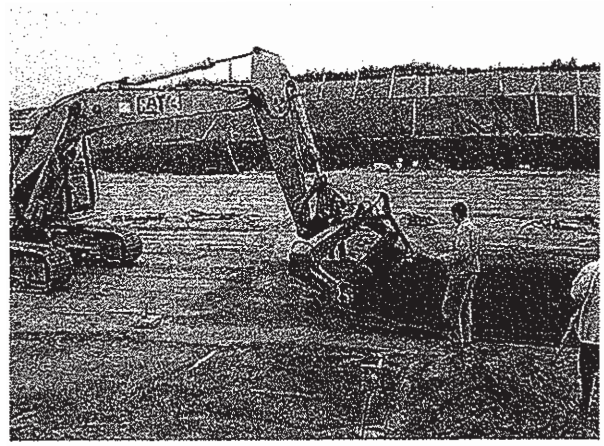


Figure 15

3 REINFORCEMENT MATERIAL PROPERTIES

3.1 *Residual Strength of Polymeric Reinforcement*

The tensile creep rupture strength of geosynthetic reinforcement is conventionally determined using creep-rupture curves, Figure 16. The creep-rupture

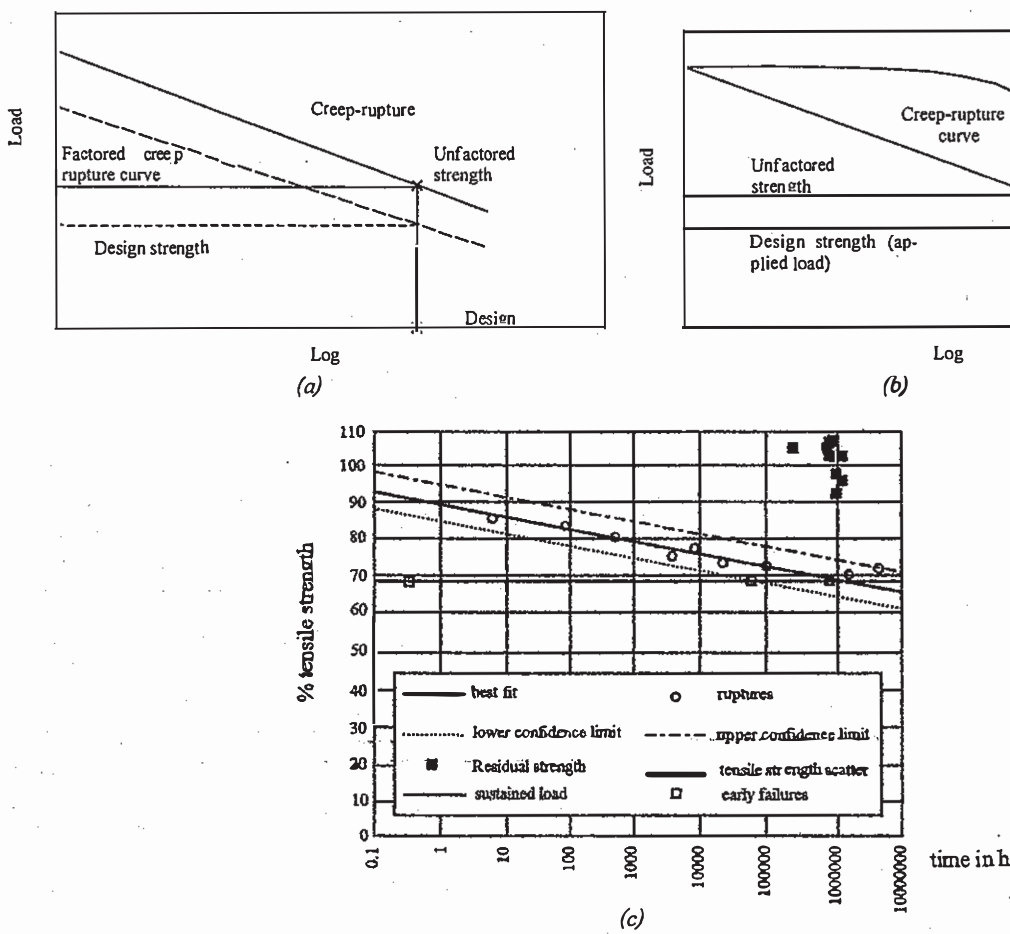


Figure 16 : Creep rupture strength of polymeric reinforcement

curve depicts sustained load against the lifetime under load and is usually interpreted as representing a reduction in strength against time, Figure 16(a). It has now been determined that the strength of the polymeric reinforcement is maintained until late in its service life, Figure 16(b), Greenwood (1998), Greenwood *et al* (2001); Orsat *et al* (1998).

The unfactored strength derived from the stress-rupture diagram is the sustained load which is predicted to lead to failure on the last day of the design life. The design load is equal to the unfactored strength divided by a safety factor, γ_m, to allow for the variability in material properties, installation damage and enviromnental factors. Under this lesser load the tensile strength of the polymeric reinforcement remains at a higher level up to and beyond the end of the design life, Figure 16(b) and (c). The ratio of the strength of the polymeric reinforcement to the design load is thus higher than the intended safety factor and many polymeric reinforced fill structures are (unintentionally) over-designed. The difference between Figures 16(a) and (b) in respect of design philosophy and the assumed and actual design strength of the structure is profound.

3.2 *Residual Strength and Seismic Loading*

The retention of the strength of polymeric reinforcement during its lifetime is of importance in seismic design. During seismic conditions a short term increase in the design strength of the reinforcement is accepted, Jones (1996). The increase in required strength to counteract seismic forces can be in the order of 50–100 percent of the design strength. Inspection of Figure 16(a) suggests that an increase in design strength of polymeric reinforcement from 40 percent of the characteristic strength to 60/80 percent could be accommodated early in the design life, but could not be sustained late in the life of the structure. This raises concern over the long term stability of reinforced fill structures formed using polymeric reinforcement. This concern is resolved if the residual strength of the reinforcement is considered, Figure 16(b) and (c).

A study involving the effect of simulated seismic events imposed during creep testing of polymeric reinforcement is nearing completion and will be reported at the 7th International Geosynthetics Conference in Nice in 2002.

3.3 *In-plane Bending Stiffness*

Modern urban developments in parts of the world require construction on soils which are very soft. The most extreme of these soils are *super soft clays* produced in tailings dams as the immediate result of land reclamation using hydraulic pumping. Super soft soil is defined as disturbed cohesive soil whose water content is higher than its liquid limit; such materials display extremely low yield stresses and represent difficult construction conditions, Fakher *et al* (1999).

An established technique used to enable construction on super soft clays is the introduction of a *primary construction stage* that is used as a working platform on which the main construction can be founded. A working platform can be produced by placing a layer of geosynthetic reinforcement over the super soft clay and covering this with a layer of cohesionless fill, Figure 17 (Yamanouchi and Gotoh, 1979). Although it has been demonstrated that this construction technique is successful, there is no general agreement with respect to the reinforcement mechanism and how the reinforcement improves the bearing capacity of working platforms.

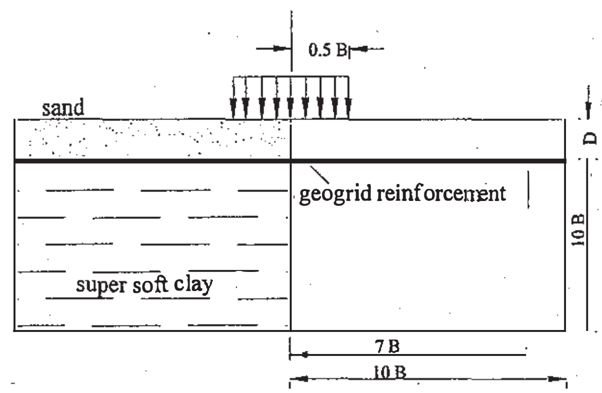


Figure 17 : The geometry of the problem

Recent studies using numerical modelling undertaken in parallel with a number of physical model studies into construction over super soft clay have demonstrated that the *in-plane bending stiffness* of the reinforcement has a critical role in the primary construction technique, Fakher and Jones (1996a), Fakher and Jones (1996b), Zakaria (1994).

In the modelling studies the super soft clay and cohesionless fill were idealised as construction materials and the reinforcements were modelled in two ways in order to investigate the effects of bending stiffness of the reinforcement on the bearing capacity of the super soft clay:

1. As cable elements, with no bending stiffness (i.e. a woven geotextile fabric). This method is frequently used to model geosynthetic reinforcements when bending is considered unimportant. The cable elements used in the analyses were one dimensional axial elements described in terms of cross sectional area, elastic modulus and yield strength of the cable.
2. As beam elements with elastic bending stiffness (i.e. some forms of geogrid reinforcement). The beam elements used in the analyses were two-dimensional elements with three degrees of free-

dom at each node providing displacement in two perpendicular directions and rotation. The beam elements were described in terms of cross-sectional area, elastic modulus, second moment of area (moment of inertia) and plastic moment. The moment capacity was assumed to be infinite.

The reinforcements were fixed to the line of symmetry, Figure 18. However, they were not fixed to any other vertical boundary so as not to cause an un-realistic pull-out resistance. The length of the geosynthetic reinforcement was $2 \times 7B$, where B is the width of the footing. In view of the low value of the shear strength of super soft clay, it was assumed that perfect adherence existed between the clay and the reinforcement.

The elastic modulus and cross sectional area of the reinforcement were assumed to be equal to 1×10^5 Pa and 0.0033 m^2 respectively. The second moment of area (I) of the beam elements was assumed to be equal to 7.64×10^{-7} m^4. However, this value was varied from 1×10^{-8} to 2.15×10^{-6} m^4 in order to study the effect of bending stiffness (EI). The bond stiffness and the bond strength of the interface of the element and soil and the yield strength of the cable elements were assumed to be 3×10^7 N/m/m; 4.5×10^4 N/m and 6.8×10^3 N respectively for all the analyses. Full details of the analysis are given in Fakher and Jones (2001).

The influence of bending stiffness (EI) of the reinforcement on displacement is shown in Figure 18, and the effect of in-plane bending stiffness of geogrid reinforcement on bearing load is shown in Figure 19. Figure 20 shows the effect of in-plane bending stiffness on the deformed shape of the reinforcement. The results show that the higher the in-plane bending stiffness the higher the potential bearing capacity.

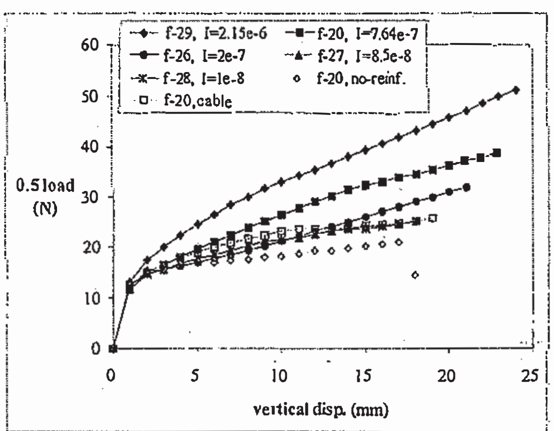


Figure 18 : The influence of the bending stiffness of the reinforcement on the displacement of the footing under different loads (the values of I are in m^4)

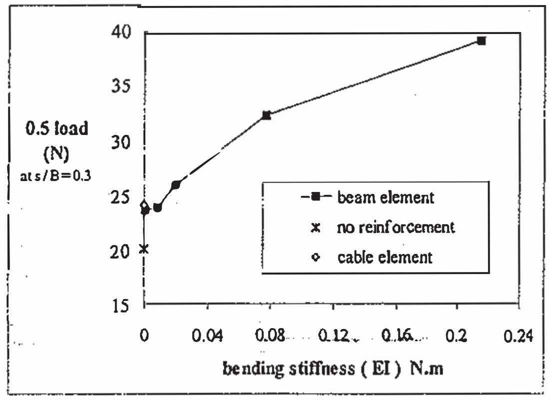


Figure 19 : Effect of bending stiffness of geogrid reinforcement on the bearing load (the bending stiffness is reported for one metre run and has the units of N.m)

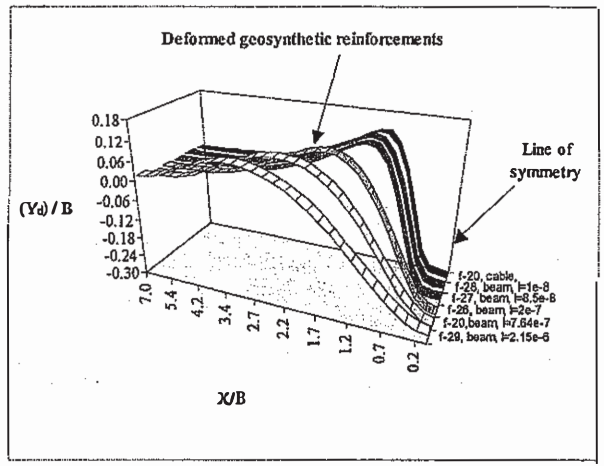


Figure 20 : Effect of bending stiffness on the deformed shape of geogrid reinforcement at (s/B) = 0.3

The thickness of the cohesionless fill layer (D) placed on top of the reinforcement and the width of any footing (B) also influences the bearing capacity ratio, defined as the ratio between bearing capacity of reinforced and unreinforced ground, Figure 21. When D/B is small, the increase in bearing capacity ratio due to bending stiffness is significant. As D/B increases, the influence of the bending stiffness of the reinforcement decreases. The deformed shape of the geosynthetic reinforcements for different ratio of D/B is shown in Figure 22. The maximum heave of reinforcement with no bending stiffness is located close to the point of the applied loading when values of D/B are small. This mirrors experience in field applications where geotextile reinforcements are found to be very difficult to lay on super soft soil as they provide no bearing capacity. Again, as the D/B ratio increases, the influence of the bending stiffness of the reinforcement decreases.

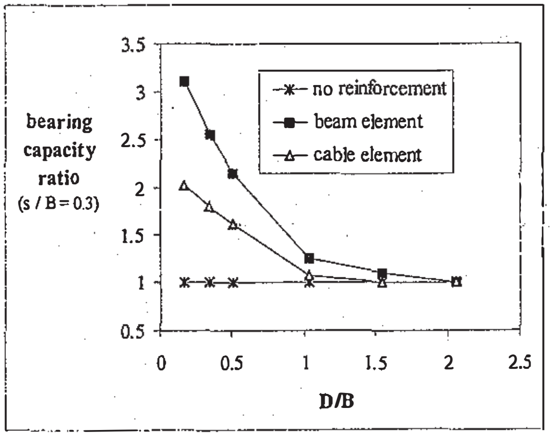


Figure 21 : Effect of D/B on the bearing capacity ratio

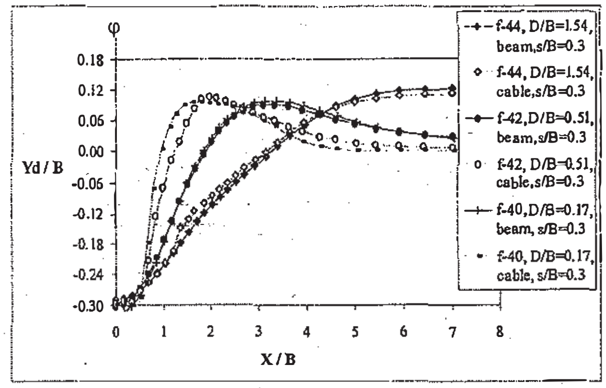


Figure 22 : Effect of D/B on the deformed shape of geosynthetic reinforcements

4 A NEW ANALYTICAL METHOD FOR REINFORCED SOIL

There are numerous methods available for the stability analysis of slopes. The majority of these may be categorised as limit equilibrium methods (Fang, 1991). The general approach is to assume a failure surface and to determine the factor of safety of a soil wedge against sliding using equilibrium equations. The basic assumption is that Coulomb's failure criterion is satisfied along the assumed failure surface, and the factor of safety is often defined as the ratio of available shear resistance to the required shear resistance.

Limit equilibrium methods can be divided into two main groups. The first group considers the equilibrium of the whole failing mass, assuming a failure surface. These methods are suitable for the analysis of homogeneous soils and specific failure surfaces. Culmann's method is an example of this group, Taylor (1984).

In the second group, a sliding wedge or "active" mass is divided into a number of vertical slices and the equilibrium of each individual slice considered. This procedure, known as the method of slices, has been adapted to any type of failure surface and soil. Figure 23 illustrates the method and the forces which act on a typical slice. A list of the governing equations and unknown parameters inherent to the vertical slice method is shown in Table 1. It can be seen that the number of unknown parameters is greater than the number of equations, and accordingly it is necessary to make further simplifying assumptions to reduce the number of unknowns.

A number of authors have presented vertical slice methods of analysis. The procedures differ principally in the equilibrium requirements which they satisfy and the manner in which they handle interslice forces which are normally dealt with in terms of vertical and horizontal components. The characteristics and the assumptions involved in some of these methods are illustrated in Table 2.

In the analysis of the stability of reinforced soil slopes the tension forces in the reinforcing elements need to be considered. Due to the method of construction and the usual orientation of the reinforce-

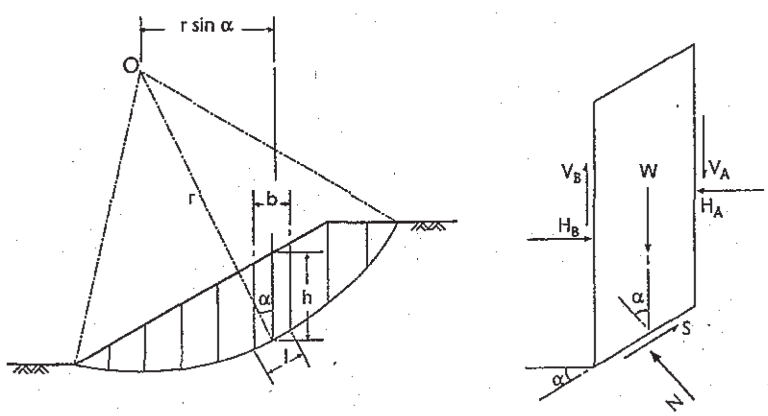


Figure 23 : Conventional method of slices

Table 1 : Equations and unknowns of vertical slice method of analysis

Equations	Number	Unknowns	Number
$\sum F_x = 0$ (each slice)	N	Horizontal interslice forces	N-1
$\sum F_y = 0$ (each slice)	N	Vertical interslice forces	N-1
$\sum M = 0$ (each slice)	N	Normal forces upon base of each slice	N
$\tau_r = \frac{\tau_f}{F.S.}$ (each slice)	N	Shear forces upon base of each slice	N
Equilibrium equations for whole mass	3	Location of horizontal interslice forces	N-1
		Location of shear and normal forces upon base of each slice	N
		Factor of Safety	1
Sum	4N+3	Sum	6N-2

N : number of slices

Table 2 : Characteristics and assumptions of some vertical slice methods of analysis

Method	Equilibrium conditions				Shape of failure surface	Assumptions
	$\sum M = 0$ (overall)	$\sum M = 0$ (individual)	$\sum F_x = 0$	$\sum F_y = 0$		
Fellenius (1936)	yes	no	no	no	circular	Resultant of side forces is parallel to base of each slice
Bishop (1955)	yes	no	yes	no	circular	Vertical side forces neglected – simplified method
Janbu (1954) Janbu *et al* (1956)	yes	yes	yes	yes	any	Location of side force resultant on sides of slices can be varied
Morgenstern & Price (1965)	yes	yes	yes	yes	any	A pattern of variation of side force inclination from slice to slice is assumed
Spencer (1967)	yes	yes	yes	yes	any	Side forces of all slices are parallel

ment, these forces are usually assumed to act horizontally. The limiting force developed in any reinforcing element, (t_j), is the lesser of the rupture strength of the reinforcement or the pull out resistance. Figure 24 shows that the orientation of the reinforcement has a direct influence on the interslice forces and that the reinforcement tensions are added unknowns in the vertical slices method analysis, Table 1. As a result the vertical slice method is not particularly suited to the analysis of reinforced soil slopes.

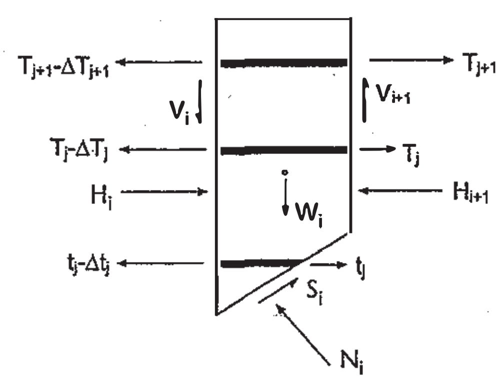


Figure 24 : Forces developed on a single vertical slice containing reinforcement

4.1 *Horizontal Slice Method of Analysis*

The limitations of the vertical slice method for the analysis of reinforced soil can be resolved by the use of horizontal slices, identified as the Horizontal

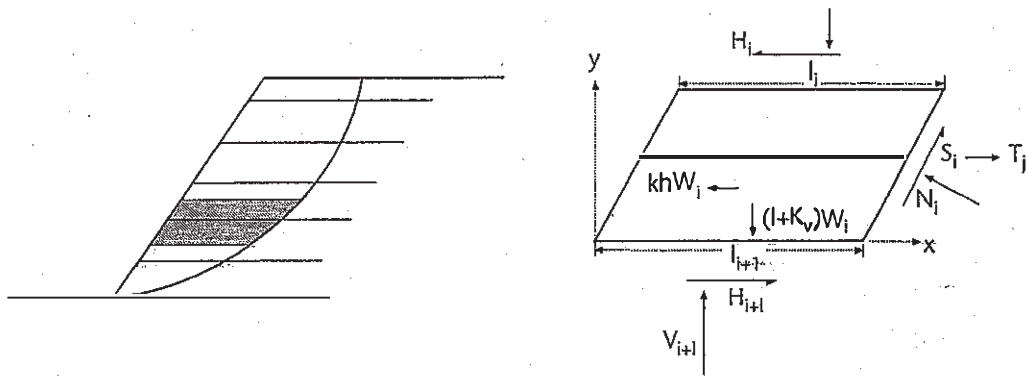


Figure 25 : Forces acting on a single horizontal slice containing reinforcement

Table 3 : Equations and unknowns of complete fonnulation of Horizontal Slice Method of analysis

Equations	Number	Unknowns	Number
$\sum F_x = 0$ (for each slice)	N	Horizontal interslice forces	N-1
$\sum F_y = 0$ (for each slice)	N	Nonnal forces upon base of each slice	N
$\sum M = 0$ (for each slice)	N	Shear forces upon base of each slice	N
$\tau_r = \dfrac{\tau_f}{F.S.}$ (for each slice)	N	Location of nonnal forces	N
		Factor of Safety	1
Sum	4N	Sum	4N

Slice Method (HSM). In this method, a failure surface is assumed and the failure wedge divided into a number of horizontal slices. The forces that act on each slice are shown in Figure 25. From Figure 25 it can be seen that no interslice forces are generated by the reinforcements. The following assumptions are made:

1. The vertical stress on an element in the soil mass is equal to the overburden pressure.*
2. The factor of safety (F.S.) is equal to the ratio of the available shear resistance to the required shear resistance along the failure surface.
3. The factor of safety for all slices is equal.
4. The failure surface can have any arbitrary shape but it does not pass below the toe of the slope or wall.

Thus, if the failure wedge is divided into N horizontal slices, there are 4N unknowns which can be determined by 4N equations, and a complete formulation is possible, as detailed in Table 3. The solution of the general formulation of the horizontal slice method with 4N unknowns is difficult and is the subject of current research. However, a simplified formulation of the HSM has been presented by Shahgholi *et al* (2001) which shows the advantage of the Horizontal Slice Method in comparison with vertical slice methods in the analysis of reinforced soil structures.

5 SOIL NAILING

The largest reinforced soil project currently being constructed in Europe is in Gibraltar. The work affects a large part of the east side of the famous Rock and entails the removal of the redundant century-old water collection sheeting formed from galvanised corrugated steel sheets, many still in good condition, and its replacement by 10,000 soil nails each 4 to 8 m long. In addition, 150,000 sq.m of erosion control geotextile is being placed on the natural sandy slope which will be vegetated, Figure 26.

6 DESIGN CODES

It has been found that the application of reinforced soil is heavily dependent upon the availability of national design codes which give the technology legitimacy. The introduction of a new design code is therefore a significant event.

A new Design Guidance Document for Reinforced Fill Structures and Slopes is being developed

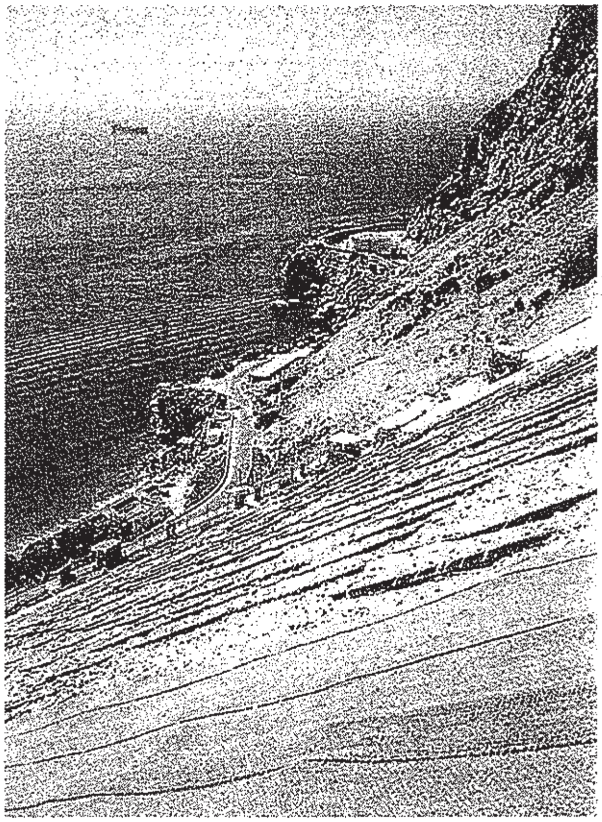


Figure 26

by the Hong Kong Geotechnical Engineering Office. The new Design Guide, which will supersede Geospec 2, is written as a limit state code. It covers the design of all forms of reinforced soil walls and slopes including segmental block walls. The Guidance Document will be fully compatible with GEO Geoguide 1 covering the design of conventional retaining structures and the Manual for Slopes.

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