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Geosynthetic materials with improved reinforcement capabilities

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ABSTRACT: The use of geosynthetic materials as reinforcement in soil is established practice. The materials used can take any form, the most common being strips, woven textiles and grids. The reinforcing mechanism is the provision of a tensile component to the soil strength developed through bond, anchorage or mechanical interlock. With the growing maturity of reinforced soil improved forms of reinforcement can be used to resolve specific technical problems.

Although all soils can be reinforced, good quality fill is usually specified and the use of fine grained soils in some codes of practice is not accepted. This can be a restriction of the use of reinforced soil, particularly when it is difficult to obtain good quality cohesionless fill. It has been shown that many soils not accepted by established codes of practice can be used to form serviceable structures. In some cases the explanation for the success of these structures is not apparent if analysed in accordance with recognised analytical procedures. However, incorporating drainage into the reinforced soil construction can improve the reinforcement mechanism and composite geosynthetic materials with the combined functions of drainage and reinforcement can be a logical choice in some reinforced soil applications. The use of electrically conductive geosynthetic materials introduces a new dimension in reinforced soil technology.

1 INTRODUCTION

The modern concept of earth reinforcement and soil structures was proposed by Casagrande who idealised the problem in the form of a weak soil reinforced by high-strength membranes laid horizontally in layers (Westergaard, 1938). The modern form of soil reinforcement was introduced by Vidal in the 1960s. Vidal's concept was for a composite material formed from flat (metal) reinforcing strips laid horizontally in a frictional soil, the interaction between the soil and the reinforcing members being solely by friction generated by gravity.

The introduction of the Vidal structures led to rapid development. Much fundamental work was sponsored by various national bodies, notably at the Laboratoire des Ponts et Chaussées (LCPC) in France (Schlosser, 1977), by the United States Department of Transportation (Walkinshaw, 1975) and by the United Kingdom Department of Transport (Murray, 1977). This work led to a better understanding of the fundamental concepts involved and the introduction of improved forms of reinforcement such as grids, first used by the California Department of Transportation (Forsyth, 1978).

Material development is interrelated with soil structure developments. Whereas historic reinforced soil structures were formed using organic materials such as timber, straw or reed for reinforcement, as early as the 19th century, Pasley (1822) recognised the potential of more advanced forms of reinforcement; his use of canvas as a reinforcing membrane is arguably the first application of geotextile reinforcement. Canvas could only have been expected to have a limited life before deterioration and Pasley's structures would not have been expected to be very durable.

The use of geotextiles for permanent reinforcement could not be contemplated until the development of synthetic polymer based materials. Synthetic fabrics were known prior to 1940 but it was not until the late 1960s and early 1970s that the advances in synthetic fabric and geotextile developments produced materials whose longevity could be assured.

1.1 *Polymeric Reinforcement Properties*

Polymer materials used as soil reinforcement have four main requirements. They must be strong, relatively stiff, durable and bond with the soil. Of critical importance is that the strength of the reinforcement is sufficient to support the force required to achieve stability of the structure. The magnitude of the required force will vary depending upon the application.

In a steep slope strengthened by a geotextile reinforcing layer each reinforcement might have to support a force of 10 - 40 kN/m; alternatively a single geotextile reinforcing layer at the base of an embankment on soft soil may be required to support a tensile force of 100 - 400 kN/m to ensure stability. The tensile strength required to provide support of structures in areas of subsidence can be significantly greater with long term strengths in excess of 1,000 kN/m.

The requirement of the geotextile to be stiff is so that the required force can be mobilised at a tensile strain which is compatible with the deformation of the soil. The concept of strain compatibility between the reinforced soil and the soil is implicit in any reinforced soil structure, Jewell (1992). The allowable tensile strain depends on the application and in the case of a reinforced slope on soft soil the allowable extension can vary from 5 - 10 per cent. In the case of a reinforced soil wall the design *allowable* tensile extension of the reinforcement is unlikely to exceed 2 - 4 per cent, with a limitation of <1 per cent strain occurring after construction. The *actual* extension is likely to be <2 per cent in total.

Durability of the polymeric reinforcement is influenced by time and has to be considered together with the environment conditions. With permanent structures durability is a dominant consideration of designers.

The mechanical requirement for bond between the reinforcement and the soil is important but often a function of the form of the polymer reinforcement. Geogrids and conventional geotextiles in the form of sheets provide good bond with the soil either due to the large surface offered by the geotextile or, in the case of geogrids, by soil/reinforcement interlock. In the case of strip or bar reinforcement, bond can become a critical consideration particularly in the top of a reinforced soil structure, Hassan (1992).

2 MATERIAL DEVELOPMENT

The successful development of many innovative technologies depends upon the active support of influential "champions". In the case of geosynthetic reinforced soil structures in the United Kingdom, the active support of the Department of Transport (DTp), the Transport Research Laboratory (TRL), the Highway Authorities and the Universities has been essential. In addition, the material manufacturers have been a major influence in initiating research and development.

Different forms of high quality geosynthetic reinforcement materials providing the essential properties detailed above have been developed by a number of manufacturers. Brief details of the main geosynthetic materials developed in the United Kingdom during the ten year period 1973-83 are shown in Table 1. Several of these have been the subject of continuous development and improvement, an example being Tensar reinforcement which over a period of 10 years shows an increase in tensile strength/weight of material of 150 per cent, Netlon (1996).

Developments of polymeric reinforcing materials have occurred in other parts of the world, notably in Europe, Japan and the United States. Nearly all are associated with different forms of grid materials or geotextiles in the form of needle-punched, heat-bonded or woven/knitted products.

2.1 *Benefits of Polymeric Reinforcement*

The initial development of polymeric reinforcement was to address the potential problem of corrosion of steel reinforcing elements and provide economies. Polymeric materials do not corrode, although durability has to be assured as described previously. A second major incentive to the use of polymeric reinforcement is the possibility of using indigenous and waste fills which can provide significant savings in construction costs. The use of pulverised fuel ash as a fill in conjunction with geosynthetic reinforcement is now established in the UK and can provide benefits not only in cost savings but also in the technical benefits arising from the cohesive nature of many of these fills which produce a lighter and inherently stronger structure, Jones *et al* (1990). The use of indigenous (cohesive) fill in steepened slopes

Table 1: Main geosynthetic reinforcing materials developed in the United Kingdom during 1973-83

Material (name)	Form	% extension at working load	Load capacity or characteristic tensile strength kN/m^2 or kN	Manufacturer (approx. date of introduction to the market)
Glass reinforced plastic (Fibretain)	strip §	0.2	16 - 80	Pilkington plc (1973)
Molecularly orientated grid (Tensar)	grid	< 3 *	varies (typically 97)	Netlon (1978)
Woven fabric (Terran RF12)	fabric	< 3 *	varies #	Imperial Chemical Industry (ICI) plc (1975)
Linear composite strip (paraweb)	tape	< 2	10 - 100	ICI/Linear Composites (1976)
Linear composite grid (paragrid)	grid	2 - 5	varies (300 - 1000+)	ICI/Exxon Chemical Geopolymers Ltd (early 1980s)

* Extension at working load is usually in range 1 - 2 %
\# Depends upon structure and weight per square metre
§ Normal width 40 - 160 mm

is common and this is the application where the majority of polymeric reinforcement is used.

The acute lack of suitable frictional fill in some countries such as parts of Japan has lead to the use of cohesive soils in major reinforced soil structures. In the case of Japan notable uses of cohesive fill include widening of railway embankments supporting the railway network. Extensive research by Japan Railways including the construction of full scale trial walls has established that geosynthetic reinforced soil structures formed using cohesive or cohesive-frictional fill are potentially more stable than structures formed from purely frictional fills, Tatsuoka *et al* (1992), Tatsuoka (1992). It is notable that this form of construction survived the Kobe earthquake, Tatsuoka *et al* (1995).

In France the development of geosynthetic reinforced structures has been continuing for over twenty years. A major argument advanced for the use of geosynthetic reinforcements is that on site material can be used for construction, whatever its nature. This concept has been demonstrated in the Lezat experimental wall where on site silt was used in the works. One of the largest structures built in France to date is a 21 m high structure, formed in three 7 m tiers with two 3 m berms, Gourc and Matichard (1994).

In Italy permanent geosynthetic reinforced soil walls have become an accepted and widely used technique. The low cost of the constituent materials forming the structures, the speed of construction and the possibility of developing alternative solutions are quoted as primary reasons for their success, Cazzuffi *et al* (1994).

3 TRIGGERS FOR ADVANCED GEOSYNTHETICS

Development of new and advanced forms of geosynthetic reinforcement is brought about by a number of factors including market forces, technical challenges, innovation and better understanding of the fundamental mechanisms of reinforced soil applications. Each of these can be shown to have been the trigger for new materials or material developments.

Market Forces As the use of reinforced soil has grown so competition in the material industry has increased. This in turn stimulates development of reinforcement products by manufacturers either using different reinforcing materials or improved manufacturing techniques. Examples of such developments can be found in the improvement in the range and quality of geogrid products offered by a number of manufacturers.

Technical Challenges As experience with reinforced soil has grown so the potential usage of the technique has expanded. Design concepts previously unknown to civil engineering have been possible; an example is the construction on super soft soil. The use of reinforced soil for different applications exposes areas where knowledge is lacking and identifies areas of uncertainty; examples have been the influence of repeated loading or seismic activity on reinforced soil structures and the soil-reinforcement bond with some forms of reinforcement. Consideration of the economics of reinforced soil illustrates that the use of waste fill or poor quality fill could be extremely beneficial.

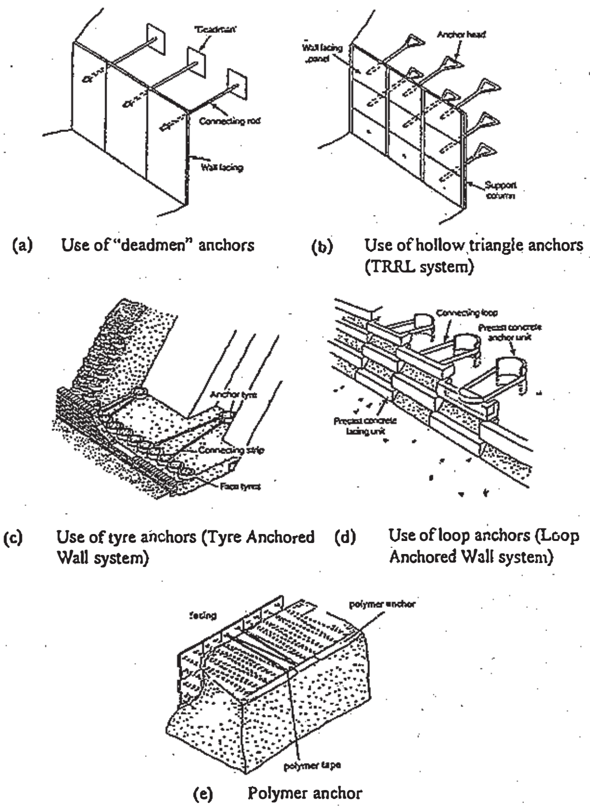
Each of these technical challenges has been the trigger for the development of new forms of reinforcement or studies leading to a better understanding of the underlying mechanism which in turn leads to better design and economic benefit. Some of these are illustrated below.

3.1 *Reinforcement Bond*

When sheets, bars and strips are used as reinforcement the coefficient of friction between the reinforcement and soil is a critical property: the higher the friction the more efficient the reinforcement. Thus an ideally rough bar, strip or sheet is significantly better than a reinforcement with a smooth surface, Schlosser and Elias (1978).

Reinforcement in the form of grids has been shown to be significantly more effective in bond. The fictional adherence between the longitudinal members of a grid and the soil is a function of the surface properties and the coefficient of fiction between the longitudinal members and the soil. However, the influence of the horizontal bearing capacity of the transverse elements is of an order greater, Forsyth (1978).

Reinforcement bond can be increased by the use of reinforcement in the form of anchors. This concept evolved simultaneously in Europe, Japan and the USA in the 1980s. The multi-anchor system was developed by Fukuoka (1980) for the Japanese Ministry of Construction. The anchor is in the form of a rectangular steel plate. The NEW retaining wall system, developed in Austria, is based upon an elemental concrete facing and polymeric ties in the form of a loop, Brandl & Dalmatiner (1986). In the USA and the UK anchors formed from waste automobile tyres illustrate both the economic and the environmental benefits of reinforced soil. Steel anchors formed from a single piece of rebar were developed by the Transport Research Laboratory in the UK, Murray & Irwin (1981). The first polymeric anchor was developed in 1992, Jones & Hassan (1992), Figure 1. The use of a polymeric anchor offers a reduction in the total quantity of reinforcing materials whilst still satisfying the design requirements of existing Codes of Practice. Using a polymeric anchor, material savings of up to 40 per cent are possible, Figure 2.



(a) Use of "deadmen" anchors

(b) Use of hollow triangle anchors (TRRL system)

(c) Use of tyre anchors (Tyre Anchored Wall system)

(d) Use of loop anchors (Loop Anchored Wall system)

(e) Polymer anchor

Figure 1: *Anchored soil retaining wall systems*

3.2 *Construction on Super Soft Soil*

In the past the construction method used when dealing with very soft soil was excavation and replacement. This is expensive and not always

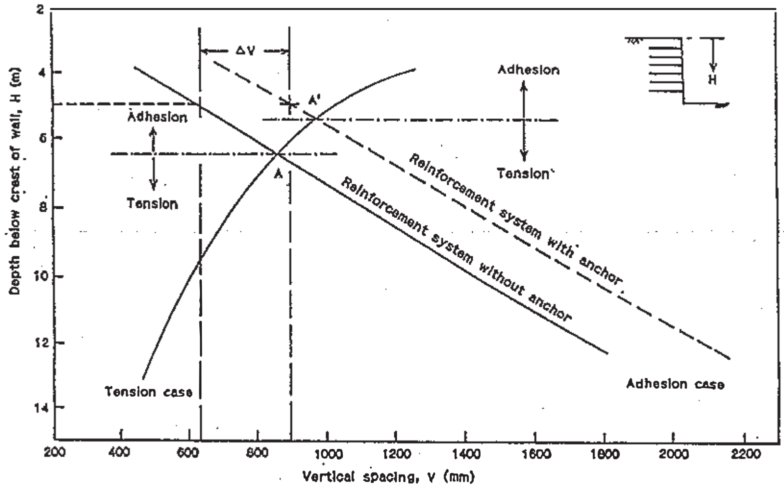


Figure 2: *Economic benefits of polymeric anchor (increase in vertical spacing Δv of reinforcement)*

practical; as a result treatment is becoming more popular. The most common method to stabilise soft soil is by preloading, frequently used in conjunction with vertical drains. When the soil is extremely soft, commonly referred to as a *super soft* soil, where the moisture content is higher than the liquid limit, preloading is not possible as the foundation has no effective bearing capacity. In this case a primary construction stage may be the creation of a working platform on which remedial treatment can be based, Yamanouchi (1970). The working platform (primary construction) frequently consists of geosynthetic reinforcement laid on the surface of the super soft soil supporting a thin uniform layer of cohesionless fill (sand), Figure 3. A number of methods have been used to form a fill layer over super soft soil using a geosynthetic membrane or reinforcement (Broms, 1987; Tan *et al*, 1994; Toh *et al*, 1994; Yano *et al*, 1985). Although this construction technique is demonstrably successful, there is no general agreement with respect to the reinforcement mechanism or how the reinforcement improves the bearing capacity.

The development of construction procedures for super soft soil was initially centred on South East Asia and Japan in particular. An early development in Japan was the use of low strength geogrids possessing an element of *in-plane* rigidity. Progressively these materials, although successful, have been replaced by more modern materials possessing high tensile strength. Recent research using model tests aimed at identifying the controlling mechanism involved in the primary stage construction concept have shown that the controlling parameters are, Zakaria (1994):

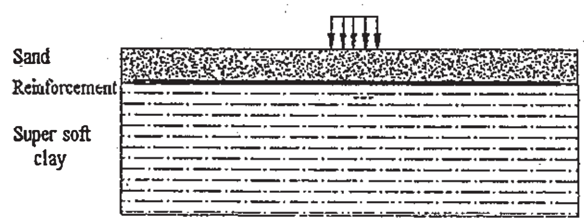


Figure 3: *Using geosynthetics and sand layer to improve bearing capacity*

$$f(q, B, D, a, S, R, c) = 0 \quad (1)$$

where f = a function which governs the system; q = bearing capacity (kN/m²); B = diameter of the applied load (footing) (m); D = thickness of fill layer (m); a = grid aperture (m); S = stiffness of the reinforcement in tension (kN/m); R = in-plane bending stiffness (rigidity) of the reinforcement (kN/m); c = shear strength of the clay (kN/m²)

By the use of *dimensional analysis* it is possible to convert data from model tests into design information for a large prototype. Dimensional analysis also reduces parametric variables leading to easier interpretation of the model tests.

Model tests of the mechanism of the primary construction stage technique indicate that geogrid reinforcement offers a higher bearing capacity than geotextile reinforcement, and that the main contributing property from the reinforcement is the in-plane rigidity, R, not the tensile stiffness, S. The size of the grid aperture was shown to be a minor contributor to the improvement of the bearing capacity. Using dimensional analysis a classification system for the bearing capacity based upon the dimensionless parameter ($R/S.B^2$) has been developed, Fakher *et al* (1996).

3.3 *Use of Cohesive Fills*

Many codes of practice do not permit the use of cohesive soil in the construction of reinforced soil structures for permanent work. The reasons given are the potential problems of low strength, high moisture content, creep and low bond strength between the reinforcement and the soil. Early research into reinforced soil associated with the development of internal friction of cohesive fill in undrained conditions showed that the parameter controlling the shear strength of reinforced soil was the relative volume of the fine grained portion of the fill, Schlosser & Long (1974). Murray & Boden (1979) constructed a full scale structure using cohesive soil reinforced with impermeable glass fibre reinforcing strips. They measured substantial rises in pore water pressures which took time to dissipate. Ingold (1979) in triaxial tests on clay reinforced with metallic foil found that the inclusion of impermeable reinforcement led to a reduction in the compressive strength with a strength ratio, F, defined as:

$$F = \frac{(\sigma_1 - \sigma_3 \text{ reinforced}}{(\sigma_1 - \sigma_3)\text{ unreinforced}} = 0.62 - 0.79 \qquad (2)$$

This effect was also noticed by Lee (1976) in tests using clay reinforced with mylor strips. The conclusion from these studies is that the inclusion of *impermeable* reinforcements in a clay fill results in induced excess pore water pressure creating possible reductions in the overall strength of the structure in the short term. If the excess pore water pressures could be reduced or removed more stable structures would result. This leads to the concept of the use of a *permeable* reinforcing element to act as a drainage layer.

The use of a permeable reinforcing material enables the build up of pore water pressure to be controlled. Primarily the reinforcing material must be permeable in the normal direction, allowing the passage of water from the soil above the reinforcement to that below. Secondly, the material needs in-plane permeability to speed the consolidation process. Ingold (1979) investigated the initial stability phase (undrained condition) of reinforced clay in a triaxial apparatus using a permeable geosynthetic reinforcement. The use of the permeable reinforcement resulted in an increase in shear strength with measurements of the pore pressure parameter A (as defined by Skempton (1954), B = 1) similar to an unreinforced control sample. However, there were still significant increases in pore water pressure. It was deduced that the permeable reinforcement had an A value lower than that of the soil; as a result one of the problems associated with the use of cohesive fill, the transfer of effective stress to the reinforcing element was effectively eliminated. Even if the reinforcement does not have high in-plane permeability the A value effect relives the pore water pressure developed by the increase in $\Delta\sigma_3$ developed by the presence of the reinforcement, allowing an immediate stress transfer. However, while the structure is approaching drained conditions at the soil-reinforcement interface, the remainder of the soil is still undergoing undrained loading which may lead to the soil failing in this region. This is dramatically illustrated in the full scale trial embankments reported by Tatsuoka & Yamauchi (1986). These tests were undertaken to study the effects of reinforcing clay slopes with non-woven permeable geotextiles. Figure 4 shows the configuration of the reinforcement in the two sides of one trial embankment and Figure 5 the resulting displacements. It is apparent that in the left-hand slope the spacing of the geosynthetic material was too large.

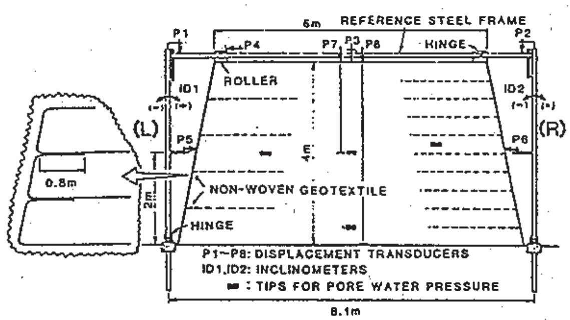


Figure 4: *Reinforced clay slope (After Tatsuoka & Yamanuchi, 1986)*

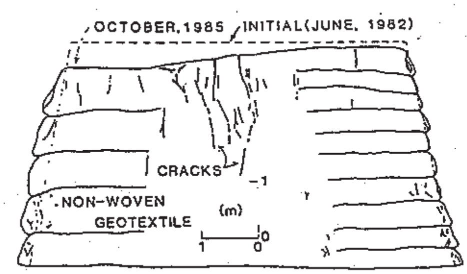


Figure 5: *Reinforced clay slope (After Tatsuoka & Yamanuchi, 1986)*

A second factor which needs to be addressed is the development of soil-reinforcement bond. Smith *et al* (1979) reported shear box texts investigating the development of bond using both woven and non-woven geotextiles. The results showed that the non-woven fabrics developed resistances to deformation at an earlier stage than the woven materials.

Non-woven geotextiles do not have great strength or in-plane stiffness and are not ideal in respect of the pure reinforcement function. An *ideal* reinforcing material for cohesive soil requires the drainage characteristics of a non-woven geotextile and the strength of a custom made reinforcement such as a geogrid. Alternatively it is possible to combine existing materials, ie using a non-woven drainage geotextile with a geogrid reinforcement. Such a combination is used in the Textomur system, Figure 6. The layout of the different geosynthetic materials in Figure 6 can be criticised as the reinforcement is placed in an area where the highest excess pore water pressure will occur and when dissipation will be slowest. The ideal arrangement would have the drainage function acting with the reinforcement function at the same place.

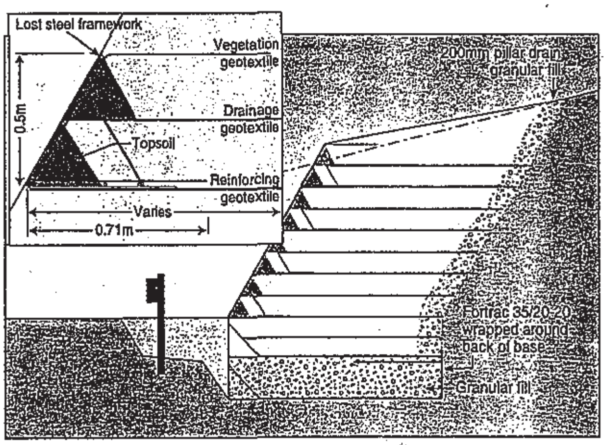


Figure 6: Textomur system

In order to study the effects of combining a drainage material with grid reinforcement in clay soil, a series of tests have been undertaken, Heshmati (1993). The objective of the tests was to identify the separate roles and the contribution of the different materials in improving the shear strength characteristics of clay. The drainage materials considered were non-woven geotextiles and the reinforcements used were grids. Grids are permeable in the plane perpendicular to the plane in which the reinforcement is laid and therefore could be expected to provide an increase in the shear strength.

A number of consolidated undrained (CU) and consolidated drained (CD) triaxial compression tests were undertaken using 100 mm diameter samples containing a single layer of different geotextiles placed at mid-height in the sample. The arrangement of the triaxial apparatus and the geotextile composite material used in the tests is shown in Figure 7. The form and nature of the materials tested are shown in Table 2.

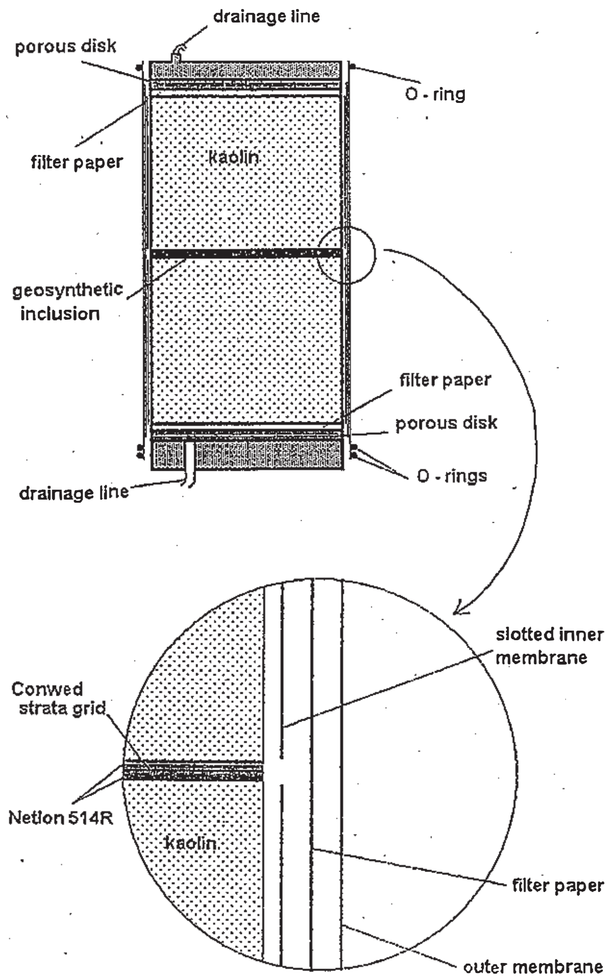
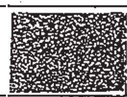


Figure 7: Arrangement of specimen within the cell

A major part of the experimental work was the deliberate separation of the reinforcement function provided by the geosynthetic materials identified as geogrids G2, G3, G4, and geotextile G5, and the drainage function provided by the materials G1 and G6, Table 2. The action of geotextile G1

Table 2: Nature and form of the geotextile materials tested

Samples	Figures	Description
K		Kaolin only, China Clay, 0.002 mm Particle Size
K + G1		Kaolin with, Netlon 514R, non woven needle punched polypropylene fibres. (providing both drainage and reinforcement)
K + G1CT		Kaolin with, Netlon 514R, non woven needle punched polypropylene fibres cut to equal wedges. (providing drainage)
K + G2		Kaolin with, Conwed Strata grid, geogrid, polyester 10x10 mm. (providing reinforcement)
K + G3		Kaolin with, Conwed 5033 geogrid, polyester, 25x25 mm. (providing reinforcement)
K + G4		Kaolin with, Tensar SS1 geogrid, polypropylene, 28x38 mm. (providing reinforcement)
K + G5		Kaolin with, non woven heat-bonded polypropylene geotextile.(providing reinforcement)
K + G6		Kaolin with, Filtram 1B1, Netlon, Composite geotextile, mesh core spacer with filter on both side. (providing both drainage and reinforcement)
K + G7		Kaolin with, Filtram 1B1, Netlon, mesh core, without filter on sides. (providing reinforcement)
K + G1G2		Kaolin with, Combined both G1 and G2 together. (providing both drainage and reinforcement)

was limited to provide a drainage function only by cutting the intact material thereby destroying any reinforcement properties (G1CT). Similarly the material G6 could be made to provide reinforcement properties only by removal of the outer filter membranes (G7). G1 and G6 were used to provide both reinforcement and drainage functions, and G1 and G2 were combined together to provide both reinforced and drainage functions (G1G2).

Unreinforced samples of kaolin were tested to provide base data and samples were tested at cell pressures of 150, 300 and 500 kPa. Samples were prepared in the laboratory in accordance with BS 1377 (1990) by compacting kaolin which had been mechanically mixed with 29 per cent water to obtain an optimum moisture content at 5 per cent voids.

Results of the consolidated undrained tests (CU) are shown in Table 3. The results of the consolidated drained tests (CD) are shown in Table 4.

It can be seen from the results that the inclusion of the drainage geotextiles in the fully consolidated undrained tests produced a *reduction* in the effective angle of internal friction (ϕ') [$K_{23.6}° \rightarrow G1_{20.1}°$], as did the cut geotextile [$K_{23.6}° \rightarrow G1CT_{20.6}°$], but a major *increase* in effective cohesion (c') [$K_{9.8} \rightarrow G1_{25.5}$], as did the cut geotextile [$K_{9.8} \rightarrow G1CT_{25}$].

In both the consolidated undrained and consolidated drained tests the geogrids (G2 and G7) acted as reinforcement which produced a *reduction* in the effective angle of internal friction (ϕ'); for the undrained tests [$K_{23.6}° \rightarrow G2_{20.4}° \rightarrow G7_{21.1}°$]; for the drained tests [$K_{23}° \rightarrow G2_{20.7}° \rightarrow G7_{20.5}°$]. However, the presence of the reinforcement resulted in a major *increase* in effective cohesion (c') for the undrained tests [$K_{9.8} \rightarrow G2_{25} \rightarrow G7_{36}$], for the drained tests [$K_{18.5} \rightarrow G2_{43} \rightarrow G7_{44}$].

The results showed that the geogrids (G3 and G4) acting as reinforcement were not as effective as

Table 3: Summary of Triaxial Test Result
Consolidated - Undrained Triaxial Compression Test (CU Test)

Sample	Effective Cell Pressure kPa 150-300-500	Rate of Strain (mm/min)	Deviator Stress at Failure kPa $(\sigma'_1-\sigma'_3)_f$	$(\sigma'_1)_f$ kPa	Strain at Failure %	Shear Strength Parameter Cohesion - Angle of C'_{cu}	ϕ'_{cu}
K	150	0.035	140	235.5	10.5		
	300	0.028	220	393.4	9.5	9.8	23.6°
	500	0.030	308	566.4	9.0		
K + G1	~~150~~	~~0.074~~	~~215~~	~~357.7~~	~~10.7~~		
	300	0.028	283	611.5	8.0	25.5	20.1°
	500	0.040	325	795.3	10.7		
K + G1CT	150	0.048	170	317.9	8.6		
	300	0.074	340	575.6	10.7	18.3	20.6°
	500	0.077	418	637.4	11.0		
K + G2	150	0.054	130	218.9	7.2		
	300	0.090	170	345.7	9.4	25	20.4°
	500	0.11	350	709.5	7.8		
K + G3	150	0.041	162	281.4	10.5		
	300	0.11	262	453.7	8.2	16	21.7°
	500	0.033	358	634.5	8.8		
K + G4	150	0.041	145	248.2	8.2		
	300	0.053	250	402.7	10.5	16	20.5°
	500	0.059	391	715.8	11.5		
K + G5	150	0.055	175	276.5	9.2		
	300	0.079	240	418.7	8.7	18	19.3°
	500	0.078	390	632.3	11.5		
K + G6	150	0.067	210	306.4	6.5		
	300	0.028	331	221.7	9.5	39.5	18.9°
	500	0.030	392	674.5	9.0		
K + G7	150	0.033	229	364.9	8.2		
	300	0.057	282	465.5	9.8	36.1	21.2°
	500	0.047	388	703.1	10.7		
K + G1G2	150	0.035	160	262.5	8.5		
	300	0.035	270	453.7	8.2	16	20.5°
	500	0.037	407	718.1	9.0		

the materials (G2 and G7) probably due to the large aperture of the grid. G1G2 was thought to have the ideal properties required to provide the dual action of drainage and reinforcement; however the results produced a *reduction* in strength compared to the same drainage and reinforcement materials acting separately, the explanation being the development of a plane of weakness at the interface of the two materials. The implication of this is that it is an essential requirement when combining the functions of drainage and reinforcement for the materials to be made *integral*. This has been confirmed by Cunningham (1995).

4 NEW GEOSYNTHETIC MATERIALS

The increasing demands for construction materials, land and the redevelopment of derelict land are leading to the development of new technologies, and the integration of existing technologies. The development of *electrically conductive geosynthetics* (EKGs) is an example of this. The concept is to develop a range of geosynthetics which in addition to providing filtration, drainage and reinforcement can be enhanced by electrokinetic techniques for the transport of water and chemical species within fine grained low permeability soils, which are otherwise difficult or impossible to deal with. In addition, transivity, sorption, wicking and hydrophobic tendencies may also be incorporated in the geosynthetic to enhance other properties.

The ability of electrokinetic phenomena to move water, charged particles and free ions through fine grained low permeability soil is established. Electrokinetic phenomena will occur in any soil; however, in medium to coarse grained soils

Table 4: Summary of Triaxial Test Result
Consolidated - Drained Triaxial Compression Test (CD Test)

Sample	Effective Cell Pressure kPa 150-300-500	Deviator Stress at Failure kPa $(\sigma'_1-\sigma'_3)_f$	Strain at Failure ε_f%	Shear Strength Parameter Cohesion - Angle of Internal Friction C'_d	ϕ'_d
K	150	118	11.7		
	300	220	9.8	18.5	23.0°
	500	330	9.2		
K + G1	150	262	10.2		
	300	420	11.0	31.5	22.5°
	500	798	11.9		
K + G1CT	150	260	9.5		
	300	396	11.0	25.0	22.5°
	500	623	9.0		
K + G2	150	377	8.0		
	300	462	10.0	43.0	20.7°
	500	650	10.0		
K + G3	150	239	9.5		
	300	362	12.4	23.0	22.2°
	500	710	10.0		
K + G4	150	220	7.7		
	300	424	8.7	27.0	21.1°
	500	602	8.7		
K + G5	150	230	9.7		
	300	419	11.5	29.5	18.7°
	500	580	12.0		
K + G6	150	245	9.6		
	300	282	8.0	41.5	18.6°
	500	372	7.6		
K + G7	150	280	9.5		
	300	400	11.0	44.0	20.5°
	500	610	11.0		
K + G1G2	150	238	6.5		
	300	418	8.0	30.0	18.9°
	500	579	8.0		

electrokinetic phenomena provide a less effective transport mechanism than hydraulic flows, due to high permeability of the soil. In the case of fine grained soils electrokinetic phenomena provide much more important transport mechanisms than hydraulic flows, due to the low hydraulic permeabilities of these soils.

The first application of electrokinetic phenomena for civil engineering processes was undertaken by Casagrande in 1939 for the dewatering and stabilisation of railway cuttings at Salzgitter, Germany, Casagrande (1952). Since then there have been many other applications of electroosmosis for dewatering and stabilisation of soils, including: (i) The use of electroosmosis for dewatering and strengthening Norwegian quick clays, Bjerrum *et. al* (1967); (ii) Electroosmotic stabilisation of West Branch Dam, Fetzer (1967); (iii) Electroosmotic treatment to improve pile friction, Milligan (1994).

4.1 *Electrokinetic Transport Phenomena*

There are five principle electrokinetic phenomena: Streaming Potential, Migration Potential, Electroosmosis, Ion Migration and Electrophoresis. The first two of these phenomena are concerned with the generation of electrical potential due to the movement of charges and charged particles respectively. The remaining three are concerned with the transport mechanisms developed upon application of an electrical field across a soil mass.

Electroosmosis When an electrical field is applied across a fine grained soil mass cations are attracted to the cathode and anions to the anode, Figure 8. As

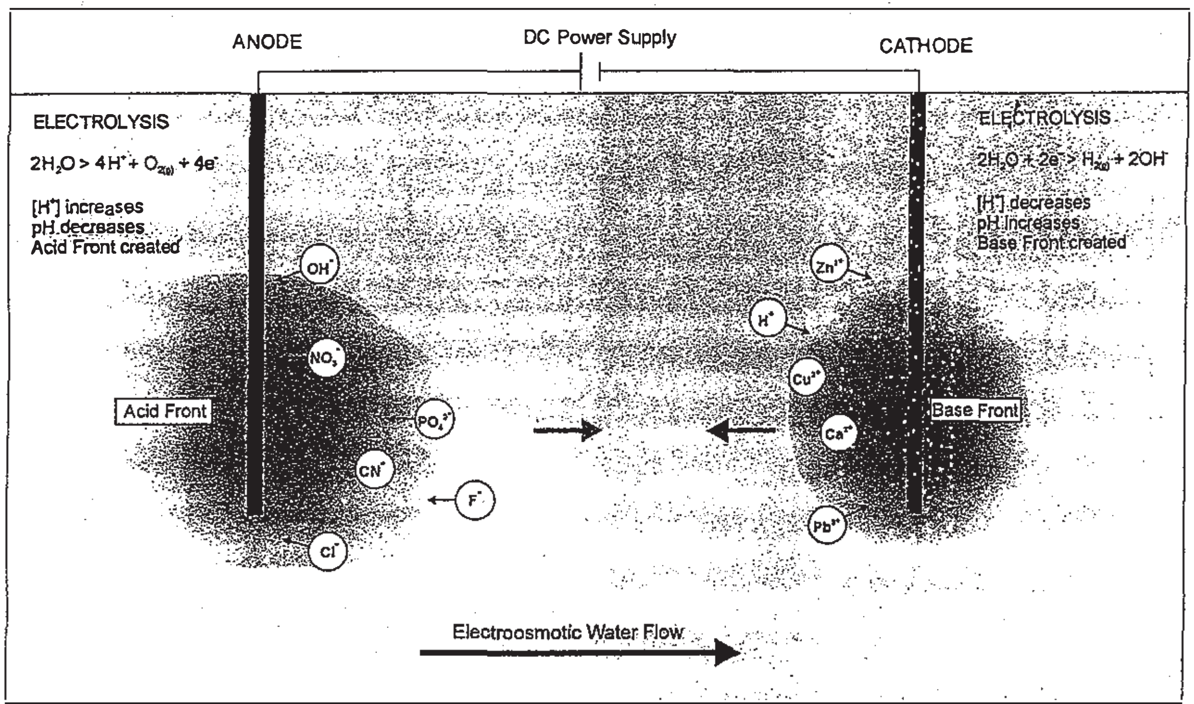


Figure 8: *Electrode reactions and electrokinetic transport of pore water (electroosmosis) and ions (ion migration)*

the ions migrate they carry their hydration water with them, and exert a frictional force on the water around them. Hence, there is a flow of water to both the anode and the cathode. In order to maintain a charge neutrality, however, there are more cations than anions in the pore fluid of a soil containing negatively charged clay particles. Therefore, there is a net flow of water to the cathode, Gray & Mitchell (1967). This electroosmotic flow depends upon the applied voltage gradient and the electroosmotic permeability of the soil.

The electroosmotic flow of water through a soil can be expressed in the form of Darcy's equation for water flow:

$$q_A = k_e i_e A$$

where q_A is the flow rate, i_e is the potential gradient $\Delta V/\Delta L$, A is the cross-sectional area and k_e is the coefficient of electroosmotic hydraulic conductivity.

The electroosmotic permeability (k_e) for most soils lies in the range $1x10^{-9}$ to $1x10^{-8}$ m²/Vs, Lageman *et al* (1989), and is independent of pore size, Casagrande (1949). As an example of the effectiveness of electroosmosis for the movement of water through fine grained soils consider a sample of white silt (Grade E Kaolinite) with:

- Cross-sectional area = 0.04 m²
- Hydraulic permeability = $5x10^{-9}$ m/s
- Applied voltage gradient = 50 V/m
- Resulting electroosmotic flow rate = 48 ml/hr = 0.0133 ml/s
- Electroosmotic permeability = 0.0133 / (50 * 0.04) * ($1x10^{6}$) = $6.65x10^{-9}$ m²/Vs
- The hydraulic gradient to yield the same flow = 0.0133 / ($5x10^{-9}$ * 0.04) * ($1x10^{6}$) = 66.5

Electromigration or Ion Migration The application of an electrical field across a soil mass causes migration of the free ions and ion complexes, which are present within the pore fluid, to the appropriate electrode, Lageman *et al* (1989), Hellawell (1994). The migration velocity for an electrical field of 1V/m is termed the effective ion mobility u_i^* (m²/Vs).

Lageman *et al* (1989) report the average mobility of ions in soils as $5x10^{-8}$ m²/Vs, which is an order of magnitude greater than the electroosmotic permeability. Hence, anions can usually overcome the electroosmotic flow and migrate towards the anode. In the remediation of ionic species ion migration is the dominant transport mechanism, Hellawell (1994).

Electrophoresis When a DC electrical field is applied across a particulate suspension (colloids, clay particles, organics), charged particles in suspension are electrostatically attracted to one of the electrodes and are repelled from the other. Positively charged particles are attracted to the cathode and negatively charged particles are attracted to the anode. Most colloids are negatively charged and are therefore attracted to the cathode.

Electrophoretic mobilities reported by Lageman *et al* (1989) and by van Olphen (1977) are in the ranges $1x10^{-10}$ to $3x10^{-10}$ m^2/Vs and $1x10^{-8}$ to $3x10^{-8}$ m^2/Vs respectively. Hellawell (1994) attributes the discrepancy to differences in the moisture content of the medium. Electrophoresis has found applications in the densification of sludges and mine tailings, Lo *et al* (1991). Esrig (1968) reports that the process is inconsequential for most naturally occurring soils.

4.2 *Electrokinetic Geosynthetics*

The new EKGs can take the form of single materials, which are electrically conductive, or composite materials, in which at least one element is electrically conductive. They can be of the same basic form as present day filter, drainage, separator and reinforcement materials, but offer sufficient electrical conduction to allow the application of electrokinetic techniques for ground improvement.

There are a number of materials which can be used to produce electrically conductive geosynthetics. The principle conductive materials which are being considered and evaluated at Newcastle are:

Carbon Fibre Materials Carbon fibres are representative of graphite materials; they are electrically conductive (> 10^2 s/cm) and chemically stable. High modulus carbon fibres are expensive and their use is limited to specialist applications where high stresses, high temperatures and radiation exposure are expected. The lower modulus carbon fibres are cheaper, their production simpler and the raw materials more abundant, Ermolenko *et al* (1990).

Activated carbon fibres (ACFs) have an open microporous structure which provides sorption sites. They provide similar or higher sorption than activated carbon particles and their shape allows more rapid and greater sorption than is found with carbon particles, Ermolenko *et al* (1990). The low modulus materials have many of the features required for the production of electrically conductive geosynthetics.

Conductively Filled Polymers Organic polymers with all carbon backbones are insulators (eg polyethylene PE, polyvinylchloride PVC, ethylvinylacetate EVA) and can be used for electrical insulation, Campbell (1994). However, if carbon black, carbon fibres or finely divided metals are used as fillers in such organic polymers, then conductive (10^{-5} to 1 s/cm) composites may be created, Cowie (1991). In these composites it is the filler which conducts and not the polymer.

If carbon black is used as the conductive filler then a loading of between 20 and 30 wt % will be required to produce a suitable conductivity. The structure of the carbon black fillers plays an important role in determining their properties. There are two main types of carbon structure of interest, the Low structure which is a jumbled array of carbon spheres, and the High structure which is a chain of spheres giving a higher conductivity.

An activated carbon black or carbon fibre filler may also be introduced to increase the sorption characteristics of polymeric materials. Sorption is another potential function of EKGs.

Metallic Fibres Metal fibres, metallised fibres or metal coated fibres can be incorporated into the manufacturing processes for geotextiles (needle punching, weaving). Metal coated fibres have only low conduction (10^{-6} to 10^{-1} s/cm) and are not suitable for EKGs. However, the metal and metallised fibres are considered suitable. Metallised fibres have conductivities of 10^0 to 10^4 s/cm.

4.3 *Ground Improvement and Reinforcement using Electrokinetic Geosynthetics*

Electroosmotic treatment of soils generates negative pore pressures and thereby increases the effective stress, leading to consolidation of the soil. The end result is similar to that gained by the application of a surcharge; however, unlike vertically surcharging a soil mass, there are no stability problems. Electroosmosis may also be used to accelerate the dissipation of positive pore pressures resulting from vertical surcharging. Indeed, once the positive pore pressures have been dissipated electroosmosis can be continued for further consolidation, due to the generation of negative pore pressures. These processes increase the *strength* of the soil and the *bond* characteristics of the soil and the reinforcement.

In addition to the electroosmotic improvement of cohesive soils there is often an increase in the strength of the soils due to chemical cementing of the soil fabric and increases in the Atterberg limits of the soil, Mitchell (1991).

4.4 *Factors Affecting the Incorporation of EKGs in Reinforced Soil*

Incorporating the electroosmotic phenomenon with the traditional function of geosynthetics is a new concept. Some of the factors which need to be considered for its successful incorporation include:

Drainage During electroosmotic treatment, water flows from the soil into the drainage media and gases in the form of oxygen at the anode and hydrogen at the cathode are produced due to the electrolysis of water. Both the water and the gases must be removed from the soil by the drainage media. The drainage media must fulfil two basic functions:

(a) Filtration - to allow the free passage of water from the soil to the drainage system.
(b) Transmissivity - to transport water and gas out of the system without hindrance.

Electrode Configuration The flow of electric current through a saturated clay mass occurs primarily through the pore fluid. An efficient mode of current transmission from the electrodes to the soil is important as it affects the efficiency of the electroosmotic treatment. The higher the interface resistance, the lower will be the effective voltage gradient for electroosmotic consolidation; consequently the magnitude of negative pore pressure generated will be reduced. Factors affecting the soil/electrode interface resistance include:

(a) form of the electrodes;
(b) gas generation during the process;
(c) current densities.

Electrode configuration plays an important role in determining the magnitude of interface resistance. An important consideration is the presence of ground water in the pore of the geosynthetic which makes its whole surface an electrode. A major advantage of the use of a geosynthetic electrode is that the surface area can be much larger than traditional electrodes.

Figure 9a shows the distribution of the negative pore pressure during consolidation of the fill layer incorporating two EKG materials. In this case, drainage is at the cathode, hence the boundary conditions would be open cathode and closed anode. The maximum negative pore pressure due to electroosmotic consolidation will be generated at the anode, Esrig (1968). At the cathode, the fill material will consolidate under its self weight. The flow of water due to electroosmotic consolidation v_e and due to self weight v_h is downward towards the cathode.

Once consolidation is completed, reversing the polarity of the electrodes would result in consolidation of the soil in the vicinity of the cathode, Figure 9b.

The effect of placing successive layers of fill and conductive reinforcement can be seen in Figures 9c to e.

4.5 *Type of Soil Suitable for Treatment*

The magnitude of negative pore pressure generated at the anode at equilibrium is given by

$$u = \frac{k_e}{k_h}\gamma_w V \tag{3}$$

where k_e is the electroosmotic permeability, k_h is the hydraulic permeability and V is the applied voltage.

Equation 3 indicates that for a given applied voltage, the lower the hydraulic permeability the higher would be the negative pore pressure generated. If the total stress does not change during electroosmotic consolidation then the magnitude of the effective stress is proportional to the negative pore pressure generated at the anode. The soils deemed suitable for electroosmotic consolidation are silts, clayey silt and silty clay, Mitchell (1993).

4.6 *Laboratory Tests - Consolidation*

A number of laboratory tests have been conducted to evaluate the use of conductive geotextiles as electrodes in electroosmotic consolidation. The tests using an electroosmotic cell simulated the reinforcement, electrodes and the drainage configuration expected in reinforced soil applications, Figure 10.

A number of geosynthetic electrodes were considered and compared with a perforated copper disk. The initial type 1 geosynthetic electrode was formed from a non-woven, needle punched geotextile. A copper wire of 300 mm length and 1 mm diameter was inserted at the centre of the geotextile to transmit current to the water in the pores of the material. Copper wire was chosen to inhibit the generation of oxygen at the anode. Any oxygen generated at the anode combined with the copper to form copper oxide which is a good

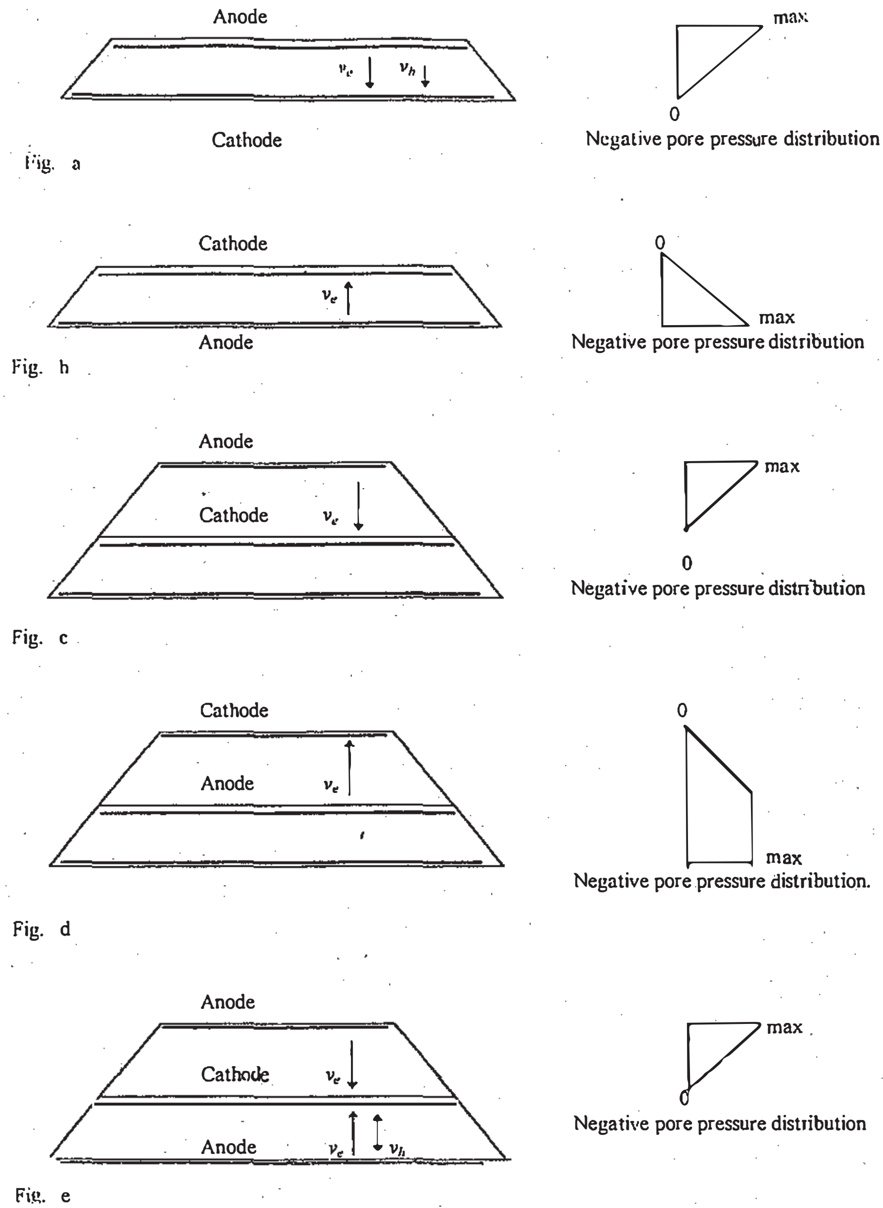


Figure 9

conductor of electricity and the generation of negative pore pressures at the anode were not affected, Lo *et al* (1991).

Commercially available Kaolin grade E was used in the tests. It was classified as silty clay with a plastic limit of 35% and a liquid limit of 55%. The electroosmotic permeability was $k_e = 3.4\times10^{-5}$ cm/sec per volt/cm and hydraulic permeability $k_h = 6\times10^{-5}$ cm/sec. By substituting the value of k_e and k_h in equation 3, the negative pore pressure generated due to electroosmotic consolidation, U=0.98V.

The soil sample for the tests was prepared from slurry at a moisture content of 1.5 times the liquid limit to ensure it would be fully saturated. Initially the soil was consolidated with an effective stress of 25 kPa.

The electroosmotic tests were conducted with vertical pressures of 50 and 100 kPa to simulate the self weight of the soil. During the tests, pore pressures, current and voltage variations across the soil sample were recorded. After treatment, tests were carried out to determine the moisture content and strength of the soil.

Type of Electrodes Figure 11 shows the applied voltage versus maximum pore pressure at the anode

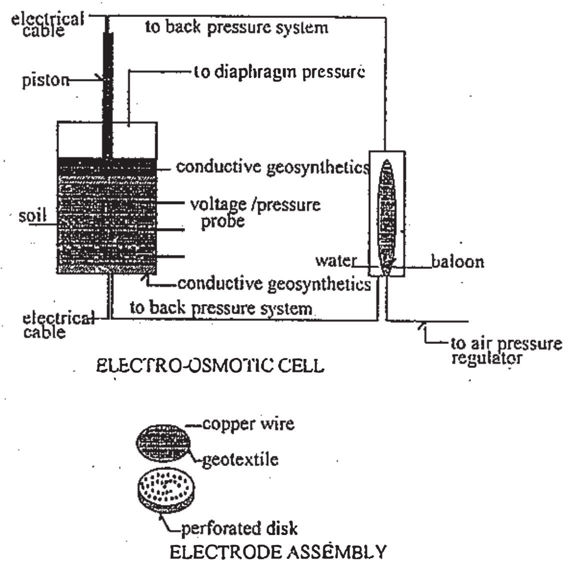


Figure 10: Schematic diagram of the electroosmotic cell and the electrode assembly

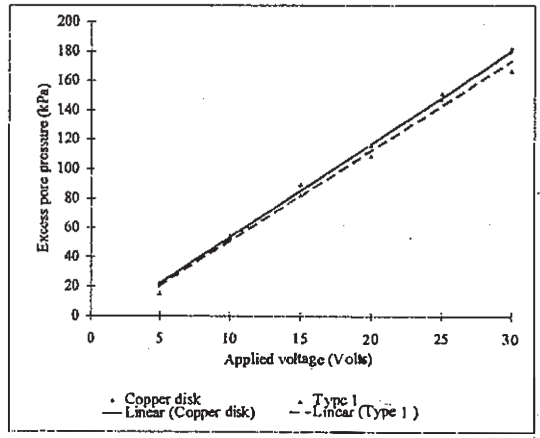


Figure 11

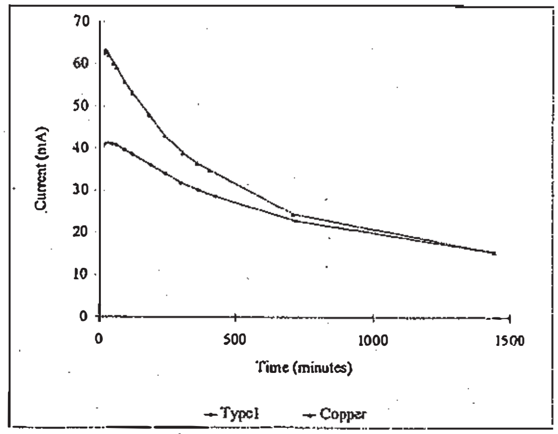


Figure 12

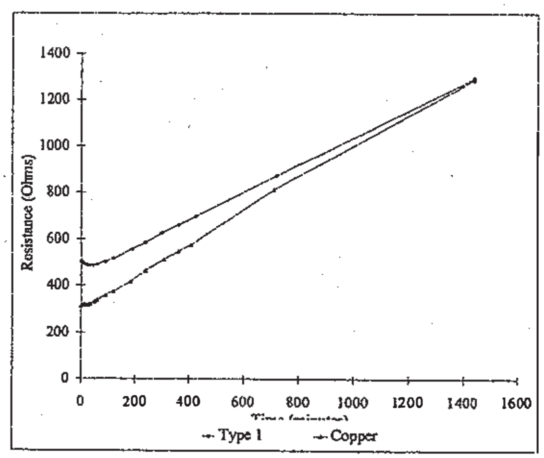


Figure 13

during electroosmotic consolidation with copper and the type 1 geosynthetic electrode. In this series of tests no vertical pressure was applied. It may be seen that the negative pore pressure varies linearly to the applied voltage for both types of electrodes. However, the two lines do not intersect at the origin but at some points along the x axis. The points of intersection at the x axes represent the voltage loss at the soil/electrode interface. The magnitude of the voltage loss is constant regardless of the applied voltage. The values of voltage loss were 1.51 volts and 1.78 volts for the copper and type 1 geosynthetic electrodes respectively.

Samples of variations of current and resistance during electroosmotic tests are shown in Figures 12 and 13 respectively. The applied voltage for the tests was 20 volts. The initial current and resistance for tests using copper disk electrodes were 63 mA and 310 Ohms while the value for type 1 geosynthetic electrodes were 40 mA and 500 Ohms. The soil resistance is the same for both tests.

Negative Pore Pressure Figure 14 shows the change in pore pressure at the anode during electroosmotic consolidation using type 1 geosynthetic electrodes with different applied voltages. Initially the back pressure was lowered by 50 kPa to simulate an applied vertical pressure. At the start of the test, the pore pressure at the anode decreased until it equalised the back pressure,

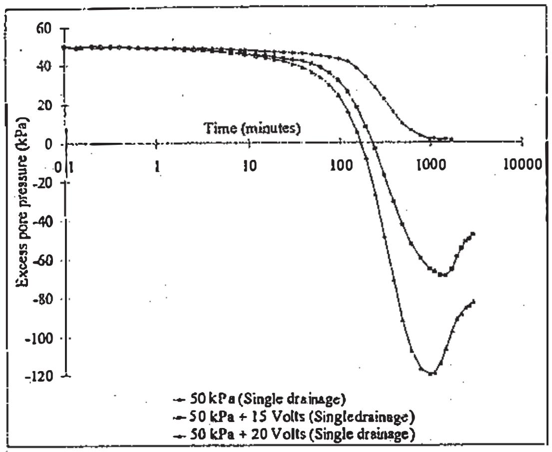


Figure 14

signifying full dissipation of the positive pore pressure. As the test continued, the pore pressure at the anode decreased to a value lower than the back pressure, signifying the generation of the negative pore pressure. Upon reaching a maximum value, the pore pressure increased before stabilising at a lower negative value. Figure 14 shows that the positive pore pressure generated by the vertical loading was dissipated in a shorter time by the incorporation of electroosmotic consolidation. The results concur with the theoretical solution by Wan *et al* (1976).

Vertical Strain The vertical strain-time curves are as shown in Figure 15. The electroosmotically induced vertical strain curve was similar to the consolidation due to vertical load. The vertical strain increased from 9.7% for consolidation with a normal load to 13% with the incorporation of electroosmotic consolidation.

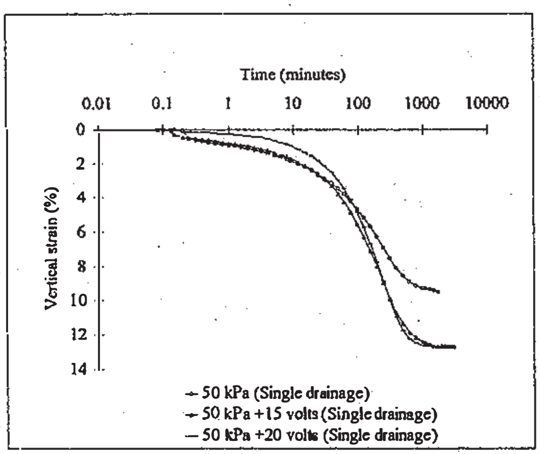


Figure 15

Moisture Content The moisture content profile is as shown in Figure 16. It indicates that moisture content for soil consolidated by a combination of vertical load and electroosmotic consolidation is lower than soil consolidated only with a vertical load. The shape of the moisture content distribution indicates that the lowest moisture content was at the centre of the specimen. However, the moisture content at the anode was still significantly reduced. The moisture content profile agrees with the finding of Lo *et al* (1991).

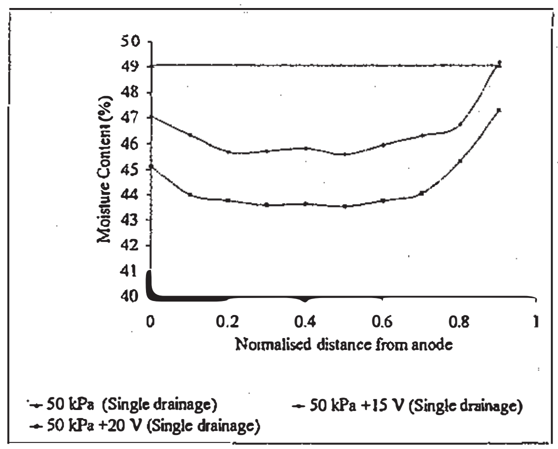


Figure 16

Shear Strength The undrained shear strength of the soil was determined at a depth of 30 mm from the surface. The test showed an increase in the shear strength up to 123% for an applied voltage of 20 volts, Table 5.

4.7 *Laboratory Tests - Soil-Reinforcement Bond*

The use of electrically conductive geosynthetic reinforcement can be shown to increase soil-reinforcement bond. A series of pullout tests have been undertaken at Newcastle University to illustrate this concept. The object of the tests was to study the effect of an electroconductive reinforcement on bond performance under undrained conditions. This represented the most severe case in the use of cohesive fill in a reinforced soil structure.

The tests were performed in a specially designed pullout cell and were conducted on soil strengthened by vertical overburden and also by a combination of overburden and electroosmotic consolidation.

Table 5 *Increase in shear strength when using electrokinetic reinforcement*

Type of Consolidation	*Shear Strength (kPa)*	*% Increase*
Soil consolidated with 50 kPa vertical pressure	11.17	-
Soil consolidated with 50 kPa vertical pressure and 15 volt applied voltage	19.16	71
Soil consolidated with 50 kPa vertical pressure and 20 volt applied voltage	25.52	123

Commercially available grade E kaolin was used for the tests. The soil was mixed at a moisture content of about 52% which was about 3% below the liquid limit.

A circular 300 mm diameter non-woven need punched geosynthetic was used as a drainage media for the cathode electrode. A 600 mm copper wire was threaded through it to make it electrically conductive. The anode was made of "Paraweb" reinforcement (33 mm width and 330 mm length) with copper wire woven into its surface to make it conductive. The embedment length for the pullout test of the Paraweb was approximately 285 mm. Paraweb was chosen as an example of an established impermeable geosynthetic reinforcing strip. The anode and cathode configuration is as shown in Figure 17.

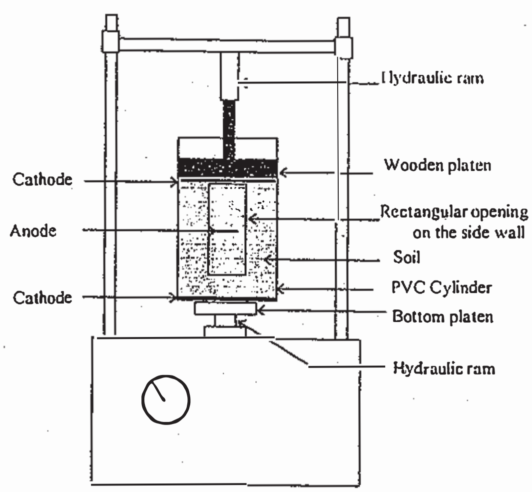


Figure 17: *Consolidation set-up of the pullout test*

A drainage layer which also served as the cathode was first placed at the bottom of the cell. The soil was then placed in the cell and compacted manually. The anode was inserted at mid height of the sample. Another layer of conductive geosynthetic was placed at the top of the soil sample forming a second cathode. Hence a two way drainage configuration was used to accelerate the consolidation process.

The soil was consolidated by the application of vertical pressure or by a combination of vertical pressure and electroosmotic consolidation. The vertical pressure was applied by a hydraulic jack capable of providing a constant vertical pressure. A potential difference of 30 volts was applied by means of a direct current power supply unit with a maximum output current of 1 ampere.

After the completion of the consolidation process, the cell was turned sideways and bolted to the platen of the frame of the triaxial testing machine as shown in Figure 18. The slit cover was then opened, exposing the geosynthetic reinforcement. The pullout tests were performed by lowering the platen at the rate of 1.5 mm per minute. The pullout resistance and the vertical displacement were automatically recorded by the load cell and the LVDT respectively. After the pullout tests, laboratory vane shear tests were carried out on the soil sample close to the anode. A small portion of the soil sample was also taken for the determination of the moisture content.

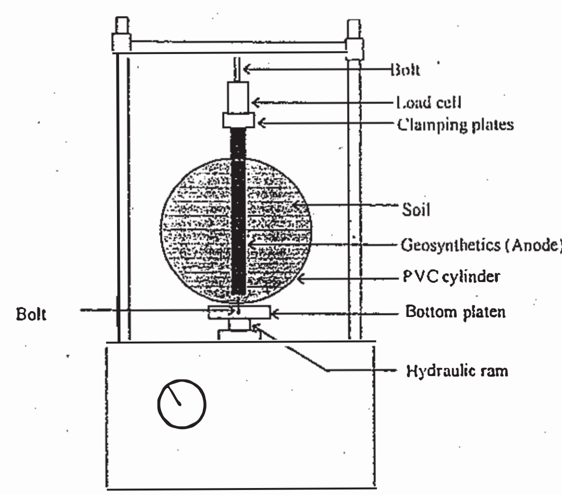


Figure 18: *Set-up for the pullout test*

Table 6 *Percentage increase in shear and soil reinforcement bond strength when using electrokinetic geosynthetic materials*

Consolidation pressure (kPa)	Increase in shear strength (%)	Increase in bond strength (%)
110	150	211
140	200	113
356	70	54

Bond Strength The increase in reinforcement-soil bond due to the use of electrically conductive reinforcement is shown in Table 6. The results show that the increase ranged from 54% to 211% at different overburden pressures.

5 CONCLUSIONS

Understanding the fundamental mechanisms involved in reinforced soil is the key to the development and correct use of soil reinforcements. Once the controlling parameters are established it is often possible to produce very effective improved reinforcements using established materials and manufacturing technologies.

The development of electrically conductive geosynthetic materials offers significant new application areas for geosynthetic products, ranging from accelerated consolidation of soft soils to soil decontamination and pollution control using bioremediation. The use of electrically conductive reinforcement in reinforced soil structures is beneficial as the concept is fully compatible with, and has a positive influence on, the parameters controlling the reinforcement mechanisms.

REFERENCES

Hassan, C A (1992) "The use of flexible transverse anchors in reinforced soil", PhD Thesis, University of Newcastle upon Tyne.

Hellawell, E (1994) "Modelling transport processes in soil due to hydraulic density and electrical gradients", PhD Thesis, Cambridge University, UK.

Heshmati, S (1993) "The action of geotextiles in providing combined drainage and reinforcement to cohesive soil", PhD Thesis, University of Newcastle upon Tyne.

Ingold, T S (1979a) "Some observations on failure mechanisms in reinforced clay", Proc. VI Panamerican Conference on Soil Mechanics and Foundation Engineering, Lima, Peru.

Ingold, T S (1979b) "Reinforced clay - a preliminary study using the triaxial apparatus", Proc. Int. Conf. on Soil Mechanics and Foundation Engineering, Brighton.

Jewell, R A (1992) "Strength and deformation in reinforced soil design", 4th Int. Conf. on Geotextiles, Geomembranes and Related Products, Dur Hoedt (ed), Rotterdam, pp.913-946, Balkema.

Jones, C J F P, Cripwell, J P and Bush, D I (1990) "Reinforced earth trial wall for Dewsbury ring road", Proc. Inst. Civil Engineering, Part 1 , Vol.88, pp.321-345.

Jones, C J F P and Hassan, C A (1992) "Reinforced soil formed using polymeric anchors", Earth Reinforcement Practice (eds. Ochiai, Hayashi and Otani), Vol. 1, pp.345-350, Balkema.

Lageman, R, Pool, W and Seffinga, G A (1989) "Electro-reclamation: state-of-the-art", In NATO/ CCMS 3rd Int. Conf. on Demonstration of Remedial Action Technologies for Contaminated Land and Groundwater, Nov. 6-9 1989, 115-136, Montreal, Canada.

Lee, K L (1976) "Reinforced earth - an old idea in a new setting", Int. Symp. New Horizons in Construction Materials, Envo. Pub. Co., Pennsylvania, Vol. 1.

Lo, K Y, Ho, K S and Inculet, I I (1991) "Field tests of electroosmotic strengthening of soft sensitive clay", Canadian Geotechnical Journal, Vol.28, 74-83.

Lo, K Y, Inculet, I I and Ho, K S (1991) "Electro-osmotic strengthening of soft sensitive clays", Canadian Geotechnical Journal, 28: 62-73.

Milligan, V (1994) "First applications of electro-osmosis to improve friction pile capacity - three decades later", In Proc. 13th Int. Conf on Soil Mechanics and Foundation Engineering, Jan. 5-10 1994, Vol.5, 34820, New Delhi, India. Balkema: Rotterdam, The Netherlands.

Mitchell, J K (1991) "Conduction phenomena: from theory to geotechnical practice", Geotechnique, Vol.41 (3), 299-340.

Mitchell, J K (1993) "Fundamentals of soil behavior", Wiley, New York.

Murray, R T (1977) "Research at TRRL to develop design criteria for reinforced earth", Symp. Reinforced Earth and Other Composite Soil Techniques, Heriot-Watt University, TRRL Sup. 457.

Murray, R T and Boden, B (1979) "Reinforced earth wall constructed with cohesive fill", Porc. Int. Conf on Soil Reinforcement, Paris.

Murray, R T, and Irwin, M J (1981) "A preliminary study of TRRL anchored earth", TRRL Supplementary Report 674, Transport and Research Road Laboratory, Crowthorne, UK.

Netlon Ltd (1996): Technical literature, Blackburn, UK.

Pasley, C W (1822) "Experiments in revetments", Vol.2, Murray, London.

Schlosser and Long (1974) "Recent results in French research on reinforced earth", Journal of the Construction Division, ASCE, Sept., pp.223-237.

Schlosser, F (1977) "Experience on reinforced earth in France", Symp. Reinforced Earth and other Composite Techniques, Heriot-Watt University, TRRL Sup. 457.

Schlosser, F and Elias, V (1978) "Friction in reinforced earth", Proc. ASCE Symp. Earth Reinforcement, Pittsburgh.

Skempton, A W (1954) "The pore pressure coefficients A and B", Geotechnique, Vol.4, pp.143-147.

Smith, M S, Bell, A L and Du Bois, D D (1979) "Embankment construction from marginal material", Proc. Int. Conf. on Soil Reinforcement, Paris.

Tan, S A Muhammad, N and Karunaratne, G P (1994) "Forming a thin sand seam on a clay slurry with the aid of a jute geotextile", Geotextiles and Geomembranes, 13(3): pp.147-163.

Tatsuoka, F and Yamauchi, H (1986) "A reinforcing method for steep clay slopes using a non-woven geotextile", Geotextiles and Geomembranes, Vol.4, No.3-4, pp.241-268, Elsevier.

Tatsuoka, F (1992) "Roles of facing rigidity in soil reinforcing", In Earth Reinforcement Practice, Ochiai, Hayashi and Otani (eds), pp.831-870, Balkema.

Tatsuoka, F, Murata, O and Tateyama, M (1992) "Permanent geosynthetic-reinforced soil retaining walls used for railway embankments in Japan", Geosynthetic-Reinforced Soil Retaining Walls, Wu (ed), Balkema, pp.101-130.

Tatsuoka, F, Tateyama, M and Koseki, J (1995) "Performance of geogrid-reinforced soil retaining walls during the Great Hanshin-Awaji Earthquake, January 17 1995", 1st Int. Conf. Earthquake Geotechnical Eng., IS-Tokyo 95, Nov.

Toh, C T, Chee, S K, Lee, C H and Wee, S H (1994) "Geotextile-bamboo fascine mattress for filling over very soft soil in Malaysia", Geotextiles and Geomembranes, 13 (6-7): 357-369.

van Olphen, H (1977) "An introduction to clay colloidal chemistry" (2nd Ed.), 318, Wiley Interscience: New York, USA.

Walkinshaw, J L (1975) "Reinforced earth construction", Dept. of Transportation FHWA Region 15, Demonstration Project No. 18.

Wan, T W and Mitchell, J K (1976) "Electro-osmotic consolidations of soils", Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 102, (GT5), May.

Westergaard, H M (1938) "A problem of elasticity suggested by a problem in soil mechanics, soft material reinforced by numerous strong horizontal sheets", Harvard University,

Yamanouchi, T (1970) "Experimental study on the improvement of the bearing capacity of soft clay ground by laying a resinous net", Proc. Symp. Foundations Interbedded Sands, Perth, pp. 102-108.

Yano, K, Watari, Y and Yamanouchi, T (1985) "Earthwork on soft clay grounds using rope-netted fabrics", Proc. Recent Developments in Ground Improvement Techniques, Bangkok, Thailand, pp.225-237.

Zakaria, N-A B (1994) "Construction on supersoft soils using geogrids", PhD Thesis, University of Newcastle upon Tyne.