Technical Paper by J.G. Zornberg and J.K. Mitchell

REINFORCED SOIL STRUCTURES WITH POORLY DRAINING BACKFILLS. PART I: REINFORCEMENT INTERACTIONS AND FUNCTIONS

ABSTRACT: In this and a companion paper (Mitchell and Zornberg 1994), the use and performance of reinforced soil structures constructed with poorly draining and/or cohesive backfills is evaluated. This evaluation shows that proper design and construction can result in stable, durable, and economical earth structures. Permeable reinforcements may be especially useful for soil structures with poorly draining backfills because the drainage capabilities of the geosynthetic can dissipate excess pore water pressures, thus enhancing stability. Consequently, the design of a safe and economical structure should address two aspects specific to poorly draining backfills: the cohesive soil-reinforcement interaction and the reinforcement drainage characteristics. The present paper focuses on experimental and analytical studies undertaken to evaluate these issues. Tensile strength, durability, and creep response of geosynthetics embedded in marginal soils are also addressed. There is strong experimental evidence that permeable inclusions can effectively reinforce clay structures.

KEYWORDS: Soil Reinforcement, Marginal Backfill, Cohesive Backfill, Triaxial Tests, Pullout Tests, Shearbox Tests, Transmissivity, Pore Water Pressures, Confined Tensile Strength, Durability, Creep.

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1 INTRODUCTION

Reinforced soil, an engineered composite material, is now extensively used for construction of earth retaining walls and embankment slopes, and in the stabilization of embankments placed over soft ground. A number of reinforcement types and proprietary systems have been developed, which offer the advantages of simple design, ease of construction, low cost, and the ability to tolerate large deformations without structural distress.

Conventionally, free draining granular material is specified for the backfill material of reinforced soil structures. Although there are several reasons for requiring good quality granular backfill, this specification has limited the use of reinforced soil structures in cases where such material is not readily available. Steel has been the most widely used reinforcement material and, since poorly draining soils are usually saturated, the possibility of corrosion of these reinforcements is high. The inherent low strength, moisture instability, possible volume changes, and creep potential of poorly draining soils are other concerns that have precluded their extensive use as backfills in reinforced soil structures. With the introduction of polymer geotextiles and geogrids, non-corrosive reinforcement systems are now available. However, considering the limited experience with cohesive backfills, common practice has been to avoid using lower quality soils in geosynthetically reinforced constructions whenever possible (Mitchell and Christopher 1990).

With the development of improved reinforcement materials and systems, as well as the expanding need for construction using alternative backfills, it is useful to evaluate the present status of reinforced structures with poorly draining backfill. This evaluation was motivated both by the lack of consensus on the mechanisms involved in cohesive soil-reinforcement interaction, and by the belief that proper design and construction can result in stable, durable, and economical reinforced soil structures. With the reassessment of different interpretations that have been put forth to explain the observed behavior, development of a consistent design methodology for reinforced soil structures with poorly draining backfills may then be possible.

The use of backfill soils capable of developing positive pore water pressures either during construction or after rainfall events is evaluated for both reinforced soil walls and slopes. Other applications, such as reinforced foundations and the use of geosynthetics to stabilize embankments over soft soils are outside the scope of this paper. Since the poor drainage characteristics of clays and silts are of major concern for the structure design, they are termed "poorly draining backfills" herein. Other terms, such as low-quality, cohesive, fine-grained, or marginal backfills have also been used in the technical literature to refer to these fill materials.

Experimental research done to investigate the cohesive soil-reinforcement interaction and the drainage function of reinforcement elements is reviewed in this paper. In a companion paper (Mitchell and Zornberg 1994), potential applications of marginal soils in reinforced soil construction and lessons learned from case histories are addressed.

2 CURRENT STANDARDS FOR BACKFILL MATERIALS IN REINFORCED SOIL STRUCTURES

Well graded, free draining granular material is usually specified for construction of reinforced soil walls and embankments. Gradation and soundness limits are given in the FHWA specifications for mechanically stabilized earth walls, as recommended by AASHTO-AGC-ARTBA, Joint Committee Task Force 27 (Table 1, Christopher et al. 1990).

The plasticity index for the backfill is also specified (PI $\leq$ 6 for walls and PI $\leq$ 20 for slopes), and magnesium sulfate soundness loss of less than 30% after four cycles is required. The maximum aggregate size should be limited to 19 mm (3/4 inch) for extensible reinforcement unless field tests are performed to evaluate potential strength reductions due to reinforcement damage during construction.

Some concerns about the use of poorly draining soils for reinforced soil construction have been (Mitchell 1981; Jewell and Jones 1981):

- Buildup of pore water pressures may reduce the backfill soil strength. Furthermore, the drained frictional strength of cohesive soils is intrinsically lower than that of cohesionless soils.
- Poorly draining cohesive soils are chemically more aggressive than cohesionless soils, and this can increase the rate of corrosion of metallic reinforcements.
- Post-construction movements may occur under sustained stresses because of the higher creep potential in poorly draining soils.
- Poorly draining soils are usually more difficult to compact.

However, these concerns may represent unrealistic restrictions in actual practice. In fact, many highway embankments are constructed of compacted clays, and to preclude their use when reinforcement is required for stability may be overly conservative. In many cases, buildup of excess pore water pressures can be avoided by adopting suitable construction techniques and drainage systems involving use of permeable reinforcements. In relation to the long-term performance issues of geotextile degradation and creep deformations, the cases reported in the literature have shown encouraging results. Furthermore, the use of geosynthetics such as nonwoven geotextile sheets has been reported to allow better compaction of cohesive soils.

Table 1. Gradation limits as recommended by AASHTO-AGC-ARTBA, Joint Committee Task Force 27.

U.S. Sieve Size	Percent Passing (walls)	Percent Passing (slopes)
4 inch (100 mm)	100	100-75
No. 4 (4.75 mm)		100-20
No. 40 (0.425 mm)	0-60	0-60
No. 200 (0.075 mm)	0-15	0-15

3 INTERACTION MECHANISMS OBSERVED USING TRIAXIAL TESTS

A number of experimental studies using triaxial tests have been done to develop an understanding of the interaction between cohesive soil and different reinforcement systems. Characteristics and conclusions drawn from the results of triaxial tests on reinforced specimens using poorly draining soils are summarized in Table 2, and an evaluation of many of these studies is presented next in this section. The general approach using triaxial tests has been to determine the strength of the unreinforced soil and the apparent strength of the same soil containing reinforcements placed at various spacings within the cylindrical sample. In this way, the change in strength caused by the reinforcement could be quantified using a *strength ratio*, which is the deviator stress at failure measured in the reinforced sample divided by the deviator stress at failure measured in the unreinforced sample.

Studies on reinforced clay using the triaxial apparatus were first reported by Ingold (1979). Results of undrained compression tests on normally consolidated, cylindrical clay samples reinforced using several disks of reinforcement were subsequently presented by Ingold and Miller (1982). The reinforcement material used was either aluminum foil or porous plastic. The test results showed that reinforcing clay specimens with continuous horizontal layers of aluminum foil caused reductions in undrained axisymmetric compressive strength of more than 50% relative to unreinforced samples. The premature failure of the specimen was attributed to pore water pressures induced in the reinforced specimen which were greatly in excess of those measured in a similar unreinforced specimen. Evidence of pore pressure generation during shear was further substantiated when tests were performed using the same clay, but with continuous horizontal porous reinforcement. In this case, the porous reinforcement was found to partially dissipate the pore water pressures induced in the clay, thus averting premature failure. Indeed, as the porous reinforcement spacing was decreased, the compressive strength of the reinforced specimen was found to increase substantially beyond that of the unreinforced specimen.

Figure 1 shows results for rapid shearing of fully saturated clay with permeable reinforcement. These samples were sheared at a strain rate of 2% per minute; this rate being deemed compatible with undrained shear in an unreinforced sample. Figure 2 shows test results for constant volume shearing, i.e. true undrained loading, of kaolin reinforced with porous plastic. Unexpectedly, there was also strength increase in this truly undrained condition. A suggested explanation to this phenomenon was that porous reinforcements decreased the Skempton pore pressure parameter A of the cohesive soil and hence increased the minor effective principal stress giving a higher undrained strength.

If the undrained compressive strength of a clay specimen can be increased by the introduction of porous reinforcement to partially dissipate the pore water pressures induced by shear, then it would be expected that even greater strength increases would be obtained if pore water pressures were allowed to fully dissipate, as occurs in a drained compression test. Ingold and Miller (1983) carried out a number of tests to investigate the drained behavior of normally consolidated clay reinforced with porous plastic disks. Test results showed that the effect of decreasing the spacing between the horizontal layers of reinforcement was an increase in both the drained shear strength

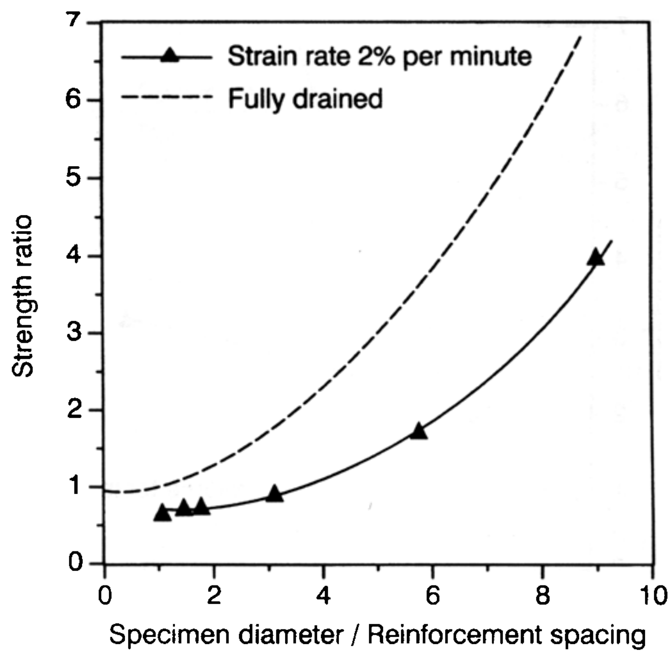


Figure 1. Effect of reinforcement spacing on strength ratio for rapid shearing of saturated clay specimens with permeable reinforcements (after Ingold and Miller 1982).
(Note: Strength ratio = Strength of reinforced specimen / Strength of unreinforced specimen)

and the secant deformation modulus of the reinforced sample. The stress-strain curves for one of the test series on reinforced kaolin clay are given in Figure 3. The ratio of sample diameter to reinforcement spacing is indicated as ρ_r in this figure. The unreinforced sample, with a ratio $\rho_r = 0.5$, is also indicated. Based on the results of a radiographic investigation, the strength enhancement was attributed, as in the case of sand reinforcement, to radial strain control arising from shear stress mobilized on the soil-reinforcement interface.

Rapid triaxial tests on partly saturated clay using impermeable reinforcement were also performed by Ingold (1985). As degree of saturation decreased, the strength ratio increased until, at a degree of saturation of approximately 70%, the strength ratio was equal to the one obtained under fully drained conditions. The results for 76 mm high samples with 6 mm reinforcement spacing are given in Figure 4, plotted in the form of a strength ratio against degree of saturation. As can be seen, there is a well–defined linear relationship between strength and degree of saturation, which appears to be independent of cell pressure. The practical implications of these results are that rapid construction using fully or nearly saturated clay fill could prove unstable if continuous impermeable reinforcements are used.

Fabian and Fourie (1986) presented the results of a series of undrained triaxial tests performed on silty clay samples reinforced with various geosynthetics having different in-plane transmissivities, including woven geotextiles, nonwovens, and geogrids.

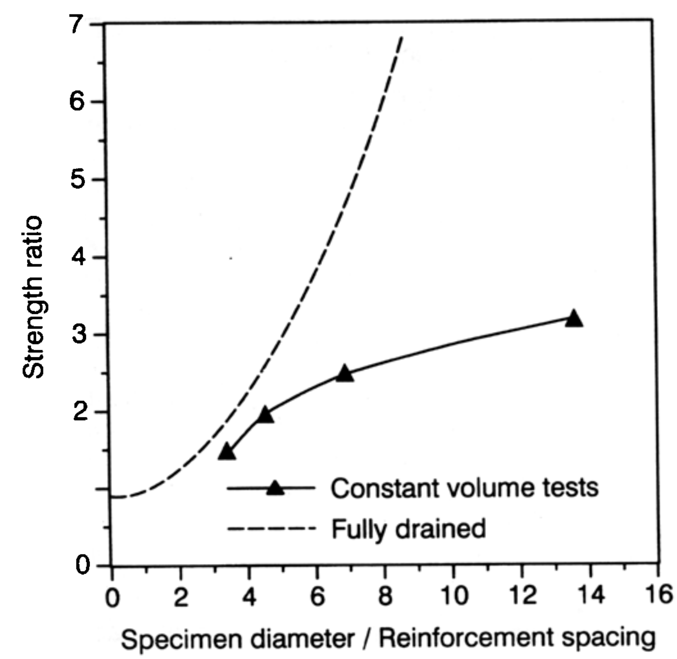


Figure 2. Effect of reinforcement spacing on strength ratio for constant volume shearing of kaolin specimens reinforced with porous plastic (after Ingold and Miller 1982).
(Note: Strength ratio = Strength of reinforced specimen / Strength of unreinforced specimen)

Unconsolidated undrained and consolidated undrained triaxial tests were done to determine the relationship between geotextile permeability and undrained strength. Their results showed that reinforcements with high transmissivity can increase the undrained strength of the clay by up to almost 40%, while reinforcements with low transmissivity can decrease the undrained strength by a similar magnitude. It was also reported that the strength ratio increased with the moisture content of the sample. This was because the undrained strength of an unreinforced clay sample decreases as moisture content increases, while the undrained strength of a reinforced clay sample was less affected by increases in moisture content. No significant strength increase was observed for samples reinforced with geogrids.

Although results from the above mentioned tests showed a decrease in strength when impermeable reinforcement was used, Al-Omari et al. (1987) obtained encouraging results using geomesh reinforced clay specimens. They presented the strength ratios obtained from 15 undrained triaxial tests on geomesh reinforced kaolin clay specimens having an overconsolidation ratio of three. Depending on the number of reinforcement layers, significant increase in the undrained strength was reported. The harmful effect of pore water pressure generation on the soil-reinforcement interface was considered to be mitigated. Results from both consolidated undrained and consolidated drained triaxial tests on overconsolidated geomesh reinforced clay were subsequently presented by Al-Omari et al. (1989). The geomesh stiffness, number of reinforcing layers, and the confining pressure were varied. The geomesh reinforcement enhanced

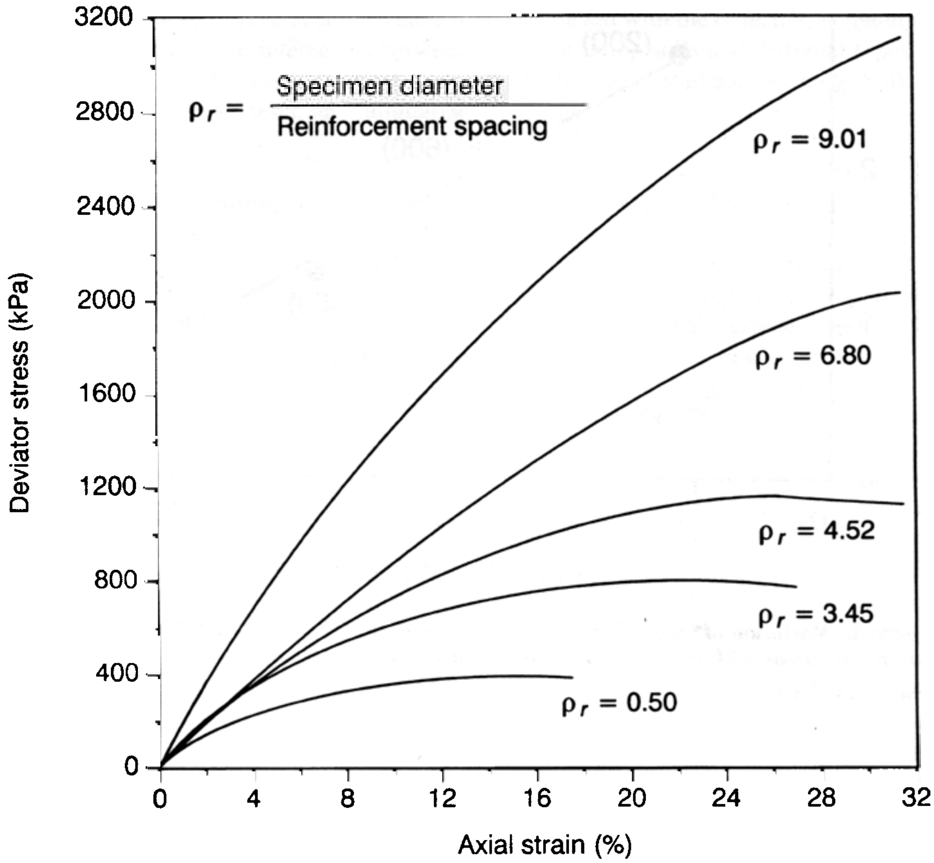


Figure 3. Stress-strain behavior of reinforced kaolin specimens during drained triaxial loading (cell pressure 250 kPa) (after Ingold and Miller 1983).

the strength in both the undrained and drained conditions. For undrained loading, the effective stress failure envelope of reinforced clay was reported to be parallel to the envelope for unreinforced clay, but with a greater cohesion intercept. For drained loading, the failure envelope of the reinforced clay indicated an increased friction angle.

The use of nonwoven geotextiles for reinforcing a near-saturated silty clay was evaluated by Ling and Tatsuoka (1993) using a plane strain device. The reinforcement effect, in terms of strength and stiffness, was reported to be more significant for anisotropically consolidated specimens than for isotropically consolidated specimens. In both cases, the reinforcement effect was greater in the drained tests than in the undrained tests. At small strain levels, excess pore water pressures adversely affected the stress-strain response of the reinforced soil samples tested under undrained conditions. In the drained tests, tensile stresses were mobilized in the geotextile ensuring a positive reinforcement effect.

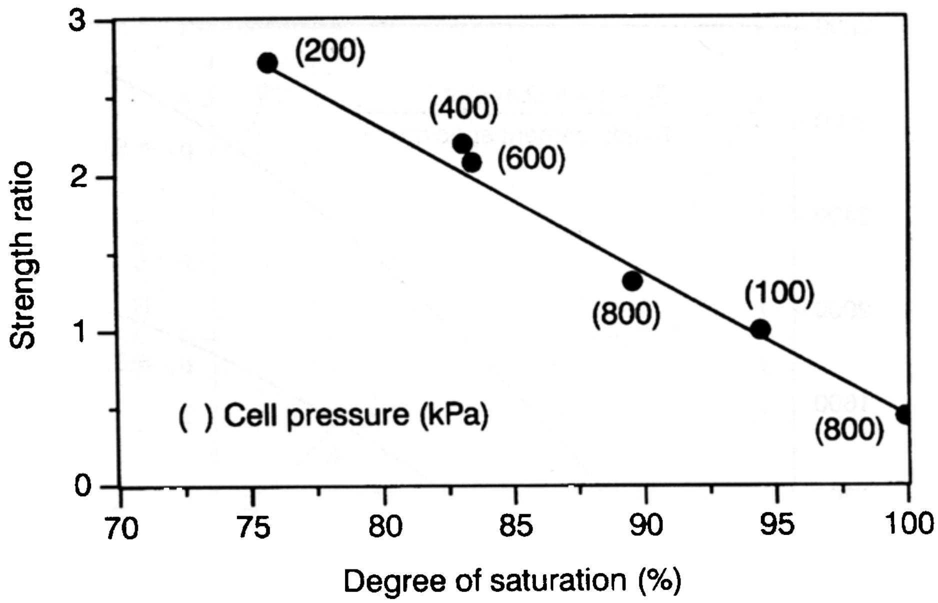


Figure 4. Variation of strength ratio with degree of saturation for rapid triaxial tests on kaolin specimens with impermeable reinforcements (after Ingold 1985).

(Note: Strength ratio = Strength of reinforced specimen / Strength of unreinforced specimen)

Strength improvements measured by triaxial compression tests do not correspond quantitatively to the improvement expected in full-scale reinforced soil structures because triaxial tests do not necessarily duplicate soil stresses and reinforcement tensions developed in the field. Nonetheless, triaxial test results on reinforced samples provide qualitative information on the strength of the soil-reinforcement composite, thus contributing to a better understanding of the nature of improvements that may be expected in full-scale soil structures. Although results obtained by different investigators using impermeable reinforcement have led to some contradictory conclusions, triaxial test results have clearly shown that poorly draining soils can be reinforced with properly selected permeable geotextiles.

4 INTERACTION MECHANISMS OBSERVED USING SHEARBOX AND PULLOUT TESTS

As the backfill material of a reinforced soil structure deforms under load, relative movements develop between the reinforcement and the soil, which mobilize bond stresses on the soil-reinforcement interface. Interface strength test results have been reported both in terms of an equivalent friction angle and an adhesion value. Collios et al. (1980) introduced the concept of *contact efficiency*, which is the ratio of the friction angle or cohesion of the soil-reinforcement interface to the friction angle or cohesion of the soil. Several studies have been conducted with the objective of quantitatively determining the interaction between poorly draining soils and different types of reinforcements. The characteristics and results of these studies, performed using shearbox and pullout tests, are summarized in Table 3.

4.1 Poorly Draining Soil-Metallic Reinforcement Interaction

The dilatancy of the compacted granular backfill has been recognized as a major factor to explain the high interface friction obtained from pullout tests using ribbed metallic strips (Guilloux et al. 1979). The resulting additional vertical stress at the reinforcements mobilizes substantial friction in spite of the narrowness of the reinforcements. Consequently, the use of granular backfill in these reinforcement systems is not only important to prevent an undrained condition, but also to induce dilatant behavior in the compacted backfill.

A laboratory study was performed by Elias (1979) to analyze the possibility of using fine grained backfills in Reinforced Earth structures. This study focused on the determination of fine soil-reinforcement friction parameters and on a qualitative evaluation of creep characteristics of ribbed reinforcements in fine grained soils. Pullout tests were performed using samples compacted at or near their optimum moisture contents. All the tested soils were either non-plastic or of low plasticity and exhibited relatively large values of undrained shear strength. The apparent friction coefficient (average peak shear stress along the strip divided by the normal pressure on the strip) was, contrary to pullout results obtained using cohesionless soils, less than the drained friction angle of the soil as measured by direct shear tests. Figure 5 shows the results of pullout tests using ribbed reinforcing strips in residual soils with different fines content. The apparent friction coefficient varies considerably with the normal pressure applied to the strip and, at all pressures, there is a drastic reduction in the coefficient magnitude with increasing fines content (Elias and Swanson 1983). The effect of compaction moisture content on the apparent friction coefficient can be observed in Figure 6, which shows a significant decrease when compaction moisture content was only 2% above optimum.

Ingold (1981) carried out shear box tests to investigate the undrained behavior of inclined reinforcements embedded in clay. Metallic reinforcements (mild steel Z-plate, plain plate, and corrugated plate), as well as polythene and polyethylene meshes were used in this study. Test results were compared using adhesion factor values that relate the apparent undrained shear strength of the reinforced clay to the true undrained shear strength of the unreinforced clay. Although the corrugated steel plate was found to perform better than the other metallic inclusions, the polyethylene mesh was the most efficient of all the reinforcements tested.

Laboratory and field pullout tests were conducted using steel grid reinforcements with cohesive-frictional backfill soils (Bergado et al. 1992a). The laboratory pullout tests were performed using a large scale pullout apparatus, and the field pullout tests were done on dummy welded-wire reinforcements embedded in a full-scale reinforced test structure. Three different low-quality backfill soils, namely, clayey sand, lateritic soil, and weathered Bangkok clay were used. The backfill material was compacted to densities of about 95% of the Standard Proctor maximum density, on the dry side of

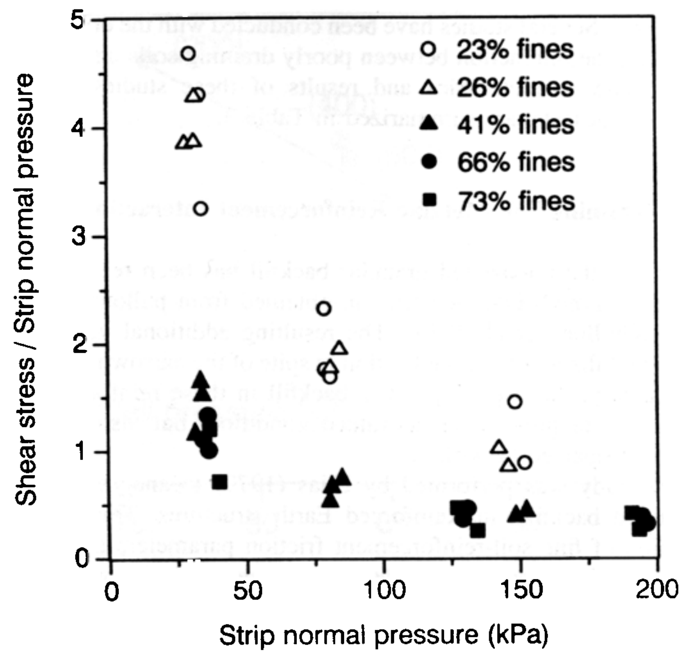


Figure 5. Strip pullout capacity versus normal pressure for soils with different fines content (redrawn after Elias and Swanson 1983).

the optimum moisture content. The laboratory pullout test results were reported to provide a conservative approximation of the field pullout resistance of the grid reinforcements. Additional pullout tests were carried out using welded-wire mild steel grids embedded in weathered Bangkok clay, after compacting the backfill material both dry and wet of optimum moisture content (Bergado et al. 1992b). Typical load-displacement relationships for dry and wet side of optimum compaction, shown in Figure 7, indicate that the pullout resistance is significantly higher for backfill compacted dry than wet of optimum. Tests performed using reinforcements with their transverse members removed showed that the pullout resistance was carried mainly by the passive component mobilized by transverse members of the welded-wire grid, with only a minimal frictional contribution from the longitudinal bars.

This review of published results on the interaction between poorly draining soils and metallic reinforcements shows that, although some metallic reinforcements were observed to effectively reinforce poorly draining soils if moisture content was low, the results were very dependent on the compaction water content. The effect of backfill saturation requires further study and the concern of metal corrosion should be addressed prior to the long-term use of metallic reinforcements in poorly draining soils.

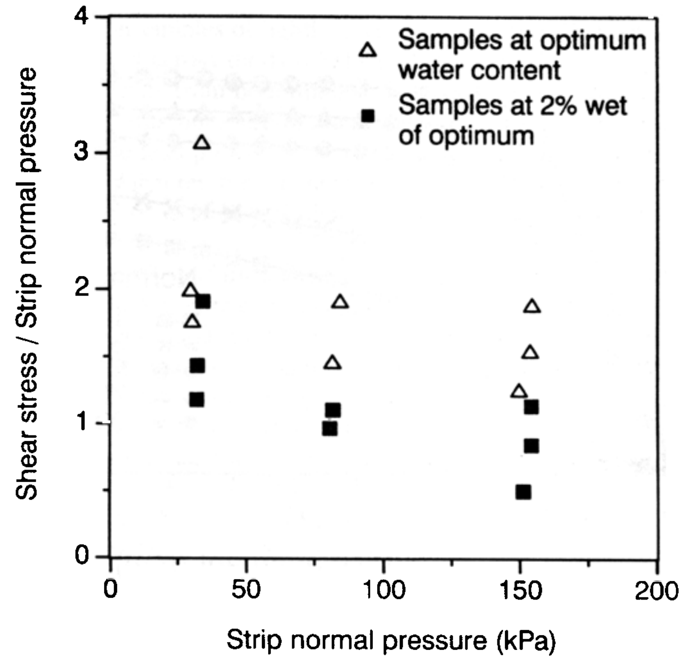
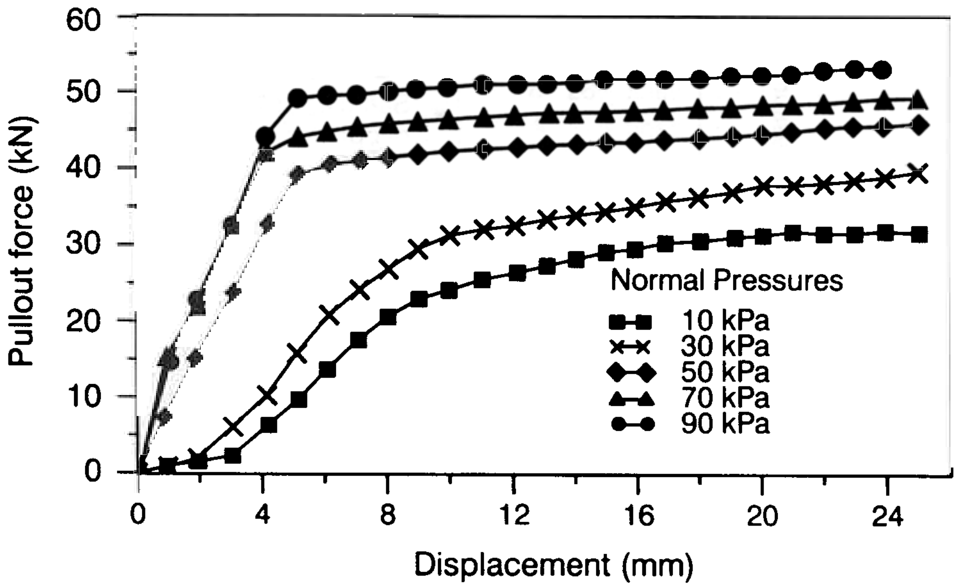


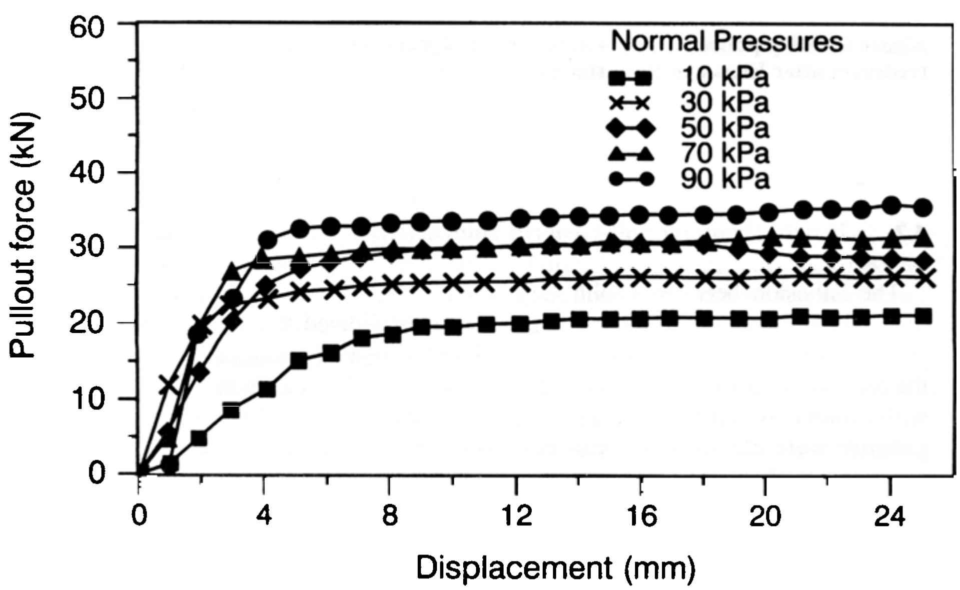
Figure 6. Strip pullout capacity versus normal pressure for samples at two water contents (redrawn after Elias and Swanson 1983).

4.2 Poorly Draining Soil-Geogrid Interaction

The adhesion between kaolin clay and polymeric and other reinforcements was investigated by Ingold (1981) using both unconsolidated undrained direct shear and pullout tests. Contact efficiency was initially investigated using a direct shear box with the reinforcements inclined across the two halves of the box. In these tests, performed with kaolin clay compacted at a moisture content close to its plastic limit, polyethylene geogrids were the most efficient reinforcements. The inclined reinforcements caused an apparent shear strength increase in the clay samples, which was interpreted as induced by the resistance of the reinforcement to rotation rather than pullout. Consequently, the bending stiffness of the reinforcement was considered to have a significant effect. Tests were then performed using horizontal polyethylene geogrids. The contact efficiencies were markedly dependent on test method, with generally higher values being obtained from the shear box and lower values from the pullout apparatus. An undrained pullout resistance equation based on these results was formulated by Ingold (1983), suggesting that geogrid pullout resistance is a function of the area of geogrid members normal and parallel to the direction of applied load rather than the embedded plan area.



(a) samples compacted dry of optimum water content



(b) samples compacted wet of optimum water content

Figure 7. Typical load-displacement curves from pullout tests using welded-wire steel grids in weathered clay (after Bergado et al. 1992b).

Direct shear tests on samples of lightly over-consolidated kaolin, also using geogrid reinforcements inclined across the two halves of the box, were performed by Jewell and Jones (1981). Results from both drained and quick direct shear tests showed that the reinforced kaolin was both stronger and stiffer than the unreinforced clay. Thin vertical threads of fine lead powder were introduced into each sample, and the prints of radiographs showed that reinforcement caused a wider zone of kaolin to be deformed. The deformation pattern is similar to that obtained by Shewbridge and Sitar (1989) on reinforcement-sand composites.

Brand and Duffy (1987) performed additional pullout tests on four types of polymeric geogrids. An expansive smectite clay was placed with a low moisture content, approximately 10%, to simulate initial placement conditions. Since similar pullout resistance values were obtained using the different geogrids, the authors concluded that pullout resistance of geogrids in low moisture content clays was relatively insensitive to the geogrid characteristics. Direct shear and pullout tests on polymer geogrid embedded in cohesive soils (clayey sand and weathered clay) were also performed by Bergado et al. (1987). Based on the results from tests performed on soil specimens compacted to 95% of Standard Proctor density at optimum moisture content, they concluded that cohesive soils can be effective backfill material for geogrid reinforced embankments. The pullout resistance of the geogrids using cohesive backfill was interpreted to be due to adhesion between the soil and the reinforcement on the plan area of the geogrids as well as passive resistance of the soil in front of all transverse members. The interaction of weathered clay with steel, bamboo, or polymeric geogrids was compared by Bergado et al. (1993). Soil specimens were compacted at the dry side of optimum to 95% of Standard Proctor density. While the steel grids moved as a rigid body during the pullout tests, the polymeric grids showed varying resistance mobilization along the reinforcement.

Results from both short- and long-term pullout tests on polymeric geogrids and geotextiles, performed in cohesive soils, were reported by Christopher and Berg (1990). The short-term tests were performed to evaluate the influence of pore water pressure generation, while the long-term pullout tests were used to investigate the soil-geogrid creep response. Consistent test procedures in a large pullout box were used for all tests, with loading rates to failure varying from several hours to several months. Although different cohesive soils were used in the test programs, the soil characteristics were similar in terms of liquid limits and plasticity indices. The main variation between the soils was the as-placed water content, which resulted in a significant variation in undrained shear strength. Drained pullout resistances were not necessarily greater than the undrained ones. Figure 8 shows displacement measurements at the front and back of a sample compacted wet of optimum for both the undrained and drained conditions. The nearly constant movements of the front and rear gages observed in this particular test indicate failure by pullout. Pullout resistance values calculated using the interaction coefficients recommended by the manufacturers were conservative in relation to the experimental test results.

The reported results from shearbox and pullout tests performed using geogrids embedded in poorly draining backfills generally showed encouraging results. Most tests were performed on samples with moisture contents that simulate typical initial field placement conditions, that is, at optimum water content or dry of optimum. However, the field placement conditions of the backfill material may not correspond

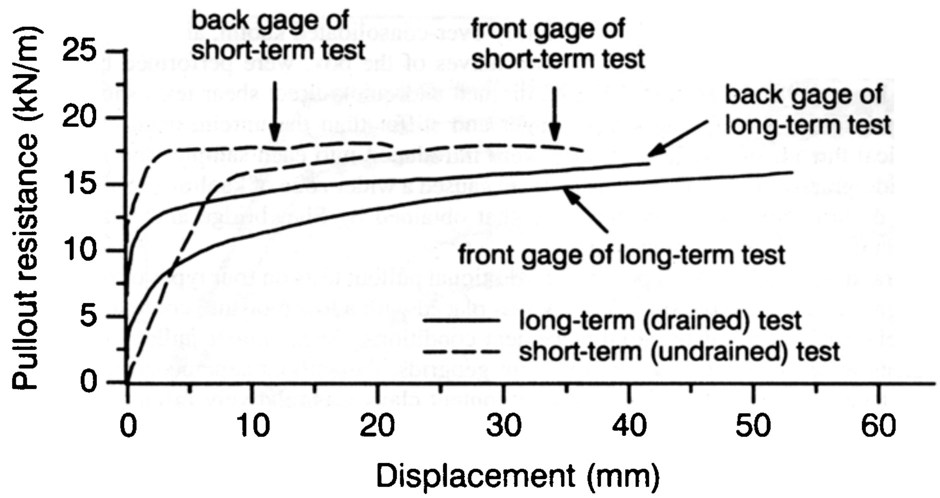


Figure 8. Load-displacement curves from pullout tests on polymer geogrids (after Christopher and Berg 1990).

to the worst case scenario over the life of the structure. Consequently, the effect of fill water content changes on the performance of reinforced structures should be further investigated. Drainage and collection systems should be used to help maintain a low moisture content in the structure backfill.

4.3 Poorly Draining Soil-Geotextile Interaction

Direct shear tests using three geotextiles (a thick nonwoven, a thin nonwoven, and a woven) were performed by Lafleur et al. (1987) to evaluate the contact efficiency of geotextiles in medium plasticity lateritic gravels and highly plastic clays. The shear testing program was undertaken to justify the choice of a geotextile in reinforced fill applications involving lateritic gravel embankments laid over a soft clay foundation. The soil strength and the soil-geotextile adherence parameters were obtained using strain rates slow enough to create a drained condition. The contact efficiency was up to 1.0 in the case of nonwovens and ranging between 0.5 to 0.6 for wovens. The smooth surface of the woven geotextiles did not permit particle penetration and the creation of strong adhesion between the soil and the geosynthetic. However, as indicated in Figure 9, the relative displacement between the reinforcement and the soil required to mobilize the total shearing resistance was significantly larger in the tests performed with the lower-stiffness nonwoven geotextiles. Lafleur et al. concluded that the nonwoven geotextiles offer superior performance because the adherence values are higher and, in cases where the loads are applied at a fast rate, they can convey water coming out of the soil from consolidation.

The contact efficiencies for five geosynthetics and seven soils, measured using a modified direct shear device, were reported by Williams and Houlihan (1987). Test

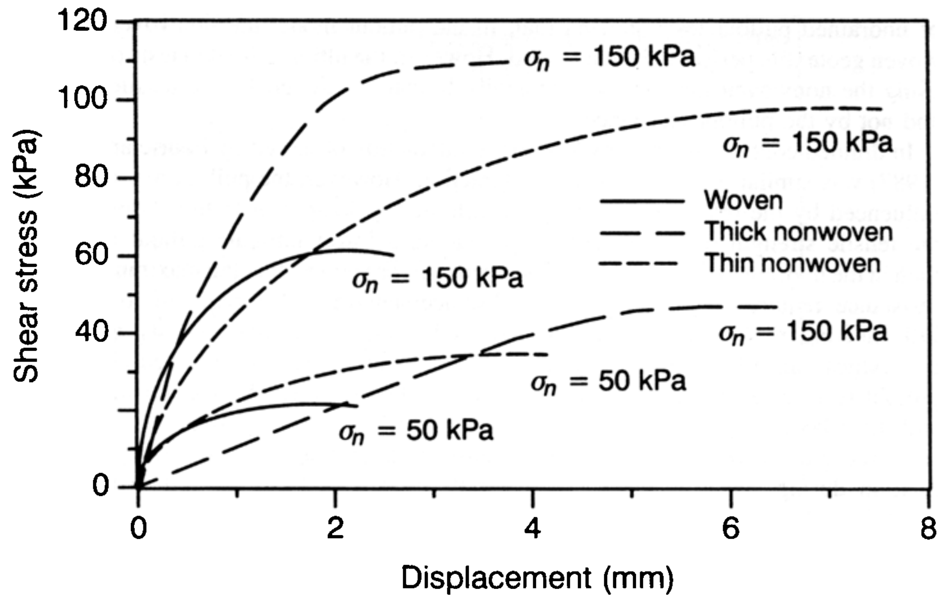


Figure 9. Direct shear test results for three types of geotextile in a plastic clay (moisture content 40%) (after Lafleur et al. 1987).

results indicated three primary modes of failure: sliding along the interface between the soil and the geosynthetic; failure along a surface in the soil parallel to the geosynthetic layer; and failure along a surface between two adjacent layers of geosynthetics. The location of the sliding surface and magnitudes of the interface friction parameters were found to be a function of the soil type, the soil water content and density, and the surface roughness of the geosynthetics. Sliding along a failure surface which develops within the soil, parallel to the geosynthetics, was reported for tests performed with nonwoven and woven geotextiles with cohesive soils. In contrast, sliding occurred on the interface between the soil and the geosynthetic in tests performed to evaluate the interface friction of cohesive soils with smooth sheets of polyvinyl chloride (PVC) and high density polyethylene (HDPE).

Fourie and Fabian (1987) performed shearbox and pullout tests to investigate the major factors governing the clay-geotextile interaction in both undrained and drained conditions. The undrained tests were actually rapid tests with shearing and pullout displacement rates of 0.9 mm/min. Woven and nonwoven geotextiles were used in the tests, performed in a small shearbox apparatus using a silty clay (CL) with a plasticity index of 13%. The authors identified the stiffness, the surface roughness and the transmissivity of the geotextile as the main factors affecting the undrained shearing interaction. The shearing strength of the clay was increased by the high-transmissivity nonwoven geotextile. At any stress level, the nonwoven reinforcement was reported to have a higher contact efficiency than the woven reinforcement. The pullout resistance was found to be strongly related to the shearing interaction, but it was also affected by the stiffness and the tensile strength of the geotextile. Load-displacement curves of undrained pullout tests showed that, in the pullout mode, the nonwoven and the woven geotextile performed equally well. However, the ultimate load in tests performed using the nonwoven material was generally limited by the geotextile tensile strength and not by the pullout resistance.

In drained conditions, the clay-geotextile interaction observed by Fourie and Fabian (1987) was similar to sand-geotextile interaction. However, the pullout resistance was influenced by the relaxation of the geotextile during long-term testing. This reduced the tensile strength of the geotextile, producing a lower ultimate pullout resistance than in the undrained tests. Load-displacement curves showed that the maximum pullout resistance required significantly more displacement than that required to mobilize full shearing resistance. The authors concluded that high-transmissivity nonwoven geotextiles can effectively reinforce clay in both undrained and drained shear. The pullout resistance of high-transmissivity geotextiles is limited by the tensile strength and the relaxation properties of the reinforcement. Woven geotextiles can effectively reinforce clay in drained conditions because of their rough surface, which induces dilation during shearing, but these materials do not perform as well in undrained conditions due to their low transmissivity.

The pullout resistance of woven geotextiles in cohesive soils was investigated by Gilbert et al. (1992) using 0.6 m by 0.6 m test specimens. Laboratory parameters were compared with prototype field tests. Three high-strength woven polyester geotextiles were tested using clays, silty clay, and silty sand specimens. The effect of increasing the rate of pullout deformation was to increase the apparent pullout resistance of the system. This increase was observed at water contents between the liquid and plastic limits and was attributed to viscosity mechanisms. Submergence below water decreased the apparent pullout resistance as a result of the loss of capillary tension. Slippage was not observed at the clay/geotextile interface, but within the soil mass, and it was interpreted that pore water pressures generated by shear strains were partly dissipated through the woven geotextiles.

Although an increase in pullout resistance was obtained by increasing the normal stress, this effect was dependent on the soil water content. Figure 10 shows how pullout resistance decreases with increasing water content and appears to reach a limiting value at about 40% water content for all normal loads. Results from laboratory pullout tests as well as full-size prototype field tests performed using low water content (26%) specimens showed increasing pullout resistance with increasing normal stress (Figure 11a). On the other hand, laboratory and full-size tests using soil at 40% water content showed that pullout resistance is affected by induced pore water pressures. This may be observed in Figure 11b, in which strength appears to be essentially constant and unaffected by normal stress.

Additional studies were done to investigate the clay-geotextile interaction for different applications and systems (Saxena and Budiman 1985; Eigenbrod and Locker 1987; Fabian and Fourie 1988; Makiuchi and Miyamori 1988; Garbulewski 1990; Gomes 1992). Test details and conclusions are summarized in Table 3.

This evaluation of shearbox and pullout tests done with geotextiles highlights the different results obtained using low permeability woven and permeable nonwoven reinforcements. Geotextiles with adequate in-plane transmissivity, namely nonwoven geotextiles, can be effectively used to reinforce clay structures under both rapid and

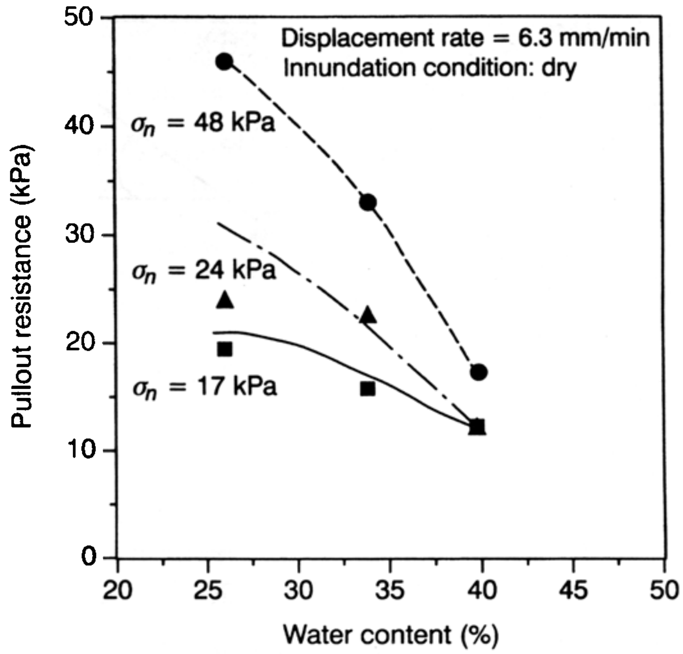


Figure 10. Effect of water content on the pullout resistance of a woven geotextile in a plastic clay (after Gilbert et al. 1992).

fully drained conditions. Reinforcement of marginal soils might be enhanced by combining the high tensile strength of woven geotextiles with the high transmissivity of nonwovens in a composite geotextile.

4.4 Poorly Draining Soil-Geomembrane Interaction

Even though geomembranes are not ordinarily used as reinforcement elements, evaluation of shearing resistance between various geomembranes and cohesive soils is important for other applications. Geomembranes used in solid waste disposal systems are often placed directly on low permeability compacted clay soils. Direct shear and pullout tests have been performed to evaluate the shear strength along clay-geomembrane interfaces, which may be critical to the stability of multilayered liner systems. A clear evidence of this hazard was the slope-stability failure of a Class I hazardous-waste landfill at Kettleman Hills (Mitchell et al. 1990; Seed et al. 1990) that resulted from slippage along interfaces within a multilayered liner system. Accordingly, a few significant references regarding geomembrane-cohesive soil interaction are reviewed herein.

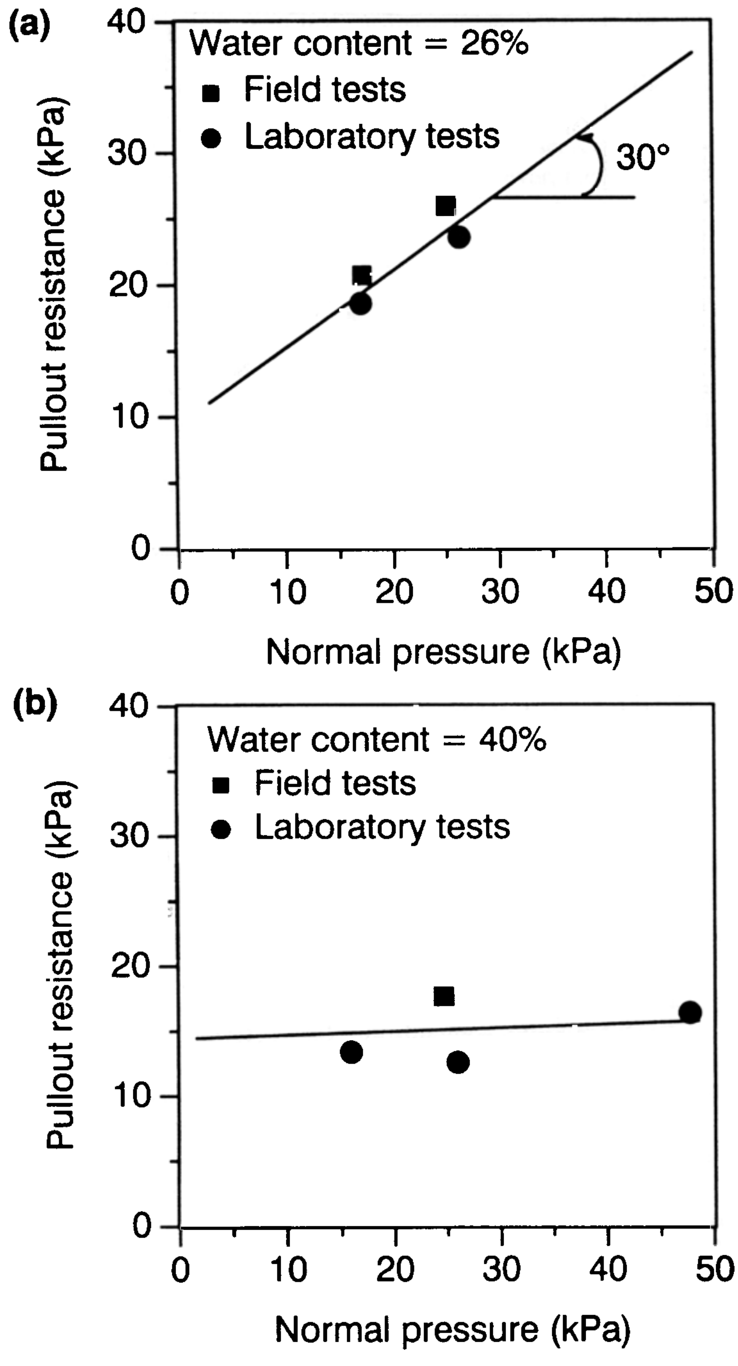


Figure 11. Pullout resistance of a woven geotextile versus normal load in clay specimens molded (a) at 26% water content; (b) at 40% water content (after Gilbert et al. 1992).

Koerner et al. (1986) determined the interface shear strength between various geomembranes and a number of cohesive soils using direct shear tests. Despite the low permeability of the soils, the tests were considered to be drained, as the soils were less than fully saturated (75-85%). A standard direct shear apparatus was considered to be useful for obtaining site-specific data on a production basis. The adhesion of geomembrane to soil was significantly lower than the cohesion value obtained for the soil. Conversely, the friction angle at the soil-geomembrane interface was reasonably high. Interface friction was as high as the soil friction angle for the case of soft (chlorinated polyethylene, ethylene propylene diene monomer) and textured (HDPE) geomembranes, being somewhat lower for harder (PVC, HDPE) geomembranes. The data base of adhesion and friction values for interfaces between common liner materials can be used for practical design considerations, such as preliminary assessments of stability.

A testing program performed to evaluate the shear resistances along different interfaces in a composite liner system was described by Mitchell et al. (1990). Both direct shear tests and pullout tests were carried out, and the obtained interface parameters were used for the stability analyses of the Kettleman Hills waste landfill slope failure. The interfaces between the various geosynthetics, and between these materials and the compacted clay in the liner system, were characterized by low frictional resistance, with interface friction angles as low as 8° for some combinations. The direct shear tests on HDPE-compacted clay liner interface samples were performed under two sets of conditions. The first test series was done on nonsaturated as-compacted samples, that showed residual friction angles between 11° and 14°. The second series was performed on samples initially compacted to field conditions and then submerged and soaked under light surcharges. In this case, the resulting interface-shear strengths were independent of the applied normal stress, exhibiting residual interface strengths of between 39 and 49 kPa. Shear tests on geotextile-compacted clay liner interfaces were performed using clay liner material first compacted to field conditions and then soaked under light surcharge. These direct shear tests were intended to represent unconsolidated undrained testing conditions, though the geotextile probably facilitated pore pressure dissipation at the geotextile-clay interface contact. A residual friction angle of 24° was obtained for this interface.

A wide range of interface shear strengths has been reported by different laboratories for apparently similar combinations of geomembranes and soil conditions. In order to investigate the causes of such differences, Seed and Boulanger (1991) analyzed shear strength data for smooth HDPE-compacted clay interfaces. Direct shear tests results, performed on smooth HDPE in contact with two different clay liner materials, showed that the interface strength varies greatly over the range of possible as-compacted conditions. Some of the test results, expressed as equivalent friction angles (ϕ_r), are shown in Figure 12. Two sets of results are shown: samples sheared as-compacted; and samples soaked under a small normal stress (12 kPa) prior to application of surcharge and undrained shear. As shown in Figure 12a, the interface shear strengths for samples sheared in as-compacted conditions are strongly influenced by compaction conditions (as-compacted density and water content). Interface friction angles may differ by a factor of two or more as a result of relatively minor variations in as-compacted dry density and water content. Zones roughly parallel to the zero air voids curve can be defined by drawing contours of shear strength values. The probable reasons for different shear strength zones were reported to be the degree-of-saturation and soil fabric effects

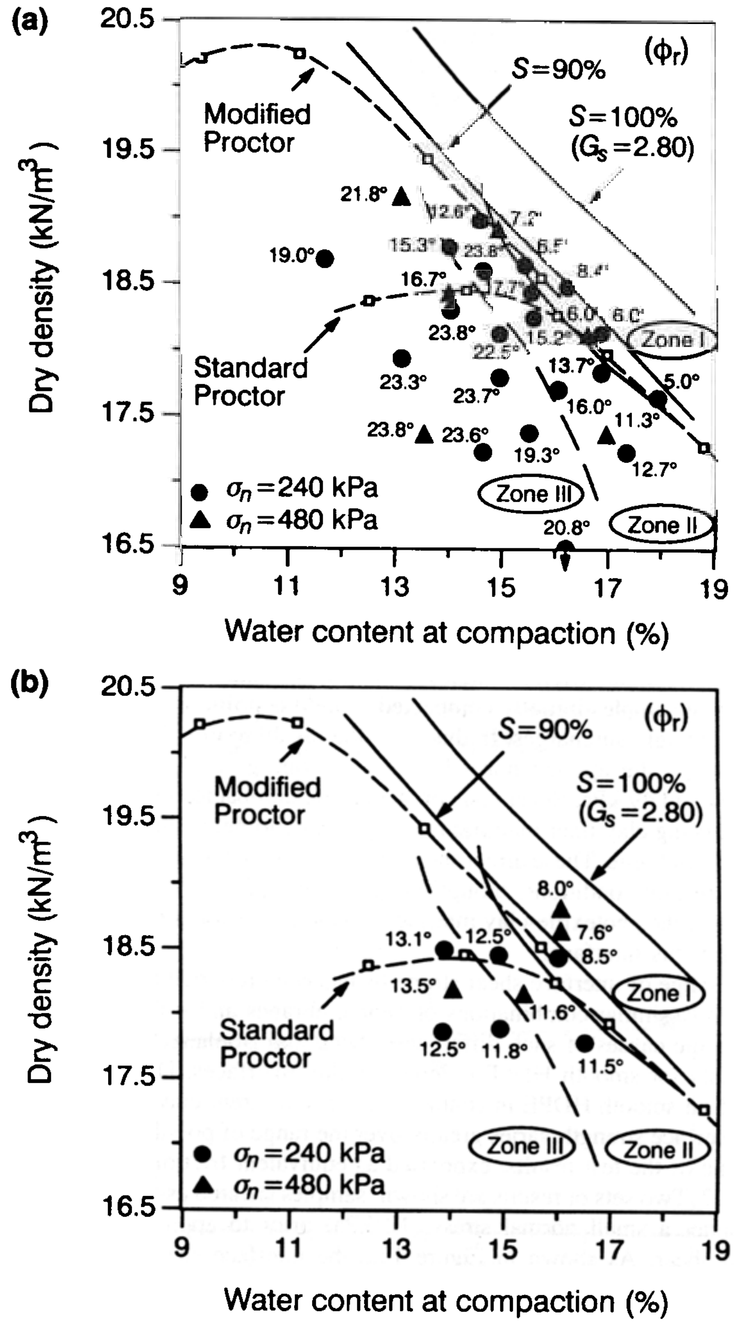


Figure 12. Shear strengths for smooth HDPE-compacted clayey till interfaces: (a) as-compacted samples; (b) samples sheared after initial pre-soaking (after Seed and Boulanger 1991).

(e.g. flocculated versus dispersed fabric). As shown in Figure 12b, the differences in strength among the zones are reduced as a result of pre-soaking. However, this is primarily a result of significant reductions in the strengths for samples in Zones II and III since the strengths of samples compacted in Zone I are not appreciably changed.

The results presented by Seed and Boulanger (1991) for cohesive soil-geomembrane interfaces suggest that compaction conditions and pre-soaking would greatly influence the interface strength between cohesive soils and other geosynthetics. Shearbox and pullout tests on metallic, geogrid, and geotextile reinforcements have not yet been done to fully address the effects of these conditions.

5 HYDRAULIC FUNCTION OF GEOSYNTHETIC REINFORCEMENTS

Permeable geosynthetic reinforcements may be especially useful for soil structures with poorly draining backfills because the drainage capabilities may help increase the structure stability by dissipating excess pore water pressures. In this way, the geosynthetic layers may work not only as reinforcements but also as lateral drains. In order to formulate a rational design that takes into account the in-plane transmissivity of the geosynthetic reinforcements, their hydraulic characteristics must be evaluated. Such evaluation includes the correct determination of the reinforcement drainage capacity, the analysis of the pore water pressure dissipation in the fill, and the assessment of the effect of pore water pressures on the structure stability.

5.1 Determination of In-Plane Hydraulic Conductivity

Relatively little study has been made of the in-plane hydraulic conductivity of geotextiles as compared to the cross-plane conductivity. The test methods commonly used for measuring the in-plane hydraulic conductivity of geotextiles are the parallel flow test (ASTM Standard D 4716), and the radial flow test. In the parallel flow device, flow occurs parallel to and between two rigid plates by applying a constant head difference across the specimen. The in-plane hydraulic conductivity is then measured by monitoring the flow rate while the geotextile is under confinement. The radial flow test uses a circular disk-shaped geotextile specimen that allows flow to enter the geotextile specimen at the inner circumference. The stream lines of the flow therefore radiate from the center of the circular disk outward in all directions.

However, the common tests used to determine the in-plane hydraulic conductivity do not always reproduce the geosynthetic field conditions. A test apparatus, capable of measuring geotextile transmissivities under specified constant hydraulic heads and under confinement, was described by Ling et al. (1990, 1993). Using this apparatus, a nonwoven geotextile and a woven/nonwoven composite geotextile were tested to measure their in-plane hydraulic conductivity under various normal stresses. Three methods of geotextile confinement were used to simulate the in-soil condition; rigid blocks, flexible membranes, and soil. Soil-confinement tests were performed by placing

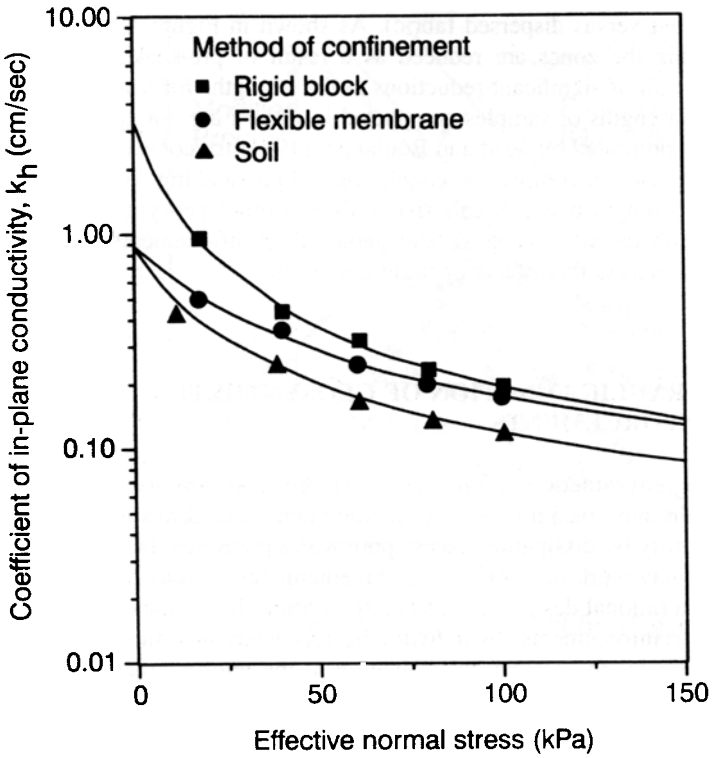


Figure 13. Relationship between coefficient of in-plane hydraulic conductivity and effective normal stress for a nonwoven geotextile (after Ling et al. 1993).

compacted cakes of volcanic ash clay on each side of the geotextile. Figure 13 shows the relationship between in-plane hydraulic conductivity and confining stress for the nonwoven geotextile. As would be expected, the in-plane hydraulic conductivity is strongly dependent upon the effective normal stress. Similar behavior was observed for the composite geotextile. As shown in the figure, the hydraulic conductivity of the geotextiles was influenced by the method of confinement. Differences in the measured hydraulic conductivity were attributed to interface flow, mainly under rigid block confinement, and to soil penetration and retention that occurs under soil confinement. Based on these results, it was concluded that in-plane hydraulic conductivity for a geotextile embedded in soil could be overestimated if the test is performed using block or membrane confinement.

Soil-confinement tests were also performed to determine the in-plane hydraulic conductivity of geotextile specimens retrieved from the field after several years of installation. The geotextile specimens were extracted from a test embankment built at the University of Tokyo (Tatsuoka and Yamauchi 1986). Figure 14 shows the in-plane hydraulic conductivities of the fresh and the exhumed nonwoven geotextiles. It can be observed that the field-retrieved specimens gave several times smaller in-plane hydraulic

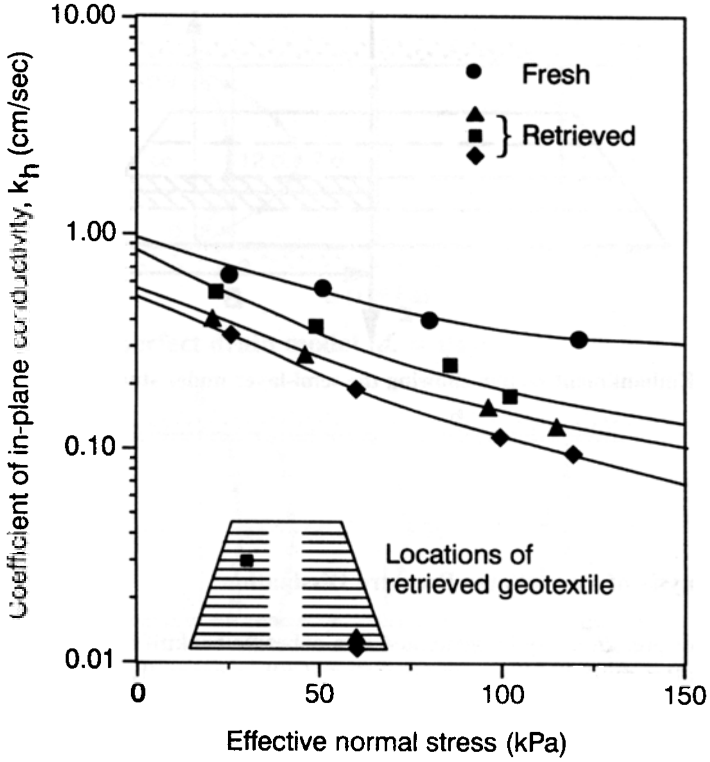


Figure 14. In-plane hydraulic conductivity of fresh and retrieved nonwoven geotextile specimens (after Ling et al. 1993).

conductivity than fresh specimens. The primary reason for the reduction of in-plane hydraulic conductivity was attributed to clogging of the geotextile by fine soil particles. A direct relationship was observed between the amount of soil retained in a geotextile and the reduction in its in-plane hydraulic conductivity. However, the reduction in in-plane hydraulic conductivity did not affect the performance of the embankment, which was satisfactory in terms of mechanical and hydraulic behavior. An equation proposed by Ling et al. (1993) shows that the reciprocal of hydraulic conductivity varies in a linear manner with effective normal stress. Additionally, a reduction factor was introduced to account for the long-term reduction in hydraulic conductivity due to soil-particle retention.

Additional field experience should be collected to further evaluate the possible clogging of geotextiles that function as reinforcement in marginal backfills. However, a preliminary assessment of the long-term equilibrium flow rate could be made using the clogging resistance criteria already designed to evaluate geotextile filters (e.g. Christopher and Fisher 1992). Different approaches to assess the long-term flow rate behavior and research needs for geotextiles used in filtration applications are addressed by Koerner et al. (1992).

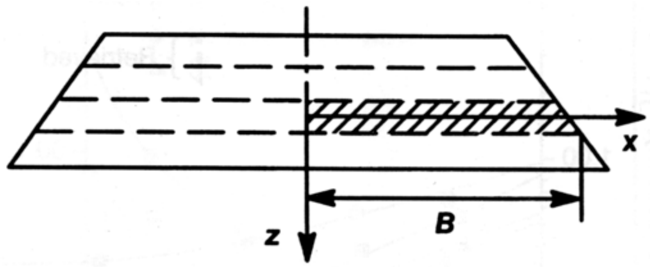


Figure 15. Embankment section showing the semi-layer under study (after Auriault et al. 1977).

5.2 Analysis of Pore Water Pressure Dissipation

Pore water pressure may be generated in a cohesive backfill during the construction process or after rainfalls. Since the compacted fill is generally unsaturated, the pore water pressure distribution in the soil mass is difficult to predict. A conservative estimate of such a distribution can be made if the backfill material is assumed to be fully saturated. This conservative assumption has led to theoretical methods to estimate the geotextile transmissivity requirements and the pore water pressure dissipation after construction of a reinforced clay structure.

Auriault et al. (1977) presented a theoretical treatment for the consolidation of a saturated fill placed between horizontal layers of permeable geotextile. The calculations assumed that the height of the fill was small as compared to its width. The geometry considered in this study is a layer of soil located between two nonwoven geotextile layers as shown in Figure 15. One–dimensional consolidation was assumed, and the system of partial differential equations that models the problem was solved analytically.The calculated rates of consolidation were considered assuming two different cases: constant thickness of the draining geotextile; and taking into account the geotextile compressibility. Numerical examples showed that a typical nonwoven geotextile (with a transmissivity of 6.6 x 10^{-6} m^2/s) placed with 2 m vertical spacing would work as a perfect drain in a clay backfill, but would not prevent the generation of positive pore pressures in a silt backfill.

A similar approach, but without the simplifying one-dimensional consolidation assumption, was presented by Bourdillon et al. (1977). The problem was formulated considering two–dimensional consolidation, and solved using finite differences. Parametric studies were carried out by varying the soil layer thickness, drain thickness, embankment width, loading method, and soil and drain hydraulic conductivities. An example of the analysis results is presented in Figure 16, where the rate of pore water pressure dissipation obtained using perfect drains is compared to the rate obtained using geotextiles with non-ideal draining capabilities in a silty backfill. A parametric

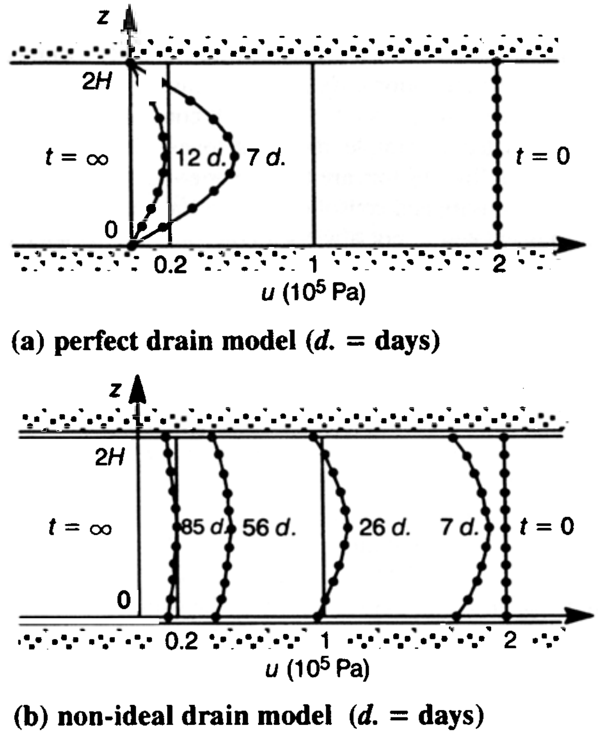


Figure 16. Diagrams of pore water pressure dissipation determined numerically for a silty backfill (after Bourdillon et al. 1977).

analysis was made to investigate the time required to dissipate 90% of the pore water pressure generated in the center of a soil layer. The minimum drain thickness and the ratio between the hydraulic conductivities of the drain and the soil were found to be critical parameters. Other variables investigated in the parametric study, such as the geotextile vertical spacing and the embankment length, were reported to have comparatively less influence on the results. The parametric study showed that a nonwoven geotextile over 2 mm thick will provide perfect drainage if its hydraulic conductivity is 10^4 to 10^5 times higher than the hydraulic conductivity of the soil.

A homogenization method, consisting of replacing the soil-geotextile system by an anisotropic equivalent material, was proposed by Auriault et al. (1982). This approach was considered to be more attractive for practical purposes, since it does not require numerical or analytic treatment as in the method presented by Auriault et al. (1977). Rapid evaluation of the efficiency of the geotextile layers in the embankment consolidation can be made using nondimensional charts. To validate the proposed homogenization simplification, the obtained results were compared to those obtained using previously proposed analytic methods. The cases for which the homogenization simplification is valid were established in terms of nondimensional parameters.

Giroud (1983) investigated the parameters governing the selection of a geotextile to be used as a horizontal drainage layer. The theoretical study assumed purely vertical flow (one-dimensional consolidation). As a design requirement, it was considered that the water pressure in the geotextile should be small compared to the applied pressure. Additional assumptions led to simple formulas to estimate the required geotextile transmissivity. The controlling factors are the rate of construction, the geotextile width, and the hydraulic conductivity and consolidation coefficients of the soil. The thickness of the soil being consolidated is not always a controlling parameter in the geotextile selection since, for the case of clayey soils, the geotextile spacing governs the consolidation rate but it has no influence on the required geotextile transmissivity. Based on the proposed formulation, it was concluded that the required transmissivity of a geotextile is higher if the soil to be consolidated is a silt rather than a clay. The reason for this result is that water is expelled faster from a consolidating silt than from a clay fill. These conclusions are consistent with the findings of Auriault et al. (1977) and Bourdillon et al. (1977). The criteria presented for selection of geotextiles to be used as draining layers can be easily implemented in practical design applications.

The hydraulic function of geotextiles used to reinforce saturated fine soils was studied by Blivet et al. (1986) using finite element calculations. Displacements, stresses and hydraulic heads were calculated as a function of time using a coupled elastic-plastic formulation. Stiffness and transmissivity of the geotextile layers are the required parameters for the reinforcement layers, modelled using one–dimensional elements. The dissipation, with the aid of geotextiles, of pore water pressures generated during an embankment construction was calculated. The lowest geotextile transmissivity that avoids generation of positive pore water pressure could also be determined.

The theoretical methods examined in this section were developed to evaluate the rate of settlement due to the fill consolidation rather than to investigate the structure stability. Nonetheless, the formulations can also be used to conservatively estimate the effect of pore water pressure dissipation on the structure stability as consolidation progresses.

5.3 Effect of Lateral Drainage on Stability

Some reinforced soil structures have already been constructed using poorly draining backfills reinforced using permeable inclusions (Mitchell and Zornberg 1994). The effect of lateral drainage on the stability of a reinforced clay structure is twofold: it will produce an increasing soil resistance to shearing along the failure surface; and will result in a pullout resistance that increases with time. However, most reported cases have not taken into account the time-dependent increase in stability provided by the permeable reinforcements. Generally, the analyses have either assumed drained behavior of the unsaturated backfill material or considered total stress soil parameters representative of placement conditions.

Few reported stability analyses take into account the increase in stability that occurs in a clay structure due to consolidation of the backfill. One such case is the analysis of a 32 m high embankment constructed using a specially devised technique, named the multiple strip-sandwich method, that used layers of quicklime and geotextile filter layers (Yamanouchi et al. 1982). Although the stability analysis did not incorporate any

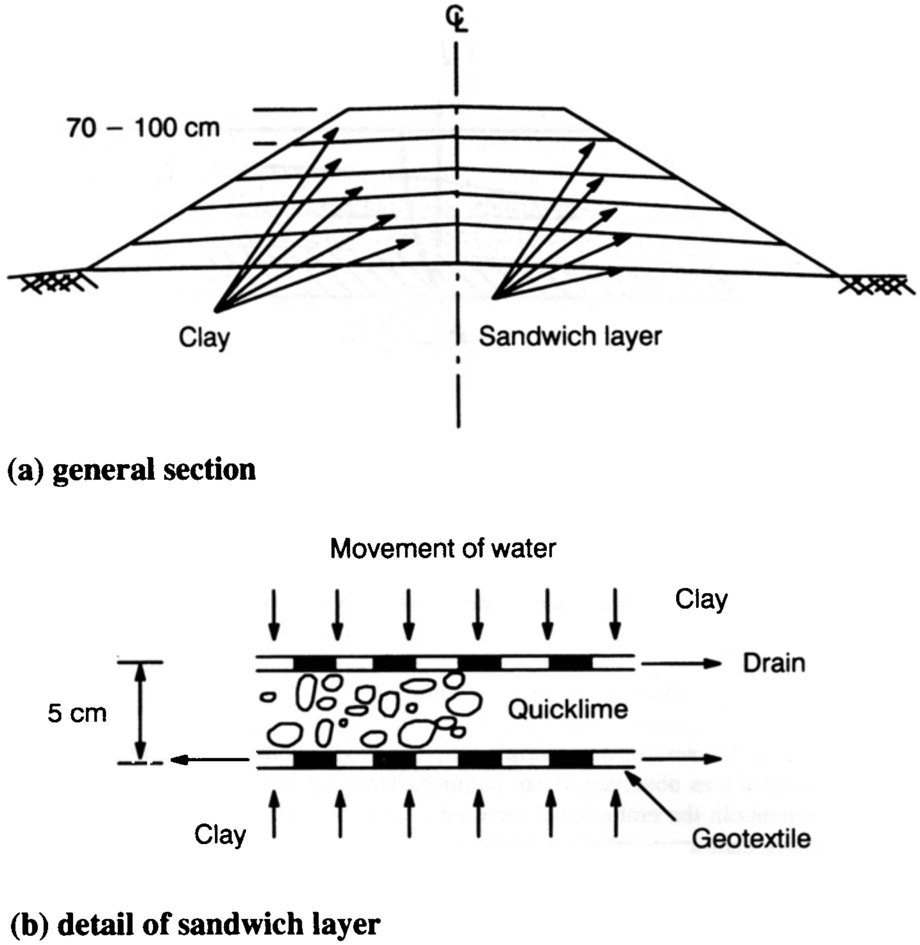


Figure 17. Multiple strip-sandwich method using geotextile-quicklime composite stripes in a clay embankment (after Yamanouchi et al. 1982).

tensile resistance for the filter layers, the strength of the cohesive soil was considered to increase during consolidation. Quicklime layers 50 mm thick, sandwiched between two sheets of geotextile, were placed in horizontal layers within the cohesive fill (Figure 17). Owing to combined actions resulting from the hydration of quicklime, particularly the absorption of water, the cohesive soil was effectively dewatered. The shear strength of the strongly weathered tuff used as backfill increased due to consolidation. Stability was calculated using the method of slices assuming circular failure surfaces, and taking into account the increase in undrained shear strength of the cohesive soil with time as consolidation progresses. The time-dependent strength gain was estimated using the average degree of consolidation formulated for drain wells. After completing the embankment work, borings were carried out on the embankment to

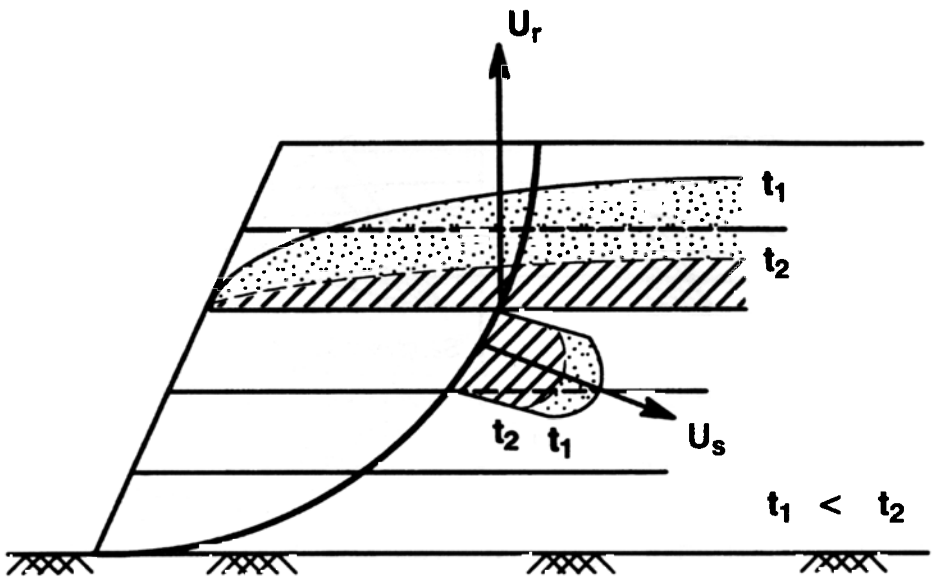


Figure 18. Effect of pore water pressures on the stability of a reinforced soil structure.

ascertain the effectiveness of the multiple strip-sandwich method. Good improvement of the fill material was observed. Also, inclinometer measurements indicated no horizontal movements in the embankment even during unusually heavy rains.

The strength gain in a cohesive backfill material due to consolidation was also reported by Yunoki and Nagao (1988) for a 20 m high fill slope built in Japan using cohesive soil. Nonwoven geotextiles were used to accelerate consolidation of the fill material and to reinforce the fill slope. The authors employed the method of slices on circular failure surfaces to analyze the stability of the slope. The analysis considered, in a simplified way, the strength gain of the backfill material due to the consolidation process. The shear strength on the base of each slice was calculated as a function of a given degree of consolidation. The formulation used to define the degree of consolidation was not reported.

Limit equilibrium methods have been commonly used in the analysis of reinforced soil slopes composed of free draining granular material. These methods are techniques for conventional slope stability analysis, adapted to take into account the stabilizing moment created by the reinforcements. In structures reinforced with permeable inclusions that used cohesive backfill placed at high water content, the generation and time-dependent dissipation of pore water pressures should be considered in the design. In this case, the permeable reinforcements work not only as reinforcements but also as drains, dissipating excess pore water pressures and enhancing stability. The lateral drainage provided by this system will enhance the structure stability by increasing both the soil shear resistance along the potential failure surface, and the pullout resistance along the soil-geosynthetic interface.

Figure 18 shows a possible pore water pressure distribution, at two different times after construction of a reinforced slope, along a segment of the shear surface and along one of the reinforcement layers. The dissipation of excess pore water pressures (u_s in the figure) causes an increase in effective stresses along the potential shear surface that results in higher resisting shear stresses and, consequently, a higher factor of safety with time. Pore water pressure dissipation along the reinforcement (u_r in the figure) will increase the effective stress along the geotextile anchorage length, increasing the pullout resistance. Moreover, as described in the following section, the increasing effective stresses along the geotextile will result in improved reinforcement mechanical properties, particularly for the case of nonwoven geotextiles.

6 ADDITIONAL CONSIDERATIONS FOR GEOSYNTHETICS EMBEDDED IN POORLY DRAINING BACKFILLS

For the safe design of a reinforced soil structure with poorly draining backfill, it is necessary to evaluate the influence of these backfills on geosynthetic mechanical properties, durability, and creep characteristics.

6.1 Confined Mechanical Properties

Correct determination of the modulus and tensile strength of geosynthetics is of fundamental importance for the design of reinforced structures. The geosynthetic mechanical properties should be measured in a manner that simulates the field conditions. This is not the case for some methods of testing, such as the grab tensile test and the wide-width test, which are commonly used in the textile industry. Several investigators have already focused on the tensile characteristics of geosynthetics under the soil-confinement condition. Among them, McGown et al. (1982) found that the mechanical properties of nonwoven and composite geotextiles significantly improve when tested under sand confinement. Christopher et al. (1986) developed a zero span test which mechanically models the confinement provided by granular soil. This test, while quick and simple to perform, yielded stress-strain information which compared favorably with the in-soil results obtained by McGown et al. (1982). Ling et al. (1992) developed a test apparatus for measuring the load-deformation properties of geotextiles under in-air, in-membrane, and in-soil conditions. They found that using a membrane for confinement of the geotextile specimen is as effective as using soil confinement, and concluded that this test is a superior alternative for determining the load-deformation properties of geotextiles under typical operational conditions.

In-soil tensile tests have also been performed under confinement by cohesive soils. Fabian and Fourie (1988) carried out tensile tests on woven and nonwoven needle-punched geotextile specimens confined in clay. They found that the geotextile modulus greatly increased due to the confinement. Due to the different mechanism of the clay-geotextile interaction, the modulus increase for the nonwoven geotextile was considerably larger (up to ten times) than for the woven geotextile (up to three times). Additional tensile tests under confinement by fine soils were performed by Chang et al. (1993). Although differing in the testing methodology, all previous studies show

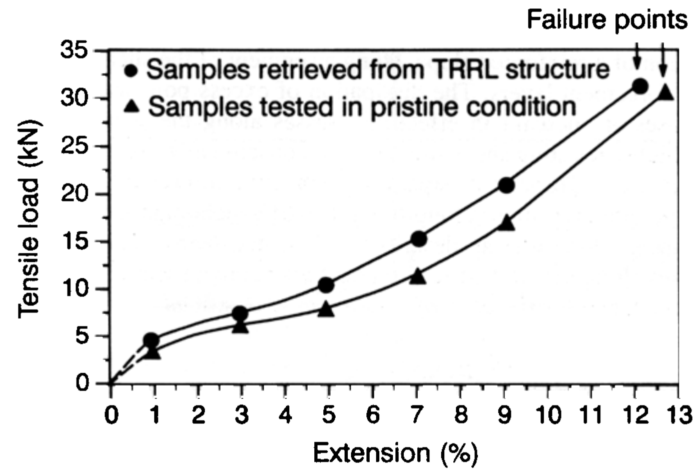


Figure 19. Load-extension characteristics of plastic strips tested after eight years of burial in cohesive soil (after Temporal et al. 1989).

that there is a significant increase in stiffness and strength of geotextiles under soil confinement, in comparison to values obtained in unconfined conditions. The tests results show that improvement in geotextile mechanical properties occurs under confinement by both granular and cohesive soils.

6.2 Durability

The design of geosynthetic reinforced structures must ensure that long-term stresses in the reinforcement do not exceed the strength of the reinforcement at any time during the design life of the structure. The service life of a reinforced soil structure depends on the durability of the reinforcing elements. Although not susceptible to corrosion, polymeric reinforcements may degrade by a number of different actions. These include: ultraviolet light; high energy radiation; oxidation; hydrolysis; and some chemical reactions (Allen 1991; Koerner et al. 1992). In addition, they are susceptible to construction damage. The issue of long-term geosynthetic strength is currently the subject of continuous research, and the experience already gained is generally based on case histories of structures built using granular backfill. However, reported cases in which geosynthetics have been used in cohesive fills have indicated satisfactory long-term performance.

Reinforcement samples were retrieved from the Transport and Road Research Laboratory experimental structure, a full-scale embankment built with cohesive fill. These samples have been used for evaluation of the long-term durability of the reinforcements (Temporal et al. 1989). Preliminary data have been reported for plastic strips and, as shown in Figure 19, no loss of strength had occurred in the first eight years of burial. However, some increase in stiffness may have developed.

A geotextile reinforced embankment was built for test purposes in 1981 and had already been exposed to three years of extreme climatic fluctuations and environmental influences by the time it was loaded in early 1984 (Werner and Resl 1986). A nonwoven needle-punched polypropylene geotextile was used as reinforcement, and the backfill material was a silty sand. After loading, nonwoven samples were taken from the interior and surface of the embankment. Tensile tests were performed on the nonwoven polypropylene samples retrieved from the embankment to determine any strength losses or material changes, showing no changes from the original mechanical characteristics. For the case of nonwoven polyesters, Colin et al. (1986) reported no significant degradation for geotextiles buried for seven years in moist organic rich soil.

The performance of two reinforced slopes, the M4 at Yattendon and the A45 Cambridge Northern bypass, were described by O'Reilly et al. (1990). The M4 Yattendon cutting was constructed with about 1% quicklime added to the excavated clay material and reinforced with layers of high density polyethylene mesh. The A45 Cambridge Northern bypass was constructed of Gault clay using polypropylene geogrid reinforcements. Both slopes and geotextile reinforcements have performed well over periods of nine and six years respectively. Also, samples of polymer reinforcement from both sites were recovered and have been tested. No significant degradation of either the mesh or the geogrid has apparently occurred, although the materials recovered from the field probably had been subjected to site damage and ultra-violet weathering prior to installation as well as aging in-situ.

6.3 Creep

Creep is the response of the reinforcement to sustained load, resulting in time-dependent deformations that may continue as long as the reinforcement is loaded. Early studies of tensile creep behavior of geotextiles found that geotextiles in unconfined tests exhibit instantaneous recoverable primary creep, long-term nonrecoverable secondary creep and tertiary creep to rupture. Although there is concern of higher creep potential for the case of structures built with poorly draining backfills, geosynthetic creep response observed in reported case histories has been encouraging.

Soil creep has been a concern for the use of marginal backfills using metallic reinforcement systems. Elias (1979) carried out creep pullout tests on ribbed steel strips buried in a wide variety of cohesive soils at optimum moisture content and in the range of 95% of the maximum density. During conventional pullout tests, strips were subjected to creep testing using a constant pullout force, 50 to 60% of the ultimate pullout load, acting for 175 hours. The test results indicated little or no tendency to creep for any of the typical soils involved in the program. Surprisingly, even kaolin clay did not exhibit creep.

Soil confinement has been found to substantially reduce the magnitude of geotextile macrostructure creep. This is because confinement tends to restrict movement of individual filaments preventing their realignment in the direction of the load. McGown et al. (1982) found the reduction in primary creep to be especially large (on the order of 40 to 60%) for nonwoven geotextiles confined in sand. They also found that the secondary creep rate is substantially reduced due to soil confinement. It was concluded that unconfined creep testing grossly overestimates the long-term creep deformations that would occur under soil confinement. The reduced geotextile creep deformations could explain the small long-term deformations observed on actual geotextile-reinforced walls.

The effect of cohesive soil confinement on the creep response of geotextiles was investigated by Fourie and Fabian (1987) by performing drained pullout tests on geotextiles in a silty clay. Drained pullout tests are not only long-term interaction tests, but long-term geotextile tensile tests as well. The pullout resistance in drained conditions was influenced by the time-dependent rearrangement of polymer molecules at a constant strain less than failure strain. Although relaxation reduced the geotextile tensile strength, producing a lower ultimate pullout resistance in the drained condition than in the undrained one, the mechanism of drained resistance developed at the clay-geotextile interface was considered not to differ from that at sand-geotextile interfaces. Christopher and Berg (1990) performed pullout tests on geogrids over extended periods of time using cohesive soils. Differences in the displacement measurements obtained at the front and back of the geosynthetic sample, as shown in Figure 8, were interpreted after evaluating the creep response of geogrid during testing. It was concluded that stress dissipation along the length of the sample, and not creep deformations, was the main cause of differences in the monitored movements of the front and rear gages of the sample.

Field monitoring of creep deformations in reinforced soil structures with granular fill has shown that deformations predicted based on laboratory tests of unconfined geotextiles have not occurred. This was the case for several portions of the geotextile-reinforced wall built at Glenwood Canyon in Colorado, that were expected to experience substantial creep deformations. However, Bell et al. (1983) reported that no significant creep deformations were measured, even though many of the wall segments had factors of safety with respect to creep rupture much less than 1.0. Acceptable creep performance may also be expected for the case of structures with marginal backfills since, as for the case of granular backfills, geotextile mechanical properties are improved under the confinement of cohesive soils. An example is the acceptable long-term performance reported for two geotextile-reinforced soil structures built with cohesive backfill and reinforced with nonwoven geotextile layers (Test Embankments I and II reported by Tatsuoka and Yamauchi 1986). Although the creep potential of nonwoven geotextiles is larger than for other kinds of geotextiles, the horizontal creep deformation of the two test embankments was found to be slight except in the first year. The creep strain rate of the nonwoven geotextile at the measurement location decreased from about $3\text{x}10^{-3}$%/day, one week after construction, to $3\text{x}10^{-5}$%/day 200 days after construction.

7 CONCLUSIONS

The requirement of granular backfill has been a major limitation to the selection of reinforced soil for many retaining wall and embankment projects. The purpose of this and a companion paper (Mitchell and Zornberg 1994) is to provide the results of a review and evaluation of published material related to the suitability of poorly draining soils for reinforced soil structures.

Although the interaction mechanisms between poorly draining soils and metallic and polymeric reinforcements could not be clearly elucidated, triaxial test results have shown that poorly draining soils can be reinforced with properly selected permeable geosynthetics. The bond strength between the permeable reinforcement and the soil can be higher than the undrained soil strength if the geosynthetic transmissivity is high enough to drain the soil-reinforcement interface.

The results of shearbox and pullout tests using metallic reinforcements, geogrids, geotextiles, and geomembranes with poorly draining backfill soils are consistent with those from triaxial tests, showing that soil-reinforcement contact efficiencies are higher with permeable geosynthetics than with impermeable reinforcements. Consequently, in addition to the required tensile strength, geosynthetics in poorly draining backfills should also have adequate drainage capabilities.

If permeable geosynthetics are used to reinforce poorly draining backfills, the geosynthetic layers can function not only as reinforcements but also as lateral drains. The hydraulic function of reinforcements should be incorporated in a rational design approach that takes into account the geosynthetic transmissivity. Laboratory procedures have already been developed to determine the reinforcement drainage capacity under operational conditions, and theoretical methods have been proposed to evaluate the dissipation of pore water pressures generated during construction using a saturated backfill. The challenge now is to account for the hydraulic function of reinforcements in the structure design and to validate the design assumptions against field monitoring results.

There is already experimental evidence that, as for the case of granular soils, the mechanical properties of geotextiles are improved under the confinement of cohesive soils. Moreover, although the long-term performance of geosynthetics embedded in marginal soils has been of major concern, experimental results so far have shown adequate durability and creep characteristics for the tested geotextile samples.

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Table 2. Triaxial tests on reinforced samples using poorly draining soils.

Research agency	Test type	Soil	Reinforcement	Test variables	Observed behavior	Conclusions	Reference
Ground Engineering Ltd, U.K.	Undrained triaxial	Saturated kaolin clay; London clay	Aluminum foil; porous plastic	Reinforcement spacing was varied	Aluminum foil reduces the sample strength. Permeable plastic reinforcements increased compressive strength in the clay sample	Permeable plastic performs better than impermeable foil. Reduction in strength is due to generation of pore water pressures	Ingold 1979; Ingold & Miller 1982
Ground Engineering Ltd, U.K.	Drained triaxial	Normally consolidated kaolin clay	Porous plastic	Tests on multi-reinforced samples and on unit cells of clay	Decreasing the reinforcement spacing increased both the drained shear strength and the secant deformation modulus	Strength enhance-ment was attributed to radial strain control mobilized on soil-reinforcement interface	Ingold & Miller 1983
Laing Design Develop. Centre, U.K.	Rapid triaxial tests on unsaturated samples	Kaolin clay	Aluminum foil	Degrees of saturation ranged from 75% to 100%	A linear relationship between strength and degree of saturation, apparently independent of cell pressure, was observed	Conditions very close to drained situations might be expected to prevail for low degrees of saturation	Ingold, 1985
Queensland Inst. of Technology, Australia	UU and CU triaxial tests	Silty clay (PL:14% ; LL:27%)	Geotextiles and geogrids	Eleven different types of geosynthetics were used	Geotextile reinforcement restricted the lateral deformations in the sample. Stress-strain curve of the reinforced sample differs from that of the unreinforced material	Permeable geo-textiles increase sample strength. Adhesion factor between clay and reinforcement improves with increasing moisture content	Fabian & Fourie 1986

Table 2. Continued

Scientific Research C., Baghdad	CU triaxial tests	Kaolin clay	Geogrid	Samples had overconsolidation ratio of 3	Enhancement in undrained strength was obtained	Strength improvement was attributed to openings in the geogrid, that cause interlocking	Al-Omari et al. 1987
Scientific Research C., Baghdad	CD and CU triaxial tests	Powdered kaolin clay	Geogrid	Geogrid stiffness, reinforcement spacing, and confining pressure were changed	Geogrid reinforcement enhanced strength in both undrained and drained conditions. The A pore pressure parameter was higher for reinforced than for unreinforced samples	Reinforcement effect in undrained condition is an increase in cohesion. The effect in drained condition is an increase in internal friction	Al-Omari et al. 1989
AIT, Bangkok	UU triaxial, consolidation tests	Soft silty clay	Nonwoven and woven geotextiles	Compactability, strength, and consolidation were investigated	Nonwoven geotextile improved the compactability of clay samples	Nonwoven geotextile performed better than woven geotextile because of its better drainage, particularly at high moisture content	Indraratna et al. 1991
U. of Tokyo, Japan	Drained and undrained plane strain tests	Nearly saturated silty clay	Nonwoven geotextiles	Tests done on isotropically and anisotropically consolidated samples	Improvement in strength and stiffness was more significant for the anisotropically consolidated specimens	Drained tests show greater reinforcement effect than undrained tests.	Ling & Tatsuoka 1993

Note: UU: unconsolidated undrained; CU: consolidated undrained; CD: consolidated drained

Table 3. Shearbox and pullout tests using poorly draining soils.

Research agency	Test type	Soil	Reinforcement	Characteristics	Observed behavior	Conclusions	Reference
Reinforced Earth Co.	Pullout and creep tests	Low plastic silts and clay	Ribbed steel strips	Deformation rate was 2.5 mm/min	Peak shear stresses along the strip were considerably less than for granular soils	Additional work is necessary prior to use of fine-grained soil in Reinforced Earth fill	Elias 1979
LRPC, Rouen, France	Direct shear	Phospho-gypsum	Woven and nonwoven	Material placed at optimum moisture content	Coefficient of friction decreased with increasing confining stresses	Friction coefficient was in excess of 0.4	Blivet & Gestin 1979
Ground Eng. Ltd, U.K.	Direct shear and pullout tests	Kaolin clay	Polyethylene mesh; metallic reinforcement	Unconsolidated-undrained tests	Inclined reinforcements on clay samples subject to undrained direct shear caused apparent increase in shear strength	Enhanced strength may be induced by resistance of reinforcement to rotation rather than pullout	Ingold 1981; Ingold 1983
Cambridge Univ., U.K.	Undrained direct shear tests	Lightly over-consolidated kaolin clay	Geogrid	Geogrid types, dimensions, and orientation were changed	Short- and long-term shear strength of cohesive soil was increased by reinforcement	Shear strength of reinforced soil may be calculated using a limit equilibrium analysis	Jewell & Jones 1981
Reinforced Earth Co.	Pullout tests	Residual low-plastic silts	Ribbed steel strips	Soils with different % of fines were tested	Apparent friction coefficient decreased with fines content. Samples compacted wet of optimum showed reduced pullout capacity	Fines content and moisture content should be carefully evaluated in structures reinforced with steel strips	Elias & Swanson 1983
Illinois Inst. of Technology	Direct shear tests	Sandy clay	Woven and nonwoven geotextiles	Shear box was placed into saturation tank	Peak shear strength was reached at relatively large displacements. Residual strength did not vary form peak strength	Soil-geotextile interface strength for woven geotextile was lower than for nonwoven fabric	Saxena & Budiman 1985

Table 3. Continued

Drexel Univ., Philadelphia	Direct shear tests	Cohesive soils	Five geomembranes	Geomembrane was placed in lower box portion	Adhesion of soil to geomembrane was smaller than the soil cohesion. Friction angles at geomembrane interfaces were relatively high	Geomembranes placed directly on clay should have low slopes (e.g. 4H to 1V)	Koerner et al. 1986
Arizona State Univ.	Pullout and creep tests	Wyoming bentonite	Polymeric geogrids	Expansive clay was placed at low moisture content	Pullout force versus normal force was linear	Pullout resistance of geogrids in low moisture content clays is rather insensitive to geogrid strength or configuration	Brand & Duffy 1987
Ecole Polytechnique de Montreal	Direct shear tests	Plastic clay (PI: 30%)	Woven and nonwoven geotextiles	Clay samples were molded at different water contents	Contact efficiency for nonwoven geotextiles exceeded 100%, and for wovens decreased to 60%	The better frictional characteristic of nonwoven geotextile were attributed to the presence of randomly oriented fibers	Lafleur et al. 1987
Georgia Inst. of Technology	Direct shear tests	Gulf coast clay; glacial till	Nonwovens wovens, PVC, HDPE	A large shear box was used	For geosynthetics with moderately rough surfaces, the sliding did not occur along the interface, but within the soil	Adhesion is primarily a function of soil type, moisture content, and the surface roughness of the geosynthetic	Williams & Houlihan 1987
Univ. of Queensland, Australia	Shearbox and pullout	Silty clay (PL: 14%; LL: 27%)	Wovens, nonwoven, geogrids	Drained and undrained tests	Shear strength of clay was increased by geotextile reinforcement in both undrained and drained loading	For high transmissivity geotextiles or for geogrids, pullout resistance is limited by the tensile strength and relaxation of the material	Fourie & Fabian 1987

Lakehead U., Canada	Direct shear	Plastic and non-plastic silty clays	Woven and nonwoven geotextiles	Samples were saturated for testing	Efficiencies were approximately 60% for woven geotextiles and 70% for nonwoven geotextiles	Direct shear tests are appropriate to determine shear strength values at soil-geosynthetic interfaces for low stress level applications	Eigenbrod & Locker 1987
AIT, Bangkok	Direct shear and pullout	Clayey sand, weathered clay	Polymer geogrid, bamboo	Samples were tested at optimum moisture	Bamboo grids showed higher pullout resistance than the polymer geogrids	Reinforcement interaction results from soil reinforcement adhesion and from bearing capacity by the geogrid transverse members	Bergado et al. 1987
Univ. of Queensland, Australia	Pullout	Silty clay. Saturation of 95%	Woven and nonwoven geotextiles	Geotextile n-soil tensile strength was determined	Geotextile modulus greatly increased due to confinement. This increase was considerably larger for nonwoven geotextiles	Soil confinement increases the tensile modulus of geotextiles due to improved interfiber friction and interlocking	Fabian & Fourie 1988
Nihon University, Japan	Direct shear	Volcanic ash clay (Kanto loam)	Woven and nonwoven geotextiles	Soil at optimum water content	No peak friction observed during shearing. Similar friction-displacement response observed for all geotextiles	Mobilized interface friction is generally a small portion of soil friction	Makiuchi & Miyamori 1988

Table 3. Continued

I.I.T., Madras, India	Direct shear	Kaolin, silty clay	Woven geotextile	Samples were tested dry and wet of optimum	Molding water content significantly affected the strength of samples	At low water content, geotextile does not affect kaolin samples, but produced loss in strength in silty clay	Krishnaswamy & Raghavendra 1988
Univ. of Alberta, Canada	Direct shear tests	Silty clay	Four geogrids, one woven geotextile	Samples were compacted dry of optimum	Reinforced and unreinforced clay exhibited similar stress-deformation relationships	Construction method and geometry of reinforcement strongly influence the shear strength behavior	Richards et al. 1989
STS Consultants Ltd.	Pullout tests	Four cohesive soils	geotextiles; geogrids	Tests performed under drained and undrained conditions	Differences in as-placed water content resulted in significant variations in undrained strength.	Current evaluations of pullout resistance are generally conservative	Christopher & Berg 1990
Warsaw Agricultural Univ.	Direct shear and pullout tests	Organic mud soils (silty clay)	Nonwoven geotextiles	Pullout tests performed in modified triaxial apparatus	Pullout tests gave frictional resistance 30% smaller than obtained from direct shear tests	Pullout and direct shear test results showed linear relationship between shear and normal stress	Garbulewski 1990
Univ. of California, Berkeley	Direct shear and pullout tests	Compacted clay	HDPE, geotextile, geonet	Components of a clay liner system were investigated	Minimum residual frictional resistance was fully mobilized at small deformations	Critical interfaces were those between HDPE and geotextile, HDPE and geonet, and HDPE and saturated compacted clay	Mitchell et al. 1990

Univ. of California, Berkeley	Direct shear tests	Compacted clay (soil-bentonite)	Smooth HDPE geomembranes	As-compacted and unconsolidated undrained tests	Contours of strength values could be drawn in zones roughly parallel to the zero air voids curve	As compacted HDPE-clay interface friction angles can change by factor of two as a result of minor variations in compaction conditions content	Seed & Boulanger 1991
Scientific Research Council, Baghdad	Pullout tests; swelling tests in oedometer apparatus	Mixtures of kaolinite and bentonite	Nonwoven polymer geogrids	Geogrid stiffness, soil plasticity index, and applied surcharge were varied	Reinforcements reduced both the final swell, and the rate of swell	Reduction in swell due to reinforcement increases with increasing stiffness of the geogrid	Al-Omari & Hamodi 1991
Indian Inst. of Science, Banglore	Pullout tests	Sawdust and Kaolin clay	Steel bars and flats	Thin zone of sand was placed around the reinforcement	Using fine-grained backfill soil with 15-mm of sand around the reinforcement increased the interface strength to that obtained using only sand as bulk material	The required thickness of sand layer depends on the surface roughness of the reinforcement and the strength of the fine-grained medium	Sridharan et al. 1991
AIT, Bangkok	Pullout tests	Clayey sand, lateritic soil, weathered clay	Steel grid	Large pullout box was used	Pullout resistance increased with increasing overburden pressure (soil compacted dry of optimum)	Laboratory pullout tests were a conservative approximation of the field pullout resistance	Bergado et al. 1992a

Table 3. Continued

AIT, Bangkok	Pullout tests	Weathered Bangkok clay	Steel grid	Soil compacted both dry and wet of optimum	Pullout resistance was much higher for backfill compacted dry of optimum	As-compacted moisture content has a significant effect on pullout resistance	Bergado et al. 1992b
Korea Institute of Construction Technology	Pullout tests	Weathered granite (CL)	Protruded members attached to reinforcements	Soils tested as compacted	Coefficient of passive resistance decreased with increasing width and/or length of reinforcements	Results are influenced by the three dimensional nature of shear failure surface and by the progressive nature of the pullout failure	Sohn et al. 1992
Univ. Fed. de Ouro Preto, Brazil	Shearbox tests	Kaolin clay	Woven and nonwoven geotextiles	Large direct shearbox was used	Clay-nonwoven interface friction was higher than soil friction	Clay-geotextile interaction mechanisms are similar to those obtained using sand backfill	Gomes 1992
WES, Corps of Engineers	Pullout and shearbox tests	Clay, silty clay, silty sand	Woven geotextiles	Tests were performed at various normal stresses and water contents	Increase of pullout rate caused increase in apparent pullout resistance. Water submergence decreased the apparent pullout resistance	Increase of normal stress causes increase in pullout resistance only in unsaturated samples	Gilbert et al. 1992
AIT, Bangkok	Large scale pullout and direct shear	Weathered Bangkok clay	Steel, bamboo, polymeric geogrids	Samples were tested dry of optimum water content	During pullout tests, steel and bamboo grids moved as rigid body, while polymer grids elongated along the reinforcement length	For steel grids, most pullout resistance were from the passive-bearing resistance of transverse members. Opposite results were obtained for bamboo grid and polymer geogrids	Bergado et al. 1993