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

Reinforced Soil Structures with Poorly Draining Backfills Part II: Case Histories and Applications

ABSTRACT: Experimental studies on poorly draining soil-reinforcement interactions were reviewed in a companion paper by Zornberg and Mitchell in 1994, leading to the conclusion that permeable geosynthetic inclusions are useful for reinforcing marginal backfills. This conclusion is strengthened by lessons learned from the case histories described in this paper. There are no design guidelines for reinforced soil structures using poorly draining backfills. Nevertheless, several of these structures have already been constructed, and the performance of some of them has been reported. Good structure performance is strongly dependent on maintaining a low water content in the poorly draining fill. Large movements occurred in reinforced structures when pore water pressures were generated, and failures were reported in marginal backfills reinforced with impermeable inclusions that became saturated after rainfalls. Benefits and applications of reinforcing poorly draining backfills are addressed, and research needs aimed at formulating a consistent design methodology for these structures are presented.

KEYWORDS: Soil Reinforcement, Marginal Backfill, Cohesive Backfill, Case Histories, Pore Water Pressures, Modes of Failure, Structure Deformations, Research Needs.

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1 INTRODUCTION

Soil reinforcement is a highly attractive alternative for embankment and retaining wall projects because of the economic benefits it offers in relation to conventional retaining structures. The rapid acceptance of soil reinforcement can be attributed to a number of factors, including low cost, aesthetics, reliability, simple construction techniques, and the ability of the reinforced soil structures to adapt to different site conditions. However, these economic benefits have often been limited by the availability of good-quality granular material, which has generally been specified for the backfill. Undoubtedly, substantial cost savings and new soil reinforcement applications would result if fine grained cohesive soils as well as industrial and mine waste materials could be used in reinforced soil construction.

Interestingly, however, the first geotextile-reinforced wall ever constructed used poorly draining cohesive soil as backfill material. The purposes of this first geotextile-reinforced structure, built in 1971 by the French Highway Administration in Rouen, were to test its stability and to verify the magnitude of deformations caused by the soil-geotextile interaction (Puig and Blivet 1973; Puig et al. 1977). The first geotextile-reinforced wall in the United States was built by the U. S. Forest Service in 1974 (Bell and Steward 1977). This wall used on-site silty sand for the backfill, and was built to reconstruct a road fill above the Illinois River in Oregon. The construction of reinforced soil structures using poorly draining backfill has been largely restricted, however, to early applications of soil reinforcement. This is probably a consequence of strong recommendations by various design agencies against the use of low-quality backfill for permanent structures.

Experimental research done to investigate the interaction mechanisms between reinforcements and poorly draining soils was reviewed in a companion paper (Zornberg and Mitchell 1994). Both this and the companion paper are condensed and updated from a more comprehensive report by Zornberg and Mitchell (1992). Although reported experimental results have led to contradictory conclusions on the effects of impermeable reinforcement layers, there is already strong experimental evidence that permeable inclusions can effectively reinforce poorly draining backfills. There is no general design methodology for reinforced soil structures built with cohesive backfills. Nevertheless, since a number of these types of reinforced structures has already been constructed, many lessons can be learned from past experience. The purpose of the present paper is to complete the assessment on the use of marginal soils by evaluating the performance of structures reported in case histories.

2 LESSONS LEARNED FROM CASE HISTORIES

2.1 General Considerations

Several aspects of the performance of those reinforced marginal soil structures for which data are available are reviewed individually in this section, including generation of pore water pressures in the fill, possible modes and causes of failure, and structure deformability.

Reduced-scale models of reinforced soil structures have been built to help define the mechanisms of soil-reinforcement interaction. Behavior and conclusions drawn from the performance of models that used poorly draining soils as backfill material are summarized in Table 1. Additionally, several full-scale mechanically stabilized structures have been built using low-quality backfills, and the performance of these structures is noted in Table 2. Full-scale experimental reinforced soil structures proved to be unique sources of information. Although generally built with a more limited instrumentation, the performance of actual (nonexperimental) reinforced clay structures also supplied valuable information. Complete details about each of the cases summarized in these tables may be found in the indicated references.

Relatively few of the reported small-scale models and full-scale structures contained metallic reinforcements (e.g. Elias and Swanson 1983; Hannon and Forsyth 1984; Bergado et al. 1991). This may be a consequence of concerns about corrosion and pore water pressure generation. Most of the reported case histories relied either on the high tensile strength offered by geogrids (e.g. Sego et al. 1990; O'Reilly et al. 1990; Burwash and Frost 1991; Hayden et al. 1991), or on the drainage capabilities of nonwoven geotextiles (e.g. Puig et al. 1977; Tatsuoka and Yamauchi 1986; Yunoki and Nagao 1988).

Silts or low plasticity clays were used as backfill material for many structures; e.g. Boden et al. (1978), Hannon and Forsyth (1984), Perrier et al. (1986), Sego et al. (1990), Burwash and Frost (1991). However, more difficult to compact plastic clays were used in some cases; e.g. Hashimoto (1979), Yamanouchi et al. (1982), Tatsuoka and Yamauchi (1986), Hayden et al. (1991). In a few cases, industrial or mine wastes were used as embankment fill (Jewell and Jones 1981).

Although there is usually a tendency to report only successful case histories, some unsuccessful cases are also described in the literature (Elias and Swanson 1983; Mitchell and Villet 1987; Burwash and Frost 1991; Huang 1992).

2.2 Pore Water Pressure Generation in Reinforced Fills

Only a small number of the reported case histories included monitoring of the generation and dissipation of pore water pressures in a cohesive backfill. Since many of these structures were constructed using unsaturated compacted clay, the fill material was often considered to have a drained behavior. Analytic prediction of the generation or dissipation of pore water pressures has generally not been done. Some theoretical methods have been proposed for the analysis of consolidation between horizontal geotextiles (Zornberg and Mitchell 1994). Although they assume full saturation in the fill, this conservative assumption could be eventually used to estimate the pore water pressure dissipation in a reinforced clay structure reinforced with permeable inclusions.

2.2.1 *Structures Reinforced Using Impermeable Elements*

To investigate the feasibility of using cohesive fills, a full-scale experimental reinforced wall was constructed by the Transport and Road Research Laboratory (TRRL), U.K. The construction and instrumentation are described by Boden et al. (1978), and the early performance by Murray and Boden (1979). This structure was a vertical sided 6 m high embankment, with three layers of different fill materials, each occupying about one-third of the height (Figure 1). A wet cohesive fill was placed at the lowest

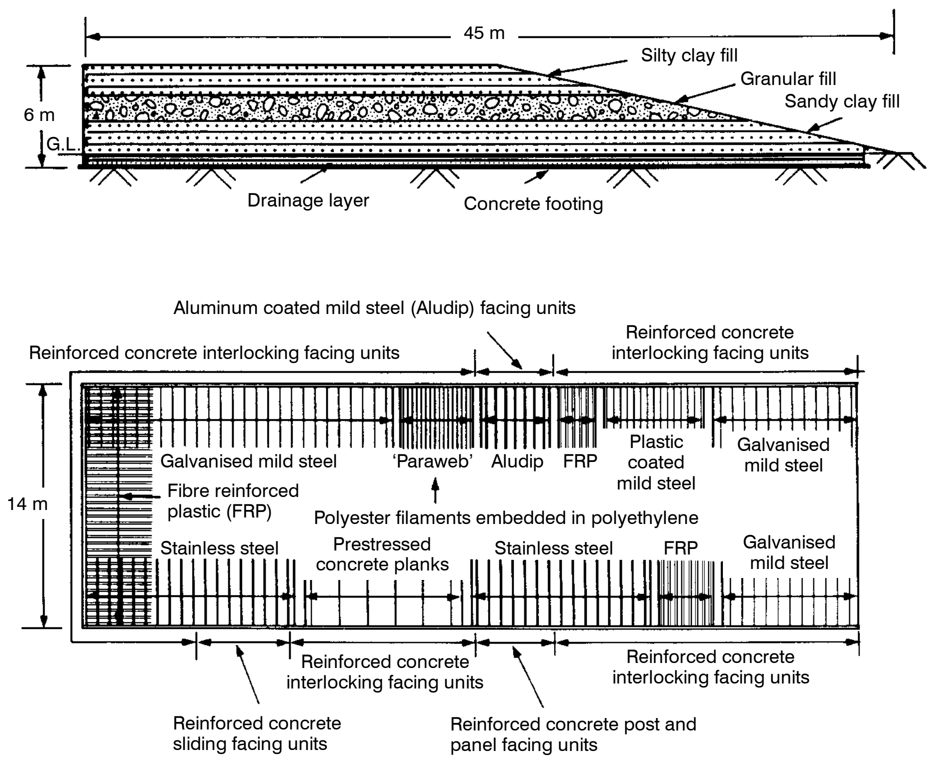

Figure 1. Transport and Road Research Laboratory (TRRL) experimental reinforced wall (after Boden et al. 1978).

level, granular fill was used for the central layer, and a cohesive fill at lower moisture content was placed in the upper part of the structure. A range of different types of impermeable reinforcing elements, basically plastic and steel strips were used. Pore water pressures were monitored during construction of the embankment. An indication of the relatively high excess pore water pressures generated in the lower clay layer can be observed in Figure 2, which shows the excess pore water pressure condition immediately after construction, and six months later at a distance of 3 m from the facing. Higher pore water pressures were measured at a location 5 m from the facing, and negligible pore water pressures were recorded at distances less than 1 m from the facing. Pore water dissipation was reported to agree well with that predicted using the coefficients of consolidation from laboratory tests. No preferential drainage along the reinforcements (plastic and metal strips) appears to have occurred.

Four half-scale embankments, including a control and three geogrid reinforced embankments, were constructed in stiff overconsolidated clay soils (London Clay) and loaded to failure (Irvin et al. 1990). The response of the embankments to vertical surcharge loading applied through hydraulic jacks was monitored by extensive instrumentation. Piezometers were installed to monitor the effect of geogrid layers on the distribu-

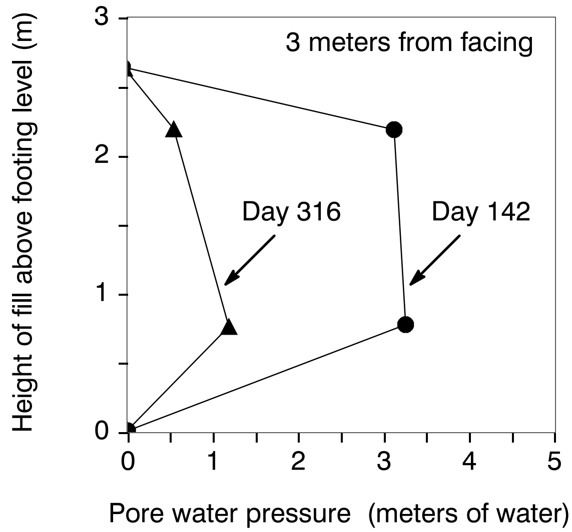

Figure 2. Vertical distribution of pore water pressure in the lower cohesive layer of the TRRL experimental wall (after Murray and Boden 1979).

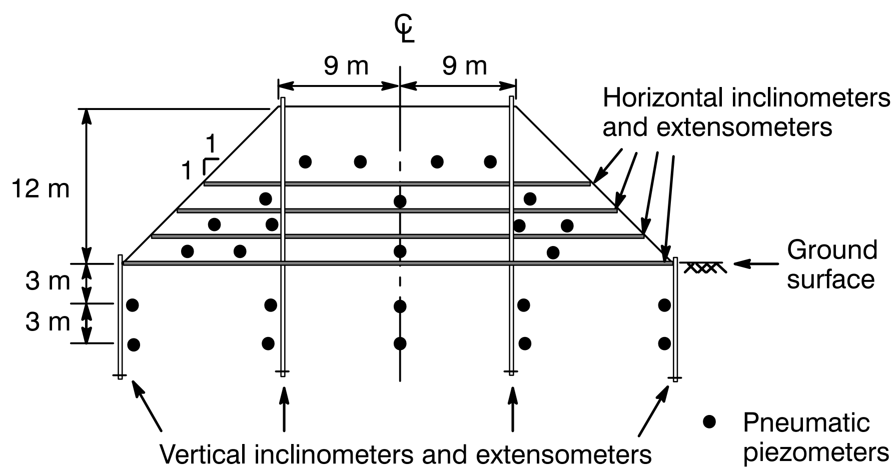

Figure 3. Instrumentation in Devon test fill (after Scott et al. 1987).

tion of pore water pressures, showing that changes in pore water pressures generally reflected the changes in applied load. Some piezometers in the upper part of the embankment showed increasingly negative pore water pressures as the load increased. It was suggested that dilation of the clay, associated with widespread shearing of the soil, may have occurred. The response of piezometers at the level of the geogrid layers and midway between them was similar, indicating that geogrid layers did not provide preferential drainage paths.

An insight into the interaction between pore water pressure generation, soil displacements, and geogrid strains may be gained from the analysis of the field measurements done at the Devon test fill. This test fill, built near Devon (Alberta, Canada), is a 12 m high test embankment with three sections reinforced with different geogrid materials and one unreinforced test section (Scott et al. 1987). The fill material is a silty clay that was compacted wet of optimum moisture content to ensure significant deformations and straining of the reinforcements.

The location of instruments installed within the embankment is shown in Figure 3. A series of field measurements has been reported for one of the test sections, showing the effect of pore water pressure on the deformations within the embankment (Sego et al. 1990). The reported field data are from instrumentation located in the fill, 3 m above the foundation level, where the second level of primary reinforcement was installed. Figure 4a shows the fill height versus time throughout the construction period. Inclement weather and short construction seasons resulted in a 26 month long fill construction period. Figures 4b and 4c present the horizontal and vertical displacement recorded at the 3 m level within the embankment and at various distances (2, 6, 10, and 14 m) behind the slope face. Pore water pressures measured 5 m from the slope face at the 3 m elevation (Figure 4d) increased in direct response to the loading during the fill placement periods.

Figure 4e illustrates the geogrid strains at various distances from the slope face, also 3 m above the base of the fill. The geogrid began to strain as the embankment underwent vertical and horizontal deformation during embankment construction. After the first 3 m of fill were placed above the geogrids, the reinforcement strains measured 5 m from the slope face were about 0.6%. Also, up to 20 and 15 mm of horizontal and vertical deformations occurred 3 m above the base during the same period, while the pore water pressures increased from 0 to 34 kPa. During the winter shut down (after day 430), significant settlements occurred as the pore water pressures dissipated from 34 to 10 kPa. Since the soil was becoming stronger as effective stresses increased, the geogrids were not required to carry much additional load, and the measured strains decreased slightly.

The placement of an additional 6 m of fill caused the geogrid strains and the horizontal and vertical displacements to increase, and pore water pressures within the fill increased from 10 to 30 kPa. After the embankment reached the fill height of 12 m, pore water pressures at the 3 m level continued to increase from 30 to 50 kPa. This increase was attributed to shear deformations occurring within the embankment, and to pore water pressure migration from the center of the embankment towards the slope face. During the year following completion of the fill the geogrids gradually strained as the pore pressures increased. Although full understanding of the interaction between the geogrid reinforcement and the soil may require further analysis, it was clear that the increase in strain within the geogrid, and thus load in the reinforcements, was in direct response to both horizontal and vertical deformations in the embankment soil. The measured deformations, in turn, can be interpreted in terms of the generation and dissipation of pore water pressures.

In the previously described monitored case histories, the pore water pressures were generated during construction of the reinforced soil structures. Another critical situa-

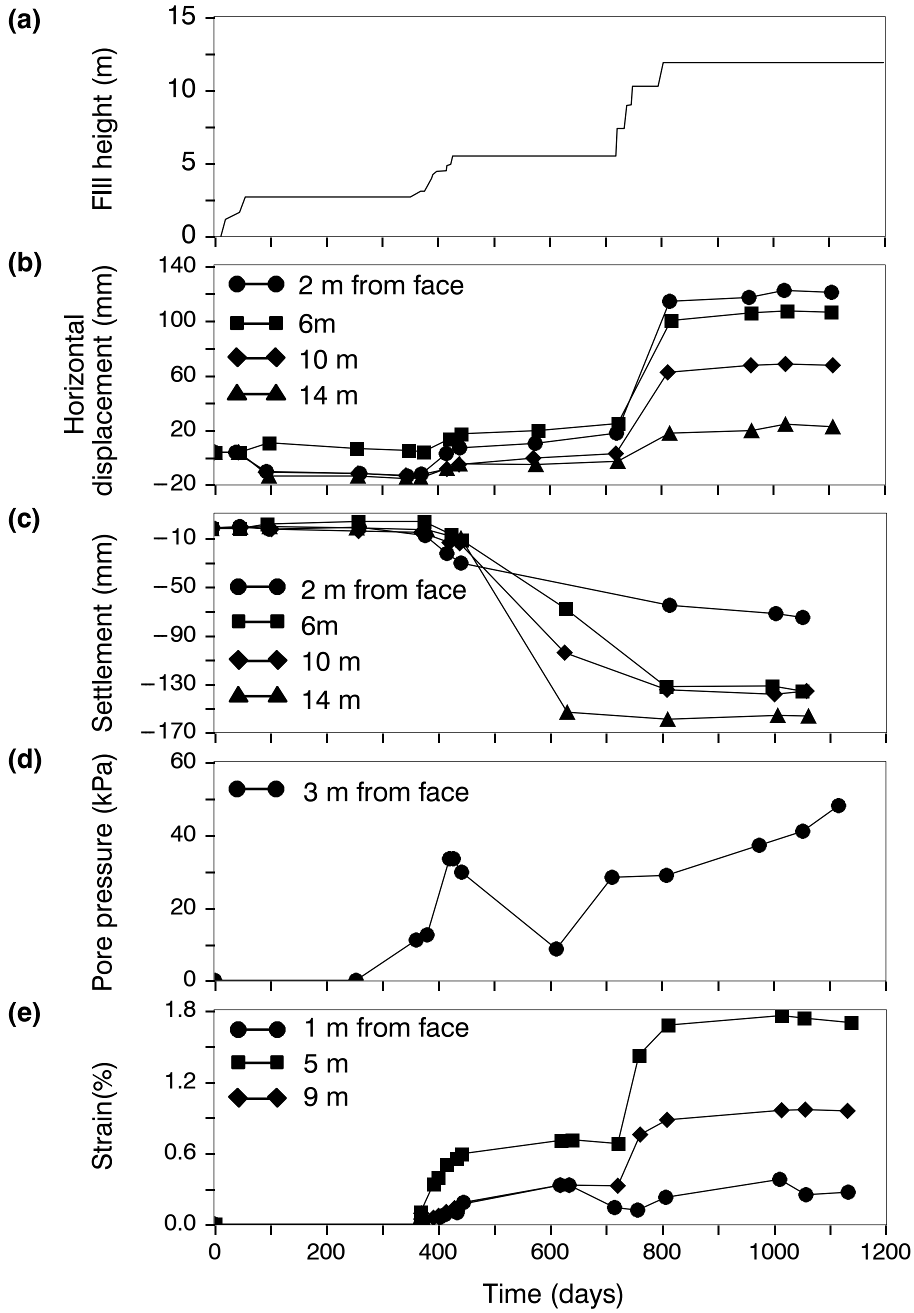

Figure 4. Field measurements at 3 m above base and at various distances from slope face within embankment and geogrids of Devon test fill: (a) fill height; (b) horizontal displacements; (c) settlements; (d) pore pressures; (e) geogrid strains (after Sego et al. 1990).

tion results from water infiltration after rainfall events, but no case histories have been found that monitored this condition. However, failure cases of reinforced soil structures with poorly draining backfills were reported to have been caused by the saturation of the backfill due to water infiltration (Elias and Swanson 1983; Mitchell and Villet 1987; Burwash and Frost 1991; Huang 1992). These structures, some of them described in Section 2.3.1, were constructed with marginal backfill soils reinforced using impermeable inclusions.

2.2.2 *Structures Reinforced Using Permeable Elements*

An experimental embankment at Rouen, France, provided information on the combined mechanical and hydraulic functions of permeable geotextiles (Perrier et al. 1986). Pore water pressures were monitored in this 5.6 m high experimental structure, built with a silt backfill having a water content 5% wet of optimum. The structure consisted of three sections reinforced with different types of woven geotextiles and one section reinforced with a composite nonwoven bonded to a polyester geogrid. Figure 5 shows positive and negative pore water pressures as a function of time recorded at different locations within the fill. The pressure sensor inside the embankment and beyond the reinforcement region, indicated as location (4) in the figure, recorded placement excess pore water pressures of as much as 60 kPa at the end of construction. Along the woven geotextile, positive pore water pressures on the order of 20 kPa were registered at the end of construction, 3.5 m from the wall face. These pore water pressures were dissipated in 350 days, becoming negative near the facing. Along the composite geotextile, on the other hand, negative pore water pressures were registered over the whole length of the reinforcement even at the end of construction. As indicated in the figure, pore water pressures along the composite geotextile were systematically lower than those recorded along the non-draining woven textile. The limited drainage provided by the woven geotextiles affected the stability of the structure, since pore water pressures along these reinforcing layers may result in sliding along the interface. As an example, anchorage failure was observed in a nearby test section reinforced with woven polyester (Delmas et al. 1988).

The effect of nonwoven geotextile reinforcements on the stability and deformation of clay embankments was investigated through a series of field tests in Japan (Tatsuoka and Yamauchi 1986; Tatsuoka et al. 1990). A sensitive volcanic ash clay called Kanto loam was used as backfill for these geotextile reinforced embankments which ranged in height from 4 to 5.5 m. The Kanto loam had a degree of saturation of 83 to 90%, and the as-constructed water content was 100 to 120%. Even though the test embankments have been subjected to heavy rainfalls and earthquakes, they have performed satisfactorily. Figure 6 shows the pore water pressure changes in a test embankment 5.2 m high (Test Embankment II) during a heavy rainfall. When the rainfall occurred, the geotextile-reinforced zones at both sides of the embankment (U1, U3, U4, and U6) were able to maintain a high degree of suction (negative pore water pressures), whereas positive pore water pressures were generated in the unreinforced zones (U2 and U5) as water infiltrated into the soil. After the rainfall, the excess pore water pressures dissipated rapidly through the geotextile layers. These results indicate that the nonwoven geotextile was effective as a drainage layer. Limit equilibrium analyses, in which the beneficial

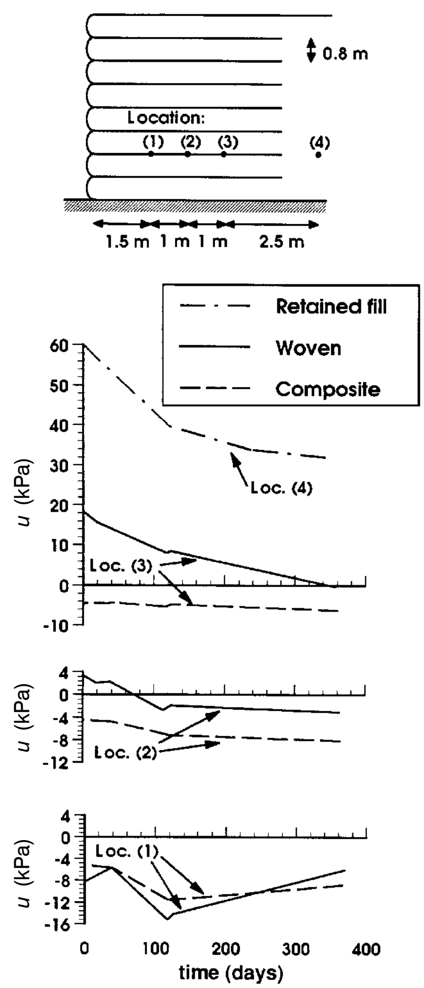

Figure 5. Pore water pressures (*u*) recorded in the Rouen reinforced wall, along a woven and a nonwoven/geogrid composite, at different locations within the silty backfill (redrawn after Perrier et al. 1986).

effect of suction was taken into account, showed that suction in the backfill material contributes significantly to the stability of the clay slopes (Yamauchi et al. 1987).

As part of a highway widening project, the U. S. Federal Highway Administration designed and supervised the construction of a permanent geotextile-reinforced slope 15.3 m high (Barrows et al. 1994). The reinforced structure is a 1H:1V (45°) slope located in Idaho's Salmon National Forest along Highway 93. Several characteristics were unique to the design: the structure was higher than usual for geotextile-reinforced slopes; it involved the use of high strength woven/nonwoven composites; and it was

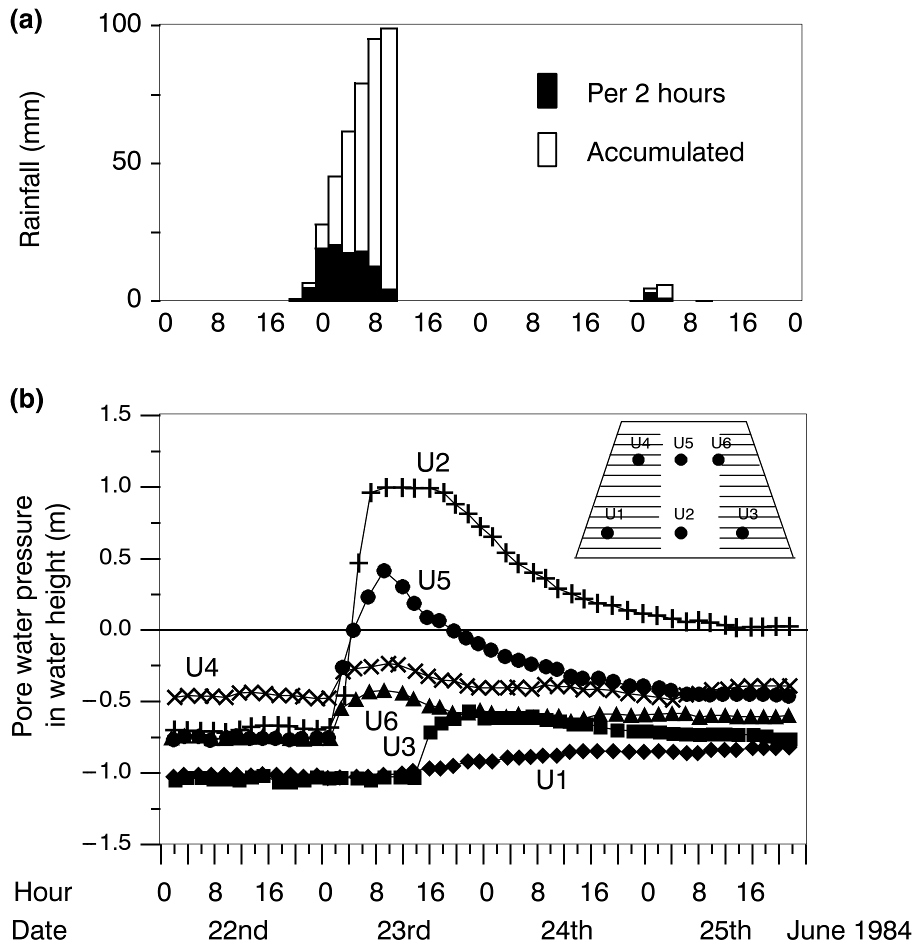

Figure 6. Variation of pore water pressures during rainfall in a clay embankment reinforced with nonwoven geotextiles: (a) rainfall recorded; (b) pore water pressures (after Tatsuoka and Yamauchi 1986).

(Note: U1, U3, U4 and U6 indicate piezometer locations within the fill.)

constructed using indigenous soil (decomposed granite) as backfill material. Consequently, the reinforced slope was considered experimental, and an extensive program of instrumentation and construction monitoring was implemented to evaluate its performance. Piezometers were installed to evaluate generation and dissipation of pore water pressures that could develop either during construction or after rainfall events. Slope construction took place during the summer of 1993. Based on the pore water pressures monitored since construction of the reinforced slope and through the following spring, it can be inferred that the destabilizing flow is not occurring within the reinforced soil mass and that, as considered in the design, a separate drainage system was not necessary at the back of the slope.

2.3 Modes and Causes of Failure

Reduced-scale models have been constructed with the purpose of studying the failure modes in reinforced soil structures using either impermeable or permeable reinforcement elements. Some experimental full-scale structures were also brought to failure to investigate the failure mechanisms and, although without instrumentation records, a few failure cases of real (nonexperimental) reinforced structures have also been reported.

2.3.1 *Structures Reinforced Using Impermeable Elements*

To assess the possibility of using clay fill in the construction of reinforced soil structures, a series of model wall tests was carried out by Ingold (1981) using kaolin clay reinforced with polyethylene geomeshes. Due to the impracticality of bringing a laboratory model to failure by self-weight only, the walls were failed under the application of a vertical surcharge as shown in Figure 7. The surcharge was applied using a rigid platen that had the effect of inducing failure along a preselected plane. Results from these tests were interpreted using total stress analyses which related the surcharge intensity at failure to the geometry and strength parameters of the clay and reinforcement. Reasonable agreement was obtained between observed and calculated values of failure surcharge loads, which were found to increase linearly with the number of layers of reinforcement in the wall.

The failure behavior of reduced-scale structures was also reported by Irvin et al. (1990) for half-scale embankments constructed with London Clay and loaded to failure

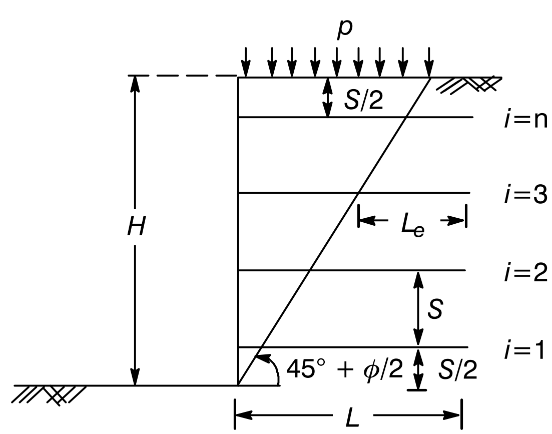

Figure 7. Arrangement of reinforcements in a clay wall model (after Ingold 1981).

(Note: p = vertical surcharge; S = distance between reinforcement layers; H = height of reinforced wall; L = length of reinforcement; L_e = equivalent length of reinforcement behind failure plane; and ϕ = soil friction angle.)

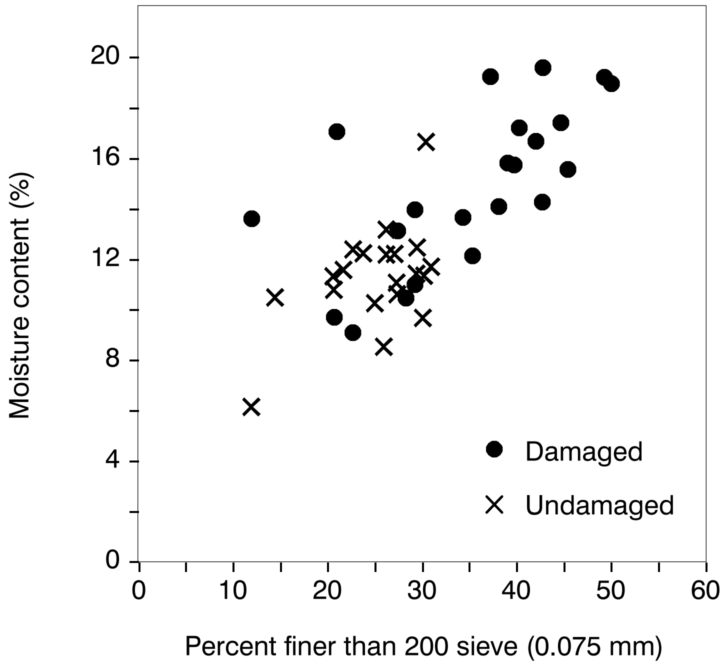

Figure 8. Moisture content and percentage fines for damaged and undamaged walls reinforced with metallic strips (after Elias and Swanson 1983).

with a vertical surcharge. Failure loading was characterized by large internal displacements, large slope face movements, and the development of a near horizontal shear plane above the geogrid layers. The information obtained during sectioning of the embankments, together with the measured displacements, confirmed that the clay fill sheared adjacent to the geogrid layers. After comparing the performance of reinforced and unreinforced embankments, the authors concluded that the geogrid reinforcement modified the mode of deformation, improving the overall stability of the structure and limiting failures to localized areas.

The failures of some full-scale reinforced soil structures constructed with low-quality backfill have been reported. Elias and Swanson (1983) reported on problems that evolved in Reinforced Earth walls constructed during the winter of 1978-1979 in Virginia. The walls varied in height, with a maximum section of approximately 7 m, and specifications required that the backfill be nonplastic with less than 15% passing the no. 200 sieve (0.075 mm). Earthwork was halted due to adverse weather conditions, and significant wall movements were later observed after above normal precipitation. Typical movements consisted of tilting 250 to 300 mm out of plumb, which caused the wall facing to apply a lateral force on some adjacent piers.

To investigate the probable cause of the movements, test borings and hand-dug excavations of the backfill were performed, and detailed tests were conducted (field sampling, moisture contents, compaction tests, and grain size analyses). The cause of the

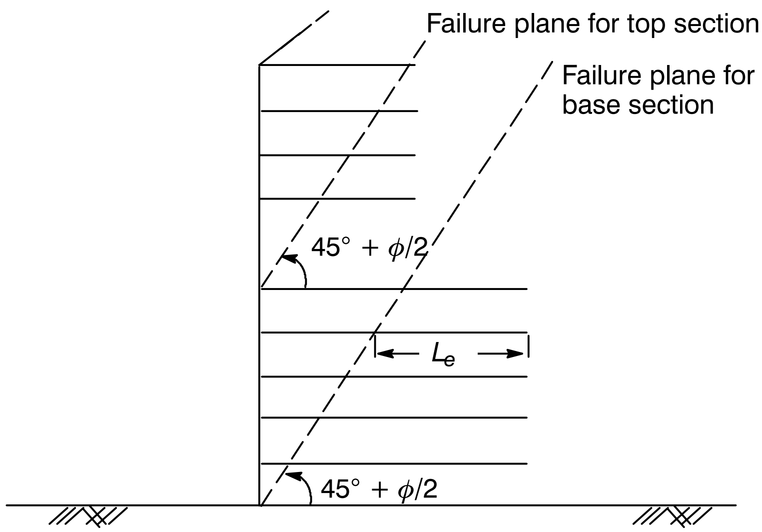

Figure 9. Configuration of welded wire wall on Interstate 580, California (after Mitchell and Villet 1987).

(Note: ϕ = soil friction angle; and L_e = equivalent length of reinforcement behind failure plane.)

problem is shown in Figure 8, which indicates that the reinforced walls with the most severe damage were composed of excessively wet fill with a high fines content. The investigation revealed that a significant portion of the backfill was not within the project gradation specifications since, in the areas of severe wall distress, the backfill contained well over 30% and up to 50% fines. Plasticity limits were also outside of the project specifications. Based on this investigation, the areas of reinforced backfill with more than 25% fines were identified, excavated, and replaced with select backfill. Elias and Swanson concluded that backfill with a high percentage of fines in structures reinforced with steel strips may result in a significant reduction in pullout capacity, decreasing the internal stability of the wall.

A welded wire wall was constructed in 1982 on Interstate 580, near Hayward, California (Mitchell and Villet 1987). This vertical faced wall ranged in height from 1.8 m to 9 m and was about 137 m long. The reinforcing mats in the top section were substantially shorter than those in the bottom section of the wall, as shown in Figure 9. Following construction, a section of the upper portion of the wall was gradually tilting outward, and cracks began appearing at the back of the wall. A 600 mm wide fissure was observed, and remedial backfilling did not solve the problem. Testing of representative soils indicated that, instead of the specified granular backfill, a sandy clay with a moderate potential for expansion had been used. The soil was found to have a water content generally well in excess of optimum and above the plastic limit. The primary cause of the problem was considered to be poor drainage of surface water. Although the original plans called for positive drainage on top of the wall, water was allowed to saturate the backfill material. Remedial measures involved removal of the top layers of

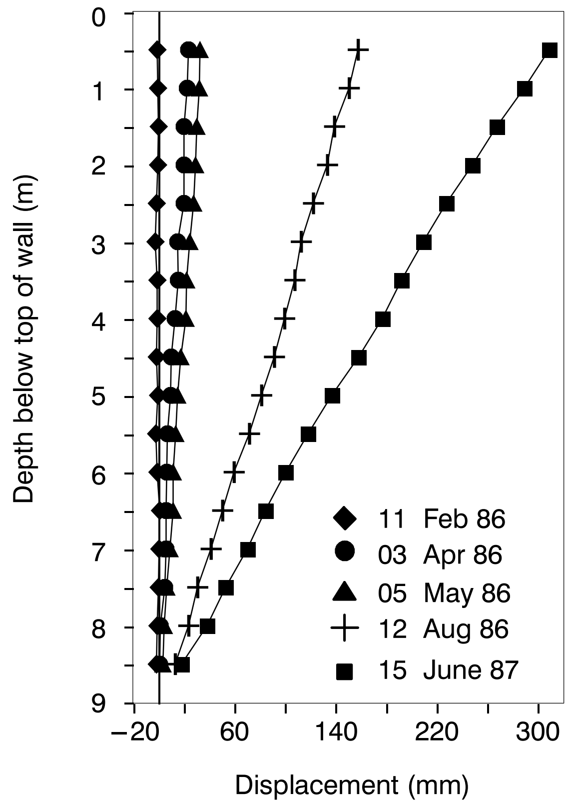

Figure 10. Horizontal displacements versus depth recorded at a geogrid reinforced wall (after Burwash and Frost 1991).

mats, their replacement with longer mats and select backfill, and improved surface drainage to prevent water migration into the wall. The wall has performed satisfactorily since completion of this work.

A 9 m high retaining wall reinforced with polymeric geogrids and backfilled with cohesive soil was constructed in Calgary, Canada, in 1984 (Burwash and Frost 1991). The wall performed satisfactorily for 16 months when signs of settlement were first observed in the fill behind the wall. Conditions gradually deteriorated and, over the next 22 months, settlement of the backfill approached 900 mm in one area. The top of the retaining wall rotated outward about the toe and a deflection of 310 mm was recorded with a slope indicator over a 17 month period (Figure 10). The rates of displacement were, in general, constant. The post-construction site investigation showed that the moisture content of the clay backfill had increased significantly from that measured during construction of the wall. The upper 3 m of the fill appeared to be saturated and was much softer than when placed. The poor performance of this retaining wall was then believed to be related to saturation of the clay backfill which was placed 4% dry of optimum. Saturation occurred by ponding of surface run-off near the face of the wall and, consequently, the geogrids were subjected to increased loads to compensate for the resulting loss in soil strength. Approximately 3 years after completion of construction, the upper 6 m of wall was replaced with a free standing 2H:1V slope.

2.3.2 *Structures Reinforced Using Permeable Elements*

Reduced-scale models were constructed by Fabian and Fourie (1988) to study failure modes in walls reinforced using permeable nonwoven geotextiles. The clay wall models were tested by applying a vertical load using a rigid plate, while strains in the geotextile reinforcements were monitored. The peaks on the strain distribution curves indicated the location of the failure surface. The authors considered that even in undrained loading conditions the true failure surface should be inclined at $45°+\phi'/2$. A good agreement was reported between the inclination of the observed failure plane and the theoretical one.

Centrifuge models of geotextile reinforced and unreinforced vertical walls were reported by Goodings (1990). Models were built of kaolin clay placed at its plastic limit and compressed using a pressure of 200 kPa applied to each layer of soil. The models were reinforced with nonwoven geotextiles with variable vertical spacings and lengths. Two modes of failure were observed in the models after centrifuge loading until catastrophic failure. In lightly reinforced walls, the characteristic mode of failure was the opening of a tension crack followed by overturning and geotextile breakage (Figure 11a). In intermediate to heavily reinforced models (Figures 11b and 11c), failure was characterized by opening of a tension crack followed by development of an inclined sliding failure surface that emerged on the face of the wall. Failure occurred by geotextile breakage in all cases, never by pullout. Models were also built using mixes of kaolin

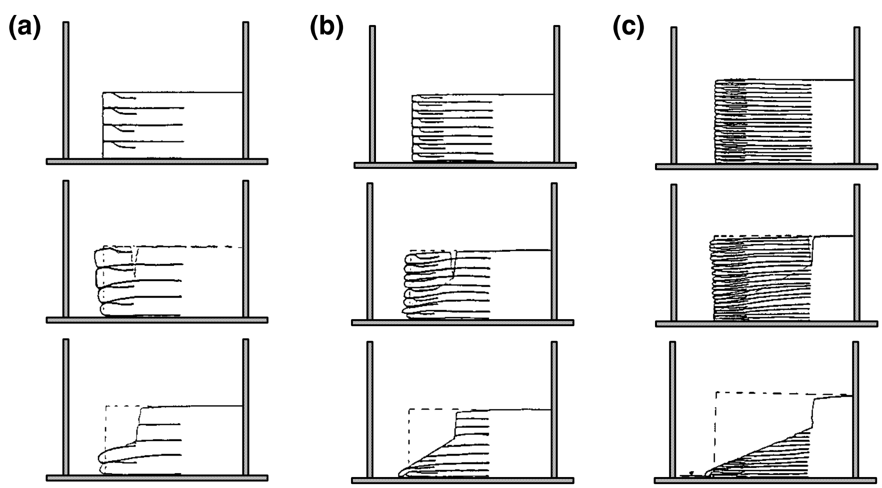

Figure 11. Sequence of failure for centrifuge models of kaolin clay reinforced with nonwoven geotextiles: (a) lightly reinforced model; (b) intermediate reinforced model; (c) heavily reinforced model (after Goodings 1990).

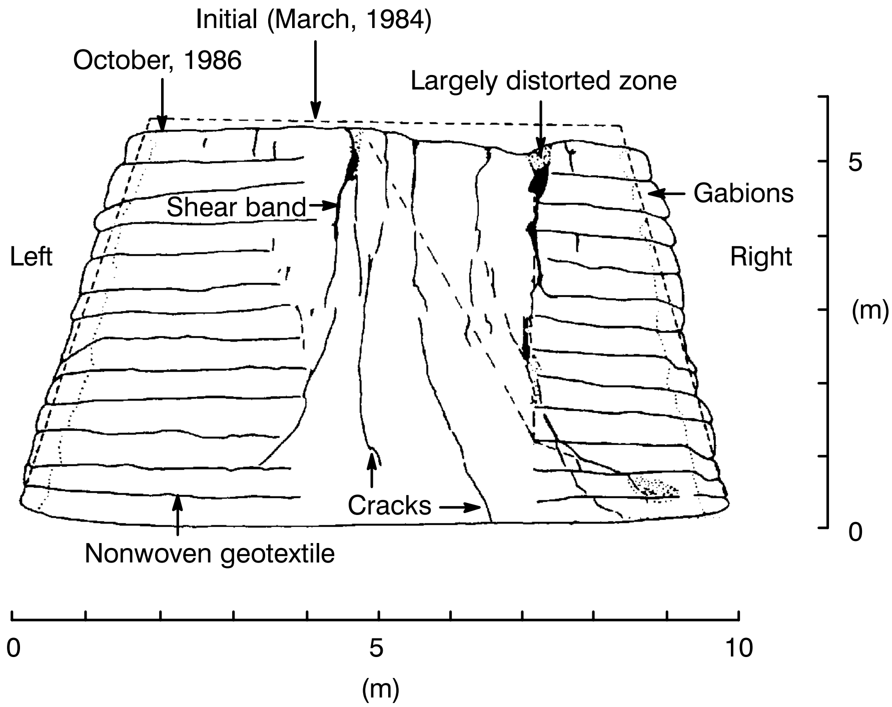

Figure 12. Cross-section of clay Test Embankment II, observed at dismantling (after Yamauchi et al. 1987).

with different percentages of sand as well as different natural soils. The equivalent prototype height of the reinforced walls at failure was compared to the equivalent height of unreinforced walls at failure showing that, in all tested models, reinforcement had a significant beneficial effect. The reinforcement effectiveness increased with the number of reinforcement layers and, for models reinforced with sixteen layers, an equivalent height at failure approximately three times higher than for unreinforced models was achieved.

Five full-scale test embankments, having near-vertical slopes and using permeable reinforcements, were constructed using a nearly saturated clay (Tatsuoka and Yamauchi 1986; Yamauchi et al. 1987). The embankments were made using a volcanic ash clay with a high natural water content and high sensitivity (4 to 5). Test Embankment II was constructed using two layers of gabions that were placed at the edge of each previous layer of the slope, before placing the soil layer. These gabions helped to achieve better compaction of the soil near the slope faces and prevented local failures during and after filling. A spun-bonded polypropylene nonwoven geotextile that demonstrated good in-plane drainage capabilities was used as reinforcement for this embankment.

Two years after construction, the slopes of Test Embankment II did not show any noticeable displacements. It was concluded that the slopes would not displace under natural heavy rainfall. Subsequently, a total supply of about 70 m^3 of water was allowed to

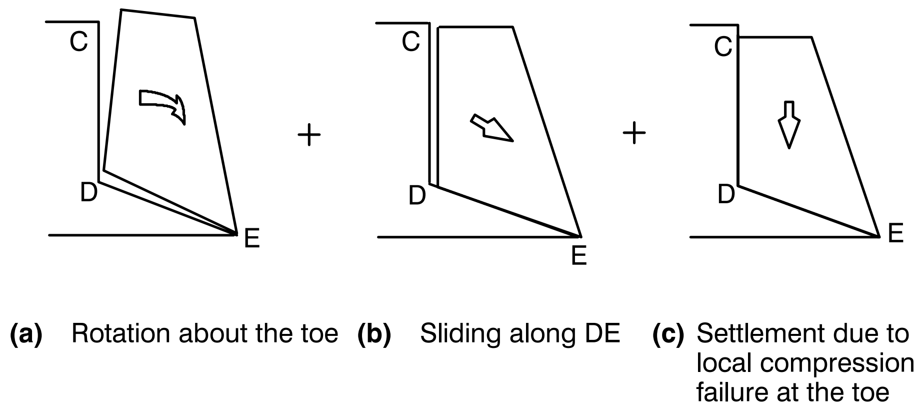

Figure 13. Schematic diagram showing deformation of right-hand slope of Test Embankment II in Figure 12 (after Yamauchi et al. 1987).

percolate from the crest of the embankment over a period of eight days. After the artificial rainfall, several large cracks appeared in the embankment, as shown in Figure 12. The cracks appeared only in the unreinforced fill behind the reinforced zones. Moreover, in spite of the large deformations experienced during the wetting, the long-term deformations observed after the artificial rainfall were very small. Analysis of the cross-section in Figure 12 obtained after dismantling of Test Embankment II indicated that three modes of deformation took place. They are rotation about the toe, sliding along a shear band, and local compression near the toe (Figure 13). Displacements due to the rotational mode were considered to be the largest of the three modes. Since the reinforced zone at the right hand slope rotated as a monolith about the toe and no cracks or slip surfaces were observed in the reinforced zones, it was concluded that the nonwoven geotextiles were effective in reinforcing the cohesive backfill.

2.4 Displacement Evaluation

The magnitude of displacements that occur during and after construction are important considerations in the performance of reinforced soil structures. However, even for reinforced soil structures using good-quality backfill, there is no standard method for prediction of the lateral displacements. Horizontal movements depend on compaction effects, reinforcement extensibility, reinforcement length, reinforcement to facing connection details, and deformability of the facing system (Mitchell and Christopher 1990). Finite element analyses have shown that while reinforcement length has only little effect on the maximum tensions in the reinforcements, its effect on lateral deformation is large. Based on the ratio of reinforcement length to wall height, an estimate of the lateral displacements that may occur during construction of simple structures with granular backfill can be made using Figure 14.

Considering the difficulty involved in the analytical prediction of movements in reinforced soil structures, displacement predictions rely heavily on the reported performance of similar structures. Relevant information about reported displacements on reinforced clay structures is reviewed in this section.

2.4.1 *Structures Reinforced Using Impermeable Elements*

The TRRL embankment, one of the first full-scale embankments constructed using cohesive fill, incorporated seven types of reinforcement (basically plastic strips and steel), four types of facing panel, and three different soils (Boden et al. 1978; Murray and Boden 1979). The layout of this 6 m high trial structure is shown in Figure 1. Because the sandy clay at the bottom of the structure was placed very wet, excess pore water pressures were generated during construction in the bottom layer (see Section 2.2.1), and large horizontal movements and vertical settlements occurred over the first two years after placement of this fill material. Maximum values of vertical settlement of up to 50 mm were recorded just behind the facing panels, and up to 40 mm were measured near the center of the structure. Deviations of the facing panels from vertical were large, with typical values of about 200 mm and extreme values up to 400 mm. Little difference was seen in the vertical profiles between comparable sections of the wall supported by metallic and non-metallic reinforcements.

The performance of a Reinforced Earth wall built in Japan using a volcanic clay as backfill material at a water content greater than 50% is described by Hashimoto (1979).

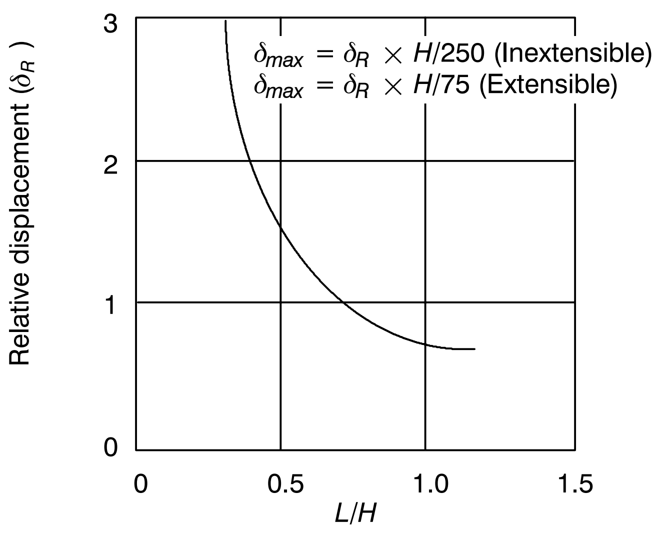

Figure 14. Curve for estimation of lateral displacement anticipated at the end of construction of reinforced walls (after Mitchell and Christopher 1990).

(Note: Based on a 6 m high wall, the relative displacement increases approximately 25% for every 19 kPa of surcharge. Experience indicates that for higher walls the surcharge effect may be greater. L = length of reinforcement; H = wall height; δ_R = relative displacement; and δ_{max} = maximum displacement.)

Lateral displacements of 40 mm were measured in this 8.7 m high wall, while maximum vertical displacements reached 910 mm at the top of the wall.

Battelino (1983) reported the performance of a 3.5 m high wall, reinforced with polyester strips, that used a clayey silt backfill material at a water content of about 20%. Lateral displacements were monitored and reached 35 mm 152 days after the end of construction. The rate of deformation decreased rapidly and was negligible at the end of this period.

To prevent significant movements when impermeable reinforcements are used, water content conditions should be controlled during construction, and appropriate drainage systems should be adopted. An example of reinforced soil structures where appropriate drainage systems were used with impermeable bar-mat reinforcements was reported by Hannon and Forsyth (1984). Four mechanically stabilized embankment walls were constructed for the widening of Interstate 80, near Baxter, California. Two of the four walls were instrumented with strain gauges, pressure cells, reference monuments, plumb points, and piezometers, to monitor the effects of using a low-quality backfill. The maximum wall height was 4.9 m. The material used for the embankments was a sandy silt with about 50% of the material passing the No. 200 sieve (0.075 mm), which is considered excessive for most reinforced soil walls. Since this on-site material was not free-draining and was subject to considerable strength loss when saturated, a subsurface drainage system was constructed. Because of intermittent rains, the fine-grained backfill material became excessively saturated, and construction was forced to stop more than once since additional time was required to dry out the material before work could be resumed. The wall was completed in the fall of 1982. Monitoring of the wall during and after the record rainfall of the 1982-1983 winter showed no significant lateral or vertical wall movements.

Field measurements reported by Sego et al. (1990) for a geogrid reinforced slope constructed with silty clay showed that generated pore water pressures had a significant effect on the performance of the monitored reinforced structure (see Section 2.2.1). As indicated in Figure 4, lateral and vertical displacements were closely related to the generation and subsequent dissipation of pore water pressures.

Displacements in walls and embankments reinforced using either metallic or polymeric impermeable inclusions were also reported by Ingold (1981), Perrier et al. (1986), Temporal et al. (1989), Irvin et al. (1990), Bergado et al. (1991), and Hayden et al. (1991), as described in Tables 1 and 2. Although large movements were observed in some of the structures having a cohesive backfill placed at high water content, an acceptable performance was generally reported if no increase in water content occurred in the backfill after construction. However, as described in Section 2.3.1, the increase in water content due to heavy rains has been critical to structures reinforced with impermeable inclusions.

2.4.2 *Structures Reinforced Using Permeable Elements*

Fabian and Fourie (1988) measured deformations in wall models built using a silty clay soil as backfill material and nonwoven needle-punched geotextiles as reinforcement. Models with and without geotextile reinforcement were failed under the application of a vertical surcharge. The results showed that the vertical load-bearing capacity of the wall can be significantly increased with these geotextiles. Figure 15 shows curves

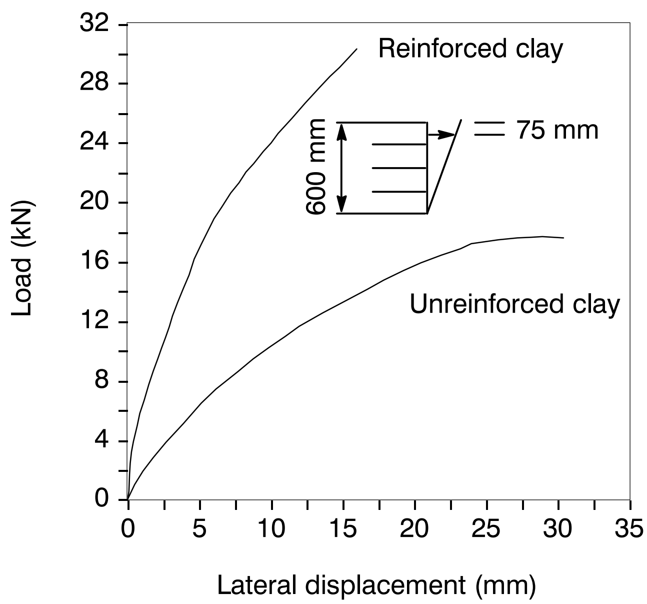

Figure 15. Load-horizontal displacement curves of reinforced and unreinforced clay wall models (after Fabian and Fourie 1988).

of load versus horizontal displacement, at the location of the top geotextile layer, for geotextile-reinforced and unreinforced wall models. Since failure was reached in less than 20 minutes in most of the tests, the loading condition was regarded as undrained. Clearly, the reinforced wall model did not reach failure at the displacement that caused failure in the unreinforced wall.

The first geotextile-reinforced wall was built by the French Highway Administration in Rouen (Puig and Blivet 1973; Puig et al. 1977). Weathered chalk, silt and fire stone were used as backfill material, and a surcharge load was placed on top of the vertical faced wall. The structure, 4 m high and 20 m long, was founded on very compressible peat having a natural moisture content of 300%. As illustrated in Figure 16, layers of polyester needle-punched nonwoven geotextile were placed extending 5 to 6 m behind the wall face, and the wall face was formed by wrapping geotextile layers around 0.5 m thick backfill layers. A berm was raised on the passive side of the wall as construction proceeded and was partially removed after the end of construction. The purpose of this berm was to provide stability for the wall and its compressible foundation, and to support a temporary wood-form system used for the facing. Lateral deformations on the order of 20 mm were recorded on the wall face, and were confirmed by an inclinometer located in the reinforced fill. A total settlement of 1.1 m, and differential settlements of about 250 mm over a length of 3 m were observed. The drainage action of the geotextiles in this structure was later confirmed by traces of deposited calcite found on nonwoven samples taken from the wall in 1986 (Delmas et al. 1988).

The stabilizing function of structural facing elements in steep reinforced clay embankments was examined based on the behavior of five full-scale test embankments

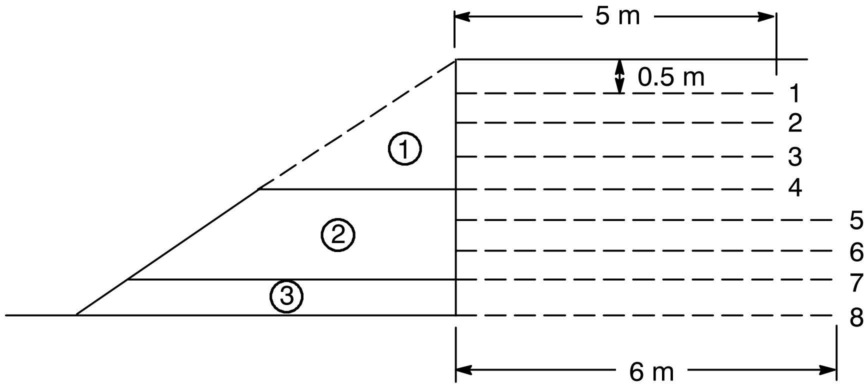

Figure 16. Geotextile reinforced wall on Autoroute A15, France (after Puig and Blivet 1973).
(Note: Zones 1 and 2 were removed after construction.)

constructed using a nearly saturated clay (Tatsuoka et al. 1990). The performance of one of these structures, Test Embankment II, was partly described in Sections 2.2.2 and 2.3.2. Based on the behavior of these nonwoven geotextile-reinforced embankments, the authors concluded that facing structures with various kinds of rigidities should be used to increase the stability of steep slopes. These various kinds of rigidities were classified as local rigidity, overall axial rigidity, and overall bending rigidity. The slope faces of the different structures were either wrapped around with nonwoven geotextile, covered with discrete concrete panels, or constructed with the aid of gabions. The deformations in the slopes wrapped around with nonwoven geotextiles were generally larger than those in the other two slopes. It was concluded that the use of full height continuous rigid facing would be effective in reducing the deformations in clay reinforced walls. Based on the results of this study, the authors proposed that steep clay slopes be designed using relatively short nonwoven geotextile sheets, but using structural facing elements to prevent large lateral movements.

The lateral drainage provided by nonwoven geotextiles has proved effective in reducing or eliminating pore water pressures in the backfill material. The use of geotextile composites with higher tensile strength than that of nonwovens, would expand the use of geotextiles as reinforcement for more critical, permanent structures.

3 BENEFITS AND POTENTIAL APPLICATIONS OF POORLY DRAINING BACKFILLS IN REINFORCED SOIL CONSTRUCTION

Although there are no design guidelines for reinforced soil structures using marginal soils, good performances were observed for reinforced soil structures that adequately prevented the generation of pore water pressures in the fill. Thus, it is clear that proper design can lead to the use of fine grained marginal soils as backfill material for reinforced soil construction, providing important cost savings and new soil reinforcement applications.

One potential solution for reinforcing marginal soils is the use of permeable geosynthetics that function not only as reinforcements but also as lateral drains (Zornberg and Mitchell 1994). This would lead to a number of benefits:

- reduced cost of structures that would otherwise be constructed with expensive select backfill;
- improved performance of compacted clay structures that would otherwise be constructed without reinforcements; and
- use of materials, such as, nearly saturated cohesive soils and mine wastes, which would otherwise require disposal, in civil engineering construction projects.

The generally specified granular backfill material may lead to high transportation costs, and the disposal of unused cohesive soils can also generate substantial costs. While the reinforcement materials generally account for a relatively small portion of the total cost of the structure, the cost of granular backfill may be as much as half the total cost. For example, Hollinghurst and Murray (1986) reported that from the total cost of a 6 m high reinforced earth wall only 17% represented the reinforcement elements, while 25% represented the facing, 40% the granular fill, 15% the parapets and foundation, and 3% represented the earthwork. Hayden et al. (1991) reported that constructing a geogrid reinforced clay embankment costs a total of $2.1 million, resulting in about $1.1 million savings over conventional alternatives such as the importation of granular fill.

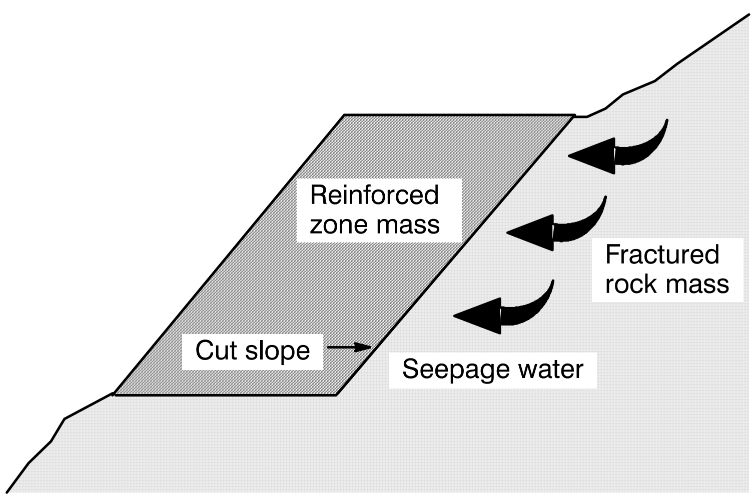

Figure 17. Water infiltration in a reinforced slope for road widening projects.

The rate of construction when using free-draining granular backfill is not a design consideration since, even for rapid loading, the fully-drained condition will prevail. This is not necessarily the case for poorly draining fills, where rapid construction is likely to be associated with undrained loading. In this case, permeable reinforcements, such as nonwoven geotextiles, could be used to increase the rate of consolidation and, consequently, speed the embankment construction. The dissipation of pore water pressure will increase both the shear strength of the cohesive backfill and the pullout resistance along the soil-reinforcement interface.

The controversial issue of what type of stability analysis should be used to design staged construction projects and to check stability during actual construction was addressed by Ladd (1991). Staged construction uses controlled rates of loading to enable soil strengthening via consolidation in order to increase the foundation stability of structures such as dams, embankments, landfills, and tanks founded on soft cohesive soils. It is also used for the operation of many tailings waste storage dams. A reinforced soil wall or embankment with poorly draining backfill and permeable reinforcements is another type of structure to be added to the list of geotechnical structures requiring staged construction. In this case, however, it is the strengthening due to consolidation of the fill material, and not of the foundation soil, that may require controlled rates of loading to guarantee stability. The speed of construction of this particular type of staged construction will be governed by the drainage capabilities of the reinforcement layers.

Permeable reinforcements would not only be useful to dissipate pore water pressures generated during construction, but can also prevent the formation of flow configurations with destabilizing seepage forces within the embankment fill. Transient and steady state seepage conditions in natural and artificial slopes have a significant effect on the slope stability. An infinite slope analysis gives an indication of the potentially adverse effect of seepage forces on slope stability: while in an infinite slope without seepage the maximum angle of stability is equal to the soil friction angle, in a slope with seepage forces parallel to the surface the maximum stable angle is approximately half the soil friction angle. Although the adverse effect of seepage forces in engineered slopes could be prevented by designing special drainage systems, a more economical design alternative could be to combine drainage and reinforcement capabilities by using permeable geosynthetics as reinforcement elements. Internal drainage is of particular concern in road widening projects, because of the potential water seepage from cut slopes, in fractured rock, into the reinforced fill, as shown in Figure 17. Geotextile layers have already been used to provide basal drainage of unreinforced embankments placed on compressible and saturated soils. Ingold (1992) analyzed the stability of an embankment where surface water infiltration threatened long-term stability (Figure 18a) and demonstrated that the flow regime obtained using a basal geotextile layer (Figure 18b) led to a substantial increase in the embankment stability. Multiple permeable reinforcement layers would also be effective in preventing destabilizing flow regimes caused by infiltrating water.

The performance of properly designed and constructed reinforced soil walls during earthquakes has been excellent (Mitchell and Christopher 1990). Qualitative assessment has been made on the performance of structures reinforced with inextensible elements and geosynthetic reinforcements that have actually experienced earthquake excitation during the Loma Prieta earthquake (The Reinforced Earth Company 1990; Collin et al. 1992) and during the recent 1994 Northridge earthquake (Stewart et al.

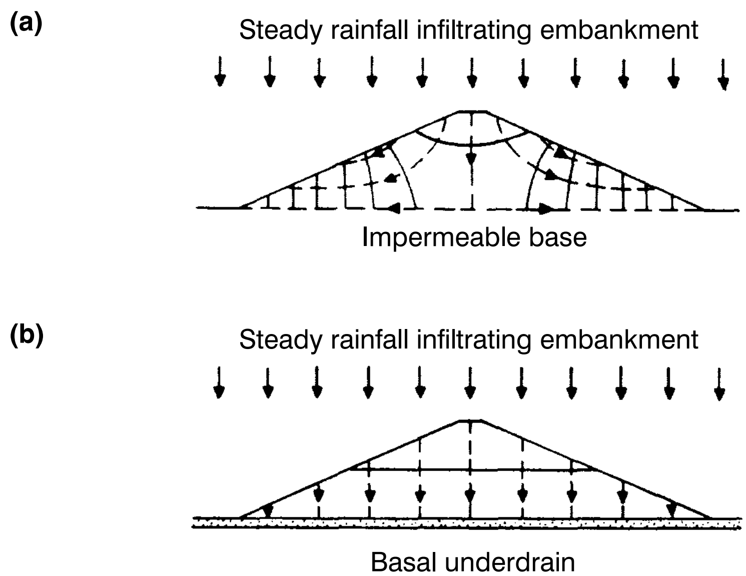

Figure 18. Flow regime for embankment: (a) on an impermeable base; (b) on a pervious base (after Ingold 1992).

1994). No significant signs of structural distress or movements have been observed during these events. A good performance was reported for an embankment built using a clay backfill and reinforced with nonwoven geotextiles, that experienced relatively large earthquake motion (Nakamura et al. 1988). Any amplification of accelerations in structures with extensible reinforcements, should be compensated by the greater damping in less stiff systems and by the higher factors of safety adopted on the reinforcement tensile strength to allow for creep under long-term static loads. It may also be speculated that lateral drainage provided by permeable reinforcements would be beneficial in dissipating excess pore water pressures generated during seismic events in a reinforced fill.

There are potentially new applications of soil reinforcement using on-site, generally marginal soils for waste landfill construction. For waste repository construction in which the waste is to be placed in an excavation, steep sidewall slopes help maximize the available waste storage volume for a given site area. However, the repository must be designed considering several possible failure modes and mechanisms of the landfill during excavation, during filling, and after closure (Mitchell and Mitchell 1992). Among them, sidewall slope failures can occur during the excavation of a repository and during the placement of liner systems prior to the commencement of filling operations. Use of reinforcements to provide stable steep sidewall slopes would be an economic design alternative. These reinforced slopes could be designed as temporary structures since the reinforcement function of the geosynthetics would be required only until filling of the basin is completed. The low reduction factors (creep, durability) on the geosynthetic tensile strength required for temporary structures, would lead to an economic design. Instead of steepening the sidewall slopes, the construction of vertical reinforced sidewalls would be another potential alternative design. Besides maximizing the storage volume, a vertical excavated wall would eliminate other potential failure modes such as pullout of liner system components from anchor trenches, and sidewall failure along interfaces within the composite liner system. Current landfill design accounts for these failure modes by using flat side slopes, that can result in considerable reduction of waste storage volume.

If the strength of industrial and mine wastes could be increased by reinforcement, the range of civil engineering uses for these materials would be greatly broadened. Embankment construction using mine waste as a backfill material has already been reported by Jewell and Jones (1981). The range of particle size distributions found for mine waste materials is highly variable and depends on many factors including the method of handling and placement. Many materials are predominantly fine-grained, but include sand and gravel sized particles. Although plasticity characteristics of mine wastes vary substantially, there are strong similarities to inorganic clays of medium plasticity. Another potential alternative is the use of reinforced waste materials not for construction purposes, but with the objective of facilitating waste placement in storage systems. The use of cheap geotextiles could be effective in preventing failures through the waste pile which is a critical failure mode for low-strength waste materials.

The use of admixture stabilizers, while not yet fully investigated, may enhance the range of materials that can satisfactorily be used for reinforced soils. The addition of lime to stabilize cohesive soil for use as fill in geotextile reinforced walls was investigated by Güler (1990). A successful performance was obtained by using quicklime and a filter geotextile for embankments in a difficult cohesive soil in Japan (Yamanouchi et al. 1982). Additionally, good performance was reported for a geogrid reinforced slope constructed at Yattendon, U.K., where clay fill was stabilized with lime (O'Reilly et al. 1990), and for a geogrid reinforced slope in Japan built for a waste disposal facility using cement-stabilized cohesive backfill (Toriihara et al. 1992). Centrifuge models of geotextile reinforced soil retaining walls using lime stabilized kaolin have been tested to failure by increasing the self-weight (Güler and Goodings 1992) and demonstrated that lime improved wall stability substantially.

The usefulness of consolidation by electro-osmosis as a technique for stabilization has been recognized in a number of geotechnical applications (Mitchell 1991). The use of electro-osmosis for accelerating the consolidation process in reinforced structures with cohesive backfill may deserve some speculation. If cohesive soil with high as-placed water content is used as backfill material, a time-dependent gain in both soil strength and pullout resistance occurs as pore water pressures dissipate. However, if a slow rate of pore water dissipation rate compromises either the stability of the embankment or the construction speed, electrically driven flow could be generated by placing electrodes along the reinforcements. A mathematical representation of the coupled flow generated by electro-osmosis would need to be formulated. Implementation, practicality, and costs involved in using this stabilization method are yet to be evaluated.

The advantages of using nonwoven geotextiles in clay embankments are not just limited to their reinforcement and drainage functions. Two problems frequently reported for embankments of (unreinforced) compacted clay are: the development of surface tension cracks, and compaction difficulties. In a reinforced soil structure, any surface tension cracks in the cohesive fill will be limited to the region above the first geosynthetic layer. Moreover, the use of nonwoven geotextiles has been reported to help in the

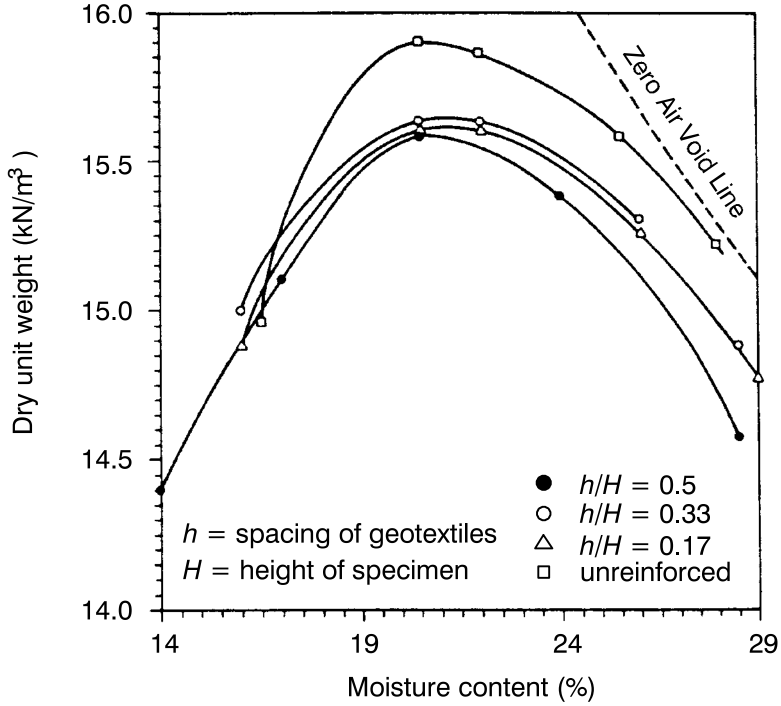

Figure 19. Effect of nonwoven geotextile spacing on compaction curves for reinforced clay specimens (after Indraratna et al. 1991).

compaction of the fill, by allowing a better distribution of the compaction effort and by draining excess pore water pressure induced during compaction (Yamauchi et al. 1987). The compaction characteristics of a geotextile-reinforced soft marine clay have been investigated by applying to reinforced soil samples, a known compactive effort, equivalent to that of the Standard Proctor test (Indraratna et al. 1991). Figure 19 shows the compaction results for specimens reinforced with an increasing number of nonwoven geotextile layers. The increase in dry unit weight was significant for the reinforced specimens, particularly at a close geotextile spacing, with no significant change in the optimum moisture content. In contrast, woven geotextiles were reported to barely contribute to the compaction of the clay specimens.

The use of geosynthetics in cohesive soils has also been suggested for purposes other than reinforcement. For example, a geosynthetic based solution to the problem of expansive clays was investigated by Al-Omari and Hamodi (1991). Experimental results revealed a significant reduction in swell due to geogrid reinforcement.

4 RESEARCH NEEDS

The results of experimental studies and the performance of several reported case histories have shown that poorly draining fills can be efficiently improved if the appropriate reinforcement systems are used. Nevertheless, soil-reinforcement interactions in cohesive backfills are still not fully understood and no generally accepted design methodologies are currently available. On the basis of the review done in this and a companion paper (Zornberg and Mitchell 1994), aspects that require further insight to achieve safer and more economical designs of reinforced soil structures with poorly draining backfills are identified in the following.

Analysis of Poorly Draining Soil-Geosynthetic Interaction. Although different mechanisms have been proposed to explain soil-reinforcement interactions, more detailed understanding is needed in order to better define the load transfer mechanisms. The influence of confinement on the stress-strain characteristics and on the transmissivity of geosynthetics requires special consideration.

Analytic Treatment of Pore Water Pressure. The effect of geosynthetic transmissivity and reinforcement spacing on the pore water pressure dissipation within a reinforced clay fill should be further investigated. As pore water pressures dissipate, there is a coupled increase in both soil shear strength and pullout resistance that requires analytic formulation. Not only the effect of permeable inclusions on the dissipation of pore water pressures generated during construction, but also their effect on preventing permanent and transient flow configurations should be addressed.

Selection of Design Methods and Failure Criteria. Even though some geosynthetics have been shown to effectively dissipate excess pore water pressures, design methods and failure criteria that take into consideration the combined effects of geosynthetic transmissivity and reinforcement have not been developed. Practical methods for predicting the increase with time of the stability factor of safety as consolidation proceeds, as well as the speed of construction required to keep a minimum factor of safety, should be developed.

Deformation Analysis of Reinforced Soil Structures with Poorly Draining Backfills. The use of cohesive backfills in reinforced soil construction produces less stiff structures than those constructed with conventional granular backfill. Consequently, reinforcements will play an even more relevant role in preventing excessive lateral deformations. The influence of reinforcement stiffness and length, intensity of soil compaction, and types of facing structures on the lateral deformations and on the reinforcement tension distribution should be addressed. The ability of permeable reinforcements, stiffer than nonwoven geotextiles, to prevent large lateral deformations should be particularly investigated.

Selection of Reinforcements. The most appropriate geosynthetic types to be selected for these reinforced structures needs better definition. When interface friction is a controlling factor in the choice of a reinforcement material, nonwoven geotextiles offer good characteristics because of their high contact efficiency and because they can convey water coming out of the soil due to consolidation. However, if tensile strength controls the design, the use of composite geosynthetics or high strength nonwoven geotextiles should be considered.

Dynamic Response Analysis. Reinforced soil structures constructed using granular fill materials have shown excellent performance during earthquakes. Greater damping may be expected in less stiff structures constructed using cohesive backfills. Nevertheless, more verification of the seismic stability of structures with poorly draining backfills is needed.

Evaluation of Geosynthetic Durability in Cohesive Materials. Since poorly draining soils constitute a more aggressive environment than cohesionless soils, there remains some concern about geosynthetic durability. Reported tests on retrieved geosynthetic samples show encouraging results (Zornberg and Mitchell 1994). Nevertheless, accumulation of field data on different exposure conditions and in different soils is essential, since durability predictions are based primarily on observations of buried materials used for other purposes. A method for classification of polymers regarding their ability to resist chemical degradation is needed.

Estimation of Geosynthetic-Cohesive Soil Creep Potential. Reinforced soil structures using sand backfill, as well as confined laboratory creep tests, have shown only very limited creep deformations. Reinforced structures with poorly draining backfills have also been reported to behave successfully in relation to long-term creep deformations (Zornberg and Mitchell 1994). Nevertheless, caution should be taken due to the higher creep potential of cohesive soils. Long-term pull-out tests in cohesive soils would provide valuable information related to clay-geosynthetic creep response.

Use of Admixture Stabilization and Electro-Osmosis for Fill Improvement. The possibility of using admixture stabilizers, such as cement and lime, for improving poor or marginal backfill soils should be further investigated. Stabilization of reinforced clay structures by electro-osmosis should be analyzed. Economical and technical viability of these backfill improvement techniques require careful examination.

Study of the Potential Use of Poorly Draining Wastes as Backfill Materials. If the strength of industrial, domestic, and mine wastes could be improved by reinforcement, then the range of civil engineering uses for these materials would be greatly increased. In view of the rapidly increasing production of mine wastes in industrialized countries, new potential applications of these materials such as in reinforced tailings dams or embankments should be considered.

5 CONCLUSIONS

This and a companion paper (Zornberg and Mitchell 1994) contain the results of a review and evaluation of published material on the use of poorly draining soils in reinforced soil structures. The cohesive soil-reinforcement interaction and the hydraulic function of reinforcements are reviewed in the companion work, while this paper is focused on the lessons learned from field case histories. Permeable geotextile reinforcements may be especially useful for soil structures with poorly draining backfills because the drainage capabilities of the geotextile helps to increase the structure's stability by dissipating excess pore water pressures. Although reported results have led to some contradictory conclusions on the effects of impermeable reinforcement layers, there is already strong experimental evidence that permeable reinforcements can effectively reinforce poorly draining backfills (Zornberg and Mitchell 1994).

The use of fine-grained poorly draining materials in reinforced soil structures would reduce the cost of projects that would otherwise require granular material to satisfy current specifications, and would broaden the range of use of soil reinforcement to new applications. Geosynthetic reinforcements with high in-plane transmissivity not only provide mechanical reinforcement to the marginal fill, but their drainage properties can prevent destabilizing water flow configurations in a reinforced slope. In addition, the reinforcement limits the development of tension cracks in the cohesive fill, and may simplify soil compaction operations. It may also be speculated that lateral drainage would be beneficial during seismic events. The use of geosynthetic reinforcements to strengthen industrial and mine wastes for use as backfill materials, instead of disposing them into a landfill, and the reinforcement of sidewall slopes in waste repository systems are examples of potential new applications.

No consistent design methodology for reinforced soil structures containing poorly draining backfills has been developed. Nevertheless, a number of structures have been constructed, and the performance of some of them has been reported. Reduced and full-scale reinforced soil structures with poorly draining backfills were evaluated, focusing particularly on, the generation of pore water pressures in the fill, on the possible modes and causes of failure, and on the structure deformability. Analysis of these case histories shows that large movements were generally recorded in reinforced structures when pore water pressures were generated in the fill, especially in those containing metallic reinforcements. Thus, good performance strongly depends on prevention of excess pore water pressure development within the fill material. This conclusion is strengthened by the fact that the failure cases reported thus far involved poorly draining backfills that became saturated due to surface run-off, and were reinforced with impermeable inclusions.

Metallic reinforcements are not strong reinforcement candidates for poorly draining backfills. Not only do they not provide lateral drainage to the cohesive fill, but also the interface friction of these systems relies on the dilatant characteristics offered by granular fills. An additional concern is the higher rate of corrosion of metallic reinforcements when embedded in cohesive soils. Polymeric grid reinforcements and woven geotextiles provide adequate tensile strength required for the design of permanent reinforced soil structures. However, since they offer a limited in-plane drainage capacity, a low moisture content in the fill should be guaranteed by appropriate drainage systems throughout the design life of the structure. Nonwoven geotextiles, having a high in-plane hydraulic conductivity, offer the desired drainage capacity both during construction and after rainfall events. However, the generally lower strength and stiffness of these materials have limited their use to low or temporary structures. In order to reinforce marginal soils, it is apparent that new synthetic materials with both high in-plane drainage capacity and high tensile strength and stiffness will be valuable. Composite geosynthetics, that combine the hydraulic properties of nonwovens with the mechanical characteristics of geogrids or wovens, are probably the most appropriate reinforcement for marginal soils.

A number of research needs should be addressed in order to formulate a consistent design methodology for reinforced soil structures with poorly draining backfill materials. They include: the analytic treatment of pore water pressures in the fill taking into account the reinforcement transmissivity; a better understanding of marginal soil-geosynthetic interactions; the development of methods for deformation prediction; and further evaluation of durability and creep potential of geosynthetics embedded in cohesive soils. Due to an increasing demand for structures constructed using indigenous soils, current needs go beyond the fundamental understanding of the problem, and a consistent design methodology for walls and embankments with poorly draining backfills should be formulated.

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REFERENCES

Al-Omari, R.R. and Hamodi, F.J., 1991, “Swelling Resistant Geogrid - A New Approach for the Treatment of Expansive Soils”, *Geotextiles and Geomembranes*, Vol. 10, No.4, pp. 295-317.

Barrows, R.J., Zornberg, J.G., Christopher, B.R. and Wayne, M.H., 1994, “Geotextile Reinforcement of a Highway Slope”, *Geosynthetics Case Studies Book for North America*, Bathurst, R.J., Ed., (in press)

Battelino, D., 1983, “Some Experience in Reinforced Cohesive Earth”, *Proceedings of the Eighth European Conference on Soil Mechanics and Foundation Engineering*, Balkema, 1983, Vol. 2, Helsinki, Finland, May 1983, pp. 463-468.

Bell, J.R. and Steward, J.E., 1977, “Construction and Observation of Fabric Retained Soil Walls”, *Proceedings of the International Conference on the Use of Fabrics in Geotechnics*, Vol. 1, Paris, pp. 23-128.

Bergado, D.T., Shivashankar, R., Sampaco, C.L., Alfaro, M.C. and Anderson, L.R., 1991, “Behavior of a Welded Wire Wall with Poor Quality, Cohesive-Friction Backfills on Soft Bangkok Clay: a Case Study”, *Canadian Geotechnical Journal*, Vol. 28, No. 6, pp. 860-880.

Boden, J.B., Irwin, M.J. and Pocock, R.G., 1978, “Construction of Experimental Reinforced Earth Walls at the TRRL”, *Ground Engineering*, Vol. 11, No. 7, pp. 28-37.

Brady, K.C. and Masterton, G.G.T., 1990, "Design and construction of an Anchored Earth Wall at Annan", *Performance of Reinforced Soil Structures*, McGown, A., Yeo, K., and Andrawes, K.Z., Eds., Thomas Telford, 1991, Proceedings of the International Reinforced Soil Conference held in Glasgow, Scotland, September 1990, pp. 127-134.

Burwash, W.J. and Frost, J.D., 1991, "Case History of a 9 m High Geogrid Reinforced Retaining Wall Backfilled with Cohesive Soil", *Proceedings of Geosynthetics '91*, IFAI, 1991, Vol.2, Atlanta, GA, USA, February 1991, pp. 485-493.

Collin, J.G., Chouery-Curtis, V.E. and Berg, R.R., 1992, "Field Observation of Reinforced Soil Structures under Seismic Loading", *Earth Reinforcement Practice*, Ochiai, Hayashi and Otani, Eds., Balkema, 1992, Proceedings of the International Symposium on Earth Reinforcement Practice, Fukuoka, Japan, Vol. 1, November 1992, pp. 223-228.

Delmas, P., Gotteland, P., Gourc, J.P. and Haidar, S., 1992, "Two Full Size Structures Reinforced by Geotextiles", *Grouting, Soil Improvement and Geosynthetics*, Geotechnical Special Publication No.30, Borden, R.H., Holtz, R.D and Juran, I., Eds., ASCE, 1992, Vol. 2., New Orleans, LA, USA, February 1992, pp. 1201-1212.

Delmas, P., Blivet, J.C. and Matichard, Y., 1987, "Geotextile-Reinforced Retaining Structures: A Few Instrumented Examples", *The Application of Polymeric Reinforcement in Soil Retaining Structures*, Jarrett, P.M. and McGown, A., Eds., Kluwer Academic Publishers, 1988, Proceedings of the NATO Advanced Research Workshop on Application of Polymeric Reinforcement in Soil Retaining Structures, Kingston, Ontario, Canada, June 1987, pp. 285-311.

Dixon, J.H., 1993, "Geogrid Reinforced Soil Repair of a Slope Failure in Clay. North Circular Road, London, United Kingdom", *Geosynthetics Case Histories*, Raymond, G.P. and Giroud, J.P., Eds., BiTech, 1993, pp. 236-237.

Elias, V. and Swanson, P., 1983, "Cautions of Reinforced Earth with Residual Soils", *Transportation Research Record* 919, pp. 21-26.

Fabian, K.J., 1990, "Time Dependent Behaviour of Geotextile Reinforced Clay Walls", *Proceedings of Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Balkema, 1990, Vol.1, The Hague, Netherlands, May 1990, pp. 33-38.

Fabian, K.J. and Fourie, A.B., 1988, "Clay-Geotextile Interaction in Large Retaining Wall Models", *Geotextiles and Geomembranes*, Vol. 7, No. 3, pp. 179-201.

Goodings, D.J., 1990, "Research on Geosynthetics in Reinforced Cohesive Soil Retaining Walls at the University of Maryland", *Geotechnical News*, June 1990, pp. 23-25.

Güler, E., 1990, "Lime Stabilized Cohesive Soil as a Fill for Geotextile Reinforced Retaining Structures", *Proceedings of Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Balkema, 1990, Vol. 1, The Hague, Netherlands, May 1990, pp. 39-44.

Güler, E. and Goodings, D.G., 1992, "Centrifuge Models of Clay-Lime Reinforced Soil Walls", *Grouting, Soil Improvement and Geosynthetics*, Geotechnical Special Publication No.30, Borden, R.H., Holtz, R.D and Juran, I., Eds., ASCE, 1992, Vol. 2, New Orleans, LA, USA, February 1992, pp. 1249-1260.

Hannon, J.B. and Forsyth, R.A., 1984, "Performance of an Earthwork Reinforcement System Constructed with Low Quality Backfill", *Transportation Research Record* 965, pp. 55-66.

Hashimoto, Y, 1979, "Comportement d'un Mur en Terre Armée Construit Avec le Limon de Kanto", *Colloque International sur le Renforcement des Sols*, Vol. 2, Paris, France, pp. 545-550. (in French)

Hayden, R.F., Schmertmann, G.R., Qedan, B.Q. and McGuire,M., 1991, "High Clay Embankment over Cannon Creek Constructed with Geogrid Reinforcement", *Proceedings of Geosynthetics '91*, IFAI, 1991, Vol. 2, Atlanta, GA, USA, February 1991, pp. 799-822.

Hollinghurst, E. and Murray, R.T., 1986, "Reinforced Earth Retaining Wall at A3/A322 Interchange: Design, Construction and Cost", *Proceedings of the Institution of Civil Engineers, Transportation*, Vol. 80, Part 1, pp. 1327-1341.

Huang, C.C., 1992, "Report on Three Unsuccessful Reinforced Walls", *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Tatsuoka, F. and Leshchinsky, D., Eds., Balkema, 1994, Proceedings of Seiken Symposium No. 11, Tokyo, Japan, November 1992, pp. 219-222.

Indraratna, B., Satkunaseelan, K.S. and Rasul, M.G., 1991, "Laboratory Properties of a Soft Marine Clay Reinforced with Woven and Nonwoven Geotextiles", *Geotechnical Testing Journal*, ASTM, Vol. 14, No. 3, pp. 288-295.

Ingold, T.S., 1981, "A Laboratory Simulation of Reinforced Clay Walls", *Géotechnique*, Vol. 31, No. 3, pp. 399-412.

Ingold, T.S., 1992, "Geotextiles used in Basal Drainage of Embankments: a Case History", *Proceedings of the Institution of Civil Engineers, Transportation*, Vol. 95, pp. 193-196.

Ingold, T.S. and Miller, K.S., 1982, "Analytical and Laboratory Investigation of Reinforced Clay", *Proceedings of Second International Conference on Geotextiles*, IFAI, 1982, Vol. 2, Las Vegas, NV, USA, August 1982, pp. 587-592.

Irvin, R.A., Farmer, I.W. and Snowdon, R.A., 1990, "Performance of Reinforced Trial Embankments in London Clay - Undrained Loading", *Performance of Reinforced Soil Structures*, McGown, A., Yeo, K., and Andrawes, K.Z., Eds., Thomas Telford, 1991, Proceedings of the International Reinforced Soil Conference held in Glasgow, Scotland, September 1990, pp. 119-126.

Jaber, M. B., 1989, *"Behavior of Reinforced Soil Walls in Centrifuge Model Tests"*, Thesis submitted in partial satisfaction of the requirements for the degree of Doctor of Philosophy, Department of Civil Engineering, University of California, Berkeley, California, USA, 239 p.

Jewell, R.A. and Jones, C.J., 1981, "Reinforcement of Clay Soils and Waste Materials using Grids", *Proceedings of the Tenth International Conference on Soil Mechanics and Foundation Engineering*, Balkema, 1981, Vol. 2, Stockholm, Sweden, June 1981, pp. 701-706.

Kern, F., 1977, "Realisation d'un Barrage en Terre avec Parement Aval Vertical au Moyen de Poches en Textile", *Proceedings of the International Conference on the Use of Fabrics in Geotechnics*, Vol. 1, Paris, France, pp. 91-94. (in French)

Ladd, C.C., 1991, "Stability Evaluation during Staged Construction", *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 4, pp. 537-615.

Leonards, G.A., Frost, J.D. and Bray, J.D., 1994, "Collapse of geogrid-reinforced retaining structure", *Journal of Performance of Construction Facilities*, ASCE, Vol. 8, No. 4, pp. 274-292.

Lucia, P.C. and Blair, S.A., 1993, "Geogrid Reinforcement for a Soil Slope. Lawrence Berkeley Laboratory, California, USA", *Geosynthetics Case Histories*, Raymond, G.P. and Giroud, J.P., Eds., BiTech, 1993, pp. 238-239.

Mitchell, J.K., 1991, "Conduction Phenomena: from Theory to Geotechnical Practice", *Géotechnique*, Vol. 41, No. 3, pp. 299-340.

Mitchell, J.K. and Christopher, B.R., 1990, "North American Practice in Reinforced Soil Systems", *Design and Performance of Earth Retaining Structures*, Geotechnical Special Publication No.25, Lambe, P.C. and Hansen, L.A., Eds., ASCE, 1990, Cornell University, NY, pp. 322-346.

Mitchell, R.A. and Mitchell, J.K., 1992, "Stability Evaluation of Waste Landfills", *Stability and Performance of Slopes and Embankments II*, Geotechnical Special Publication No.31, Seed, R.B. and Boulanger, R.W., Eds., ASCE, 1992, Vol. 2, Berkeley, California, USA, pp. 1152-1187.

Mitchell, J.K. and Villet, W.C.B., 1987, "*Reinforcement of Earth Slopes and Embankments*", National Cooperative Highway Research Program Report No.290, Transportation Research Board, 323 p.

Murray, R.T. and Boden, J.B., 1979, "Reinforced Earth Wall Constructed with Cohesive Fill", *Colloque International sur le Renforcement des Sols*, Vol. 2, Paris, France, pp. 569-577.

Nakamura, K., Tamura, Y., Tatsuoka, F., Iwasaki, K. and Yamauchi, H., 1988, "Roles of Facings in Reinforcing Steep Clay Slopes with a Non-Woven Geotextile", *Theory and Practice of Earth Reinforcement*, Yamanouchi, T., Miura, N. and Ochiai, H., Eds., Balkema, 1988, Proceedings of the International Geotechnical Symposium on Theory and practice of Earth Reinforcement, Fukuoka Kyushu, Japan, October 1988, pp. 553-558.

O'Reilly, M.P., Boden, D.G. and Johnson, P.E., 1990, "Long-Term Performance of Repairs to Highway Slopes using Geotextiles", *Performance of Reinforced Soil Structures*, McGown, A., Yeo, K., and Andrawes, K.Z., Eds., Thomas Telford, 1991, Proceedings of the International Reinforced Soil Conference held in Glasgow, Scotland, September 1990, pp. 159-162.

Perrier, H., Blivet, J.C., and Khay, M., 1986, "Stabilization de Talus par Renforcement tout Textile: Ouvranges Experimental et Reel", *Proceedings of the Third International Conference on Geotextiles*, 1986, Vol. 2, Vienna, Austria, April 1986, pp. 313-318. (in French)

Puig, J. and Blivet, J.C., 1973, "Remblai à Talus Vertical Armé avec un Textile Synthétique", *Bulletin de liaison des Laboratoires des Ponts et Chaussées*, Vol. 64, Paris, France, pp. 85-90. (in French)

Puig, J., Blivet, J.C. and Pasquet, P., 1977, "Remblai Armé avec un Textile Synthétique", *Proceedings of the International Conference on the Use of Fabrics in Geotechnics*, Paris, France, pp. 85-90. (in French)

Scott, J.D., Sego, D.C., Hofmann, B.A., Richards, E.A. and Burch, E.R., 1987, "Design of the Devon Geogrid Test Fill", *Proceedings of Geosynthetics '87*, IFAI, 1987, Vol. 1, New Orleans, LA, USA, February 1987, pp. 157-168.

Sego, D.C., Scott, E.A., Richards, E.A. and Liu, Y., 1990, "Performance of a Geogrid in a Cohesive Soil Test Embankment", *Proceedings of Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Balkema, 1990, Vol. 1, The Hague, Netherlands, May 1990, pp. 67-72.

Stewart, J., Seed, R.B., Riemer, M. and Zornberg, J.G., 1994, "Geotechnical Aspects of the Northrige Earthquake of January 17, 1994 - Geotechnical Structures", *Geotechnical News*, June 1994, pp. 59-62.

Tatsuoka, F., Murata, O., Tateyama, M., Nakamura, K., Tamura, Y., Ling, H.I., Iwasaki, K. and Yamauchi, H., 1990, "Reinforcing Steep Clay Slopes with a Non-woven Geotextile", *Performance of Reinforced Soil Structures*, McGown, A., Yeo, K. and Andrawes, K.Z., Eds., Thomas Telford, 1991, Proceedings of the International Reinforced Soil Conference held in Glasgow, Scotland, September 1990, pp. 141-146.

Tatsuoka, F., Nakamura, K., Iwasaki, K. and Yamauchi, H., 1987, "Behavior of Steep Clay Embankments Reinforced with a Non-Woven Geotextile Having various Face Structures", *Proceedings of Post Vienna Conference on Geotextiles*, Singapore, pp. 387-403.

Tatsuoka, F. and Yamauchi, H., 1986, "A Reinforcing Method for Steep Clay Slopes using a Non-woven Geotextile", *Geotextiles and Geomembranes*, Vol. 4, Nos. 3-4, pp. 241-268.

Temporal, J., Craig, A.H., Harris, D.H. and Brady, K.C., 1989, "The use of locally available fills for reinforced and anchored earth", *Proceedings of the Twelfth International Conference on Soil Mechanics and Foundation Engineering*, Balkema, 1992, Vol. 2, Rio de Janeiro, Brazil, August 1989, pp. 1315-1320.

The Reinforced Earth Company, 1990, *"Seismic Design of Reinforced Earth Retaining Walls and Bridge Abutments*", 12 p.

Toriihara, M., Matsumoto, S. and Hirama K., 1992, "Construction and Measurement of Embankment Reinforced with Geogrid using In-situ Cohesive Soil", *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Tatsuoka, F. and Leshchinsky, D., Eds., Balkema, 1994, Proceedings of Seiken Symposium No. 11, Tokyo, Japan, November 1992, pp. 295-299.

Wang, Y.H. and Wang, M.C., 1993, "Internal Stability of Reinforced Soil Retaining Structures with Cohesive Backfills", *Transportation Research Record* 1414, pp. 38-48.

Werner, G. and Resl, S., 1986, "Stability Mechanism in Geotextile Reinforced Earth Structures", *Proceedings of the Third International Conference on Geotextiles*, 1986, Vol. 2, Vienna, Austria, April 1986, pp. 465-469.

Wichter, L., Risseeuw, P. and Gay, G., 1986, "Large Scale Test on the Bearing Behavior of a Woven Reinforced Earth Wall", *Proceedings of the Third International Conference on Geotextiles,* 1986, Vol. 2, Vienna, Austria, April 1986, pp. 301-306.

Wu, J.T.H., 1991, "Measured Behavior of the Denver Walls", *Geosynthetic-Reinforced Soil Retaining Walls,* Wu, J.T.H., Ed., Balkema, 1992, Proceedings of the International Symposium on Geosynthetic-Reinforced Soil Retaining Walls, Denver, CO, USA, August 1991, pp.31-41.

Yamanouchi, T., Miura, N., Matsubayashi, N. and Fukuda, N., 1982, "Soil Improvement with Quicklime and Filter Fabric", *Journal of the Geotechnical Engineering Division,* ASCE, Vol. 108, No. GT7, pp. 953-965.

Yamauchi, H., Tatsuoka, F., Nakamura, K. and Iwasaki, K., 1987, "Stability of Steep Clay Embankments Reinforced with a Non-Woven Geotextile", *Proceedings of the Post Vienna Conference on Geotextiles*, Singapore, pp. 370-386.

Yunoki, Y. and Nagao, A., 1988, "An Application of Non-Woven Fabrics to Embankment of Cohesive Soil", *Theory and Practice of Earth Reinforcement*, Yamanouchi, T., Miura, N. and Ochiai, H., Eds., Balkema, 1988, Proceedings of the International Geotechnical Symposium on Theory and practice of Earth Reinforcement, Fukuoka Kyushu, Japan, October 1988, pp. 491-496.

Zornberg, J.G. and Mitchell, J.K., 1992, "*Poorly Draining Backfills for Reinforced Soil Structures - A State of the Art Review",* Geotechnical Research Report No. UCB/GT/92-10, Department of Civil Engineering, University of California, Berkeley, CA, USA, 101 p.

Zornberg, J.G. and Mitchell, J.K., 1994, "Reinforced Soil Structures with Poorly Draining Backfills. Part I: Reinforcement Interactions and Functions", *Geosynthetics International*, Vol. 1, No. 2, pp. 103-148.

Table 1. Reduced-scale reinforced soil models constructed using poorly draining backfills.

Research agency	Test type	Soil	Reinforcement	Characteristics	Observed behavior	Conclusions	Reference
TRRL, U.K.	Experimental wall	Silty clay	Glass reinforced plastic strips	3 m high model with hexagonal facing panels	Good vertical profile was obtained by providing temporary support of facing units	Tension in the reinforcements appear to be highly influenced by compaction procedures	Boden et al. 1978
Ground Eng. Ltd., U.K.	Small-scale reinforced clay walls	Kaolin clay	Polyethylene mesh	Walls were failed by application of vertical surcharge	Surcharge load at failure increased linearly with the number of reinforcement layers	Wall performance could be explained by total stress analysis	Ingold 1981
Geotextile Consultants Ltd., U.K.	Reinforced clay cube, reinforced foundations	Remolded London clay, kaolin	Plastic geogrid	Undrained plane-strain conditions were simulated	Tests on reinforced clay showed reasonable agreement with proposed theory to model plane strain compression	Reinforcements impart an equivalent undrained shear strength higher than the clay shear strength	Ingold & Miller 1982
Queensland Inst. of Technology, Australia	Large geotextile reinforced clay wall models	Silty clay, basically kaolinite	Nonwoven needle-punched geotextile	Uniformly distributed & discrete strip loads were applied. Soil moisture content was 19%	Vertical surcharge to failure increased nearly two times with geotextile reinforcement	Geotextile reinforcement laterally confined the wall models developing tensile stresses in reinforcement. Geotextile strains were small	Fabian and Fourie 1988
Univ. of California, Berkeley	Centrifuge tests	Low plasticity clay	Nonwoven geotextile, plastic strips	Walls were 15 cm high	Vertical cracks appeared 10 to 12 cm behind the wall facing	Reinforcement improved wall stability. Further research is needed to fully explain test behavior	Jaber 1989

TRRL, U.K.	Four half-scale embankments	Overconsolidated London Clay	Geogrid	Applied surcharge loading was monitored	Deformations on the unreinforced embankment were high. Reinforcement significantly improved the ultimate strength	Reinforcements forced the redistribution of strains such that the development of the slip surface was inhibited	Irvin et al. 1990
Univ. of Maryland	Centrifuge tests	Kaolin, kaolin-sand mix, natural soils	Nonwoven geotextile	Reinforcement spacing and length varied	Lightly reinforced models failed by overturning. Heavily reinforced models developed a sliding failure	In all tested soils reinforcement had a significant beneficial effect	Goodings 1990
Dames & Moore, Darwin	Reinforced clay wall models	Silty clay	Nonwoven geotextile	CU loading conditions were simulated	Nonwoven geotextile effectively drained the clay backfill. Long-term deformability of reinforced clay was less critical than that of unreinforced clay	High transmissivity geotextile increased undrained strength of the cohesive backfill. Time required for primary consolidation is reduced	Fabian 1990
Univ. of Colorado at Denver	Reinforced clay wall	Sand and clay mixture	Nonwoven geotextile	Wall was 3 m high with timber facing	Overall shear failure was not attained at the maximum surcharge pressure of 234 kPa	Observed wall movements were smaller in the clay wall than in a sand wall of similar dimensions	Wu 1991
Univ. of Maryland	Centrifuge models	Lime stabilized kaolin	Nonwoven geotextile	Various reinforcement lengths were used	Three failure modes were identified depending on the reinforcement length	Lime improved wall stability substantially even with short geotextile length	Güler and Goodings 1992

Note: CU = consolidated undrained

Table 2. Full-scale reinforced soil structures constructed using poorly draining backfills.

Name	Location	Date	Structure	Height (m)	Reinforcing method	Backfill	Facing	Construction	Comments	Reference
Autoroute A15	Rouen, France	1971	Highway embankment wall	4.0	Nonwoven geotextile	Weathered chalk, silt, and fire stone	Wrapped vertical	Berm on passive side partly removed after construction	First geotextile-reinforced wall. Unprotected geotextile facing. Satisfactory behavior	Puig and Blivet 1973; Puig et al. 1977
Illinois River Wall	Oregon USA	1974	Reinforced soil wall	3.5	Nonwoven, geotextile	Silty sand and angular gravel	Gunnite facing	0.3 m geotextile spacing at top, and 0.22 m at the base	First full-scale geotextile reinforced soil wall in U.S.	Bell and Steward 1977
Barrage de Maraval	Pierrefeu, France	1976	Dam spillway weir	6.5	Polyester woven geotextile	Compacted clay and schist (gravel at face)	Wrapped resin coated	Vertical face made up with polyester woven bags filled with loam	Withstood three overtoppings before end of construction without damage	Kern 1977
TRRL Experimental wall	Crowthorne, U.K.	1978	Reinforced soil wall	6.0	Several steel and plastic strips	Sandy clay, sand, silty clay	Facing panels	Vertical reinforcement spacing was 0.5 m	High pore water pressures developed in clay fill during construction, causing large deformations	Boden et al. 1978; Murray and Boden 1979
Yokohama residential complex	Tokyo, Japan	1978	Reinforced Retaining Wall	8.7	Metal strips	Volcanic clay	Facing panels	Reinforcement tension was monitored	Final settlements of up to 91 cm were observed	Hashimoto 1979
Industrial structure	U.K.	?	Industrial structure	3.2	Plastic geogrid	Mine waste	Rigid facing	Structure built on a mine waste tip	Plastic reinforcement exhibits low creep and high strength	Jewell and Jones 1981

Railway engine spur	U.K.	?	Mine waste reinforced structure	6.0	Glass fibre reinforced plastic strips	Mine waste	Rigid facing	A rigid full-height facing was used	Reinforcements were connected to facing with sliding connections	Jewell and Jones 1981
Shimonoseki Sanitary Facility	Shimonoseki, Japan	1979	Embankment	32.0	Multiple strip-sandwich method	Cohesive soil of strongly weathered tuff	Crib retaining wall	Quicklime layers placed in triangular configuration	Water content decreased about 7% after ten months. No significant movement during heavy rains	Yamanouchi et al. 1982
Virginia wall	Virginia USA	1978-79	Reinforced wall	up to 7.0	Ribbed steel strips	Residual low-plasticity silts	Concrete panels	Tilting 250 to 300 mm out of plumb occurred after precipitations	Areas of backfill with more than 25% fines were excavated and replaced with selected backfill	Elias and Swanson 1983
Highway wall at Koper	Slovenia	1982	Reinforced wall	3.5	Polyester strips	clayey silt	Reinforced concrete panels	A 50 cm wide sand drain was built along the facing panels	Facing lateral displacements reached 40 mm 152 days after end of construction	Battelino 1983
Interstate 80	Baxter, California USA	1982	Four embankment walls	5.0	Bar mat	Silt 49% passing #200 sieve	Prefabricated concrete facing	Construction forced to stop due to rainfalls	Extensive instrumentation showed no significant wall movements	Hannon and Forsyth 1984
Test Embankment I	Univ. of Tokyo, Japan	1982	Clay embankment	4.0	Polypropylene nonwoven geotextile	Volcanic Ash Clay (Kanto loam)	Geotextile sheet	One of the embankment sides had larger geotextile vertical spacing	The slope with the larger vertical spacing (80 cm) moved considerably	Tatsuoka and Yamauchi 1986
Test Embankment II	Univ. of Tokyo, Japan	1984	Clay embankment	5.2	Nonwoven geotextile	Volcanic Ash Clay (Kanto Loam)	Gabions at the face	One of the embankment sides had smaller geotextile length	Steep clay slopes reinforced with short geotextile sheets were stable during heavy artificial rainfall	Tatsuoka and Yamauchi 1986

Table 2. Continued.

Name	Location	Date	Structure	Height (m)	Reinforcing method	Backfill	Facing	Construction	Comments	Reference
Chemie Linz embankment	Austria	1984	Reinforced embankment	2.5	Nonwoven geotextile	Silty sand	Geotextile facing	Embankment exposed 3 years before loading	Embankment did not fail when loaded up to 1.7 times the theoretical failure load. No evidence of geotextile creep	Werner and Resl 1986
Otto-Graff-Institute reinforced wall	Germany	?	Geotextile reinforced wall	4.8	Polyester fabric	Weathered marl	Geotextile facing	Unit weight of 2.0 t/m^3 reached by compaction	In spite of the high loading, wall failure was not reached	Wichter et al. 1986
LCPC Experimental Embankment	Rouen, France	1984	Geotextile-reinforced embankment	6.4	Three woven geotextiles; one nonwoven/grid composite	Silt, compacted 5% wet of optimum	Geotextile, geotextile gabions	Four sections with different geotextiles. Vertical spacing was 0.8 m	Positive pore water pressures generated in sections reinforced with woven geotextiles. Negative pore pressures recorded in composite sections	Perrier et al. 1986
Kami-Onda Experimental Embankment	Yokohama city, Japan	1985	Clay embankment	5.4	Polypropylene nonwoven geotextile	Kanto Loam Volcanic Ash Clay	Concrete panels, gabions	1V:0.2H slope with 0.5 m geotextile vertical spacing	Slope was stable after heavy artificial rainfall	Tatsuoka et al. 1987
Test Embankment III	Univ. of Tokyo, Japan	1986	Clay embankment	5.5	Nonwoven geotextile	Kanto Loam Volcanic Ash Clay	Different facing systems	Longer reinforcements were used at the embankment base	Facing system should provide local rigidity	Tatsuoka et al. 1987

Devon test fill	Alberta, Canada	1988	Reinforced embankment	12.0	Geogrids	Silty clay (Activity ≈ 1.0)	Secondary and tertiary grids	1V:1H slopes. Embankment was heavily instrumented	Geogrid strains are in direct response to both horizontal and vertical deformations in the embankment	Scott et al. 1987; Sego et al. 1990
Interstate 580 Wall	Hayward, California USA	1982	Vertical faced wall	1.8 to 9.1	Welded wire mesh	Sandy clay with potential expansibility	Facing panels	There was poor drainage of surface water	Wall showed excessive movement and cracking	Mitchell and Villet 1987
Ashigara parking area	Tomei way, Japan	1988	Fill slope	20.0	Nonwoven fabric	Soft loam	No structural facing	Geometric constraints imposed a 1V:1.8H slope	Soil strength increase by consolidation was taken into account in the analysis	Yunoki and Nagao 1988
Paulsgrove experimental wall	Hampshire, U.K.	1985	Experimental wall	5.6	Steel strips	Three types of local chalk	Concrete facing panels	Negative pore water pressures were generated during construction	Horizontal wall movements were up to 15 mm 3 months after construction. No later movement	Temporal et al. 1989
JR No.2 Experimental embankment	Japan	1988	Clay embankment	5.0	Nonwoven and composite geotextiles	Kanto loam volcanic ash clay	Continuous rigid facing	Six test segments were constructed	Good performance observed two years after construction	Tatsuoka et al. 1990
AIT Experimental wall	Thailand	?	Experimental wall	5.7	Welded wire mats	Clayey sand, lateritic soil, weathered clay	Vertical wire mesh	Wall was stable. Large settlements and lateral movements occurred	Subsoil movement greatly influenced vertical pressure beneath the wall and reinforcement tensions	Bergado et al. 1991
M4 Yattendon Cutting	U.K.	1980	Reinforced slope	20.0	HDPE mesh	Clay fill with 1% quicklime	No structural facing	Reinforcement vertical spacing was 0.5 m and 1.0 m	Slope and geotextile reinforcement performed well over period of 9 years	O'Reilly et al. 1990

Table 2. Continued.

Name	Location	Date	Structure	Height (m)	Reinforcing method	Backfill	Facing	Construction	Comments	Reference
A45 Cambridge Northern Bypass	U.K.	1983	Reinforced embankment	7.0	Polypropylene geogrid	Gault Clay	No structural facing	Slope was 1V:2H	Good performance after 6 years. Recovered geotextiles showed no degradation	O'Reilly et al. 1990
Annan Bypass Retaining Wall	U.K.	1989	Retaining wall	23.0	Concrete half discs used as anchors	Clayey till	Facing panels	Anchors were connected to facing polymeric straps	Pore pressures during construction ranged between -1 and +1 m head of water	Brady and Masterton 1990
Calgary parking lot	Alberta, Canada	1984	Reinforced retaining wall	9.0	Geogrid	Low plastic clay till	H-pile and timber	Upper 6 m of wall was replaced three years after construction	Wall suffered distress due to saturation of the backfill	Burwash and Frost 1991
Cannon Creek embankment	Arkansas USA	1988	Highway embankment	23.2	Geogrid	Highly plastic and expansive clay	Intermediate geogrids	Long-term loading conditions governed the design	Good performance was observed during the first 24 months of service	Hayden et al. 1991
Experimental wall of Lezat	France	1992	Experimental wall	6.0	Woven/ nonwoven composite	Silt	LCPC patented facing	Geotextile strains were measured during construction	Experimental wall will be saturated until failure	Delmas et al. 1992
Reinforced slopes	Taiwan	?	Three reinforced slopes	Up to 10 m	Geogrid	Clayey silt	No structural facing	Failure by reinforcement breakage, pullout, and overall sliding was reported	Only qualitative description of failure mechanisms was given	Huang 1992

Waste disposal facility	Japan	?	1:1 reinforced slope	Up to 25 m	Geogrid	Cohesive soil	No structural facing	Cohesive soil was cement stabilized	Geogrid strains and slope displacements were measured during construction	Toriihara et al. 1992
Hengyang wall	China	1988	Retaining wall	Up to 6.83 m	Polypropylene strips	Silty clay	Concrete blocks	Soil was compacted to 95 % Standard Proctor	Vertical pressures at the base were bilinear, increasing and then decreasing from the face to the back of the wall.	Wang and Wang 1993
Pingshi wall	China	1988	Retaining walls	Up to 10 m	Polypropylene strips	Local cohesive soils	Concrete blocks	Soil was compacted to 95 % Standard Proctor	Lateral earth pressure coefficient decreased with depth from a maximum at the top of backfill	Wang and Wang 1993
Waterworks Corner Slope	London, U. K.	1986	Reinforced slope	8.0	Geogrid	Overconsolidated clay	Secondary geogrid	A 300 mm thick granular drainage layer was built	No movements have been noted since construction	Dixon 1993
Lawrence Berkeley Laboratory slope	Berkeley, California USA	1986	Geogrid reinforced slope	24.0	Geogrid	Silty clays, clayey sandy gravels	Intermediate geogrid	Soil used as backfill was more cohesive than assumed in the initial design	Reinforced slope has performed as intended	Lucia and Blair 1993
Barren River Plaza Shopping Center	Glasgow, Kentucky, USA	1990	Reinforced wall	3.0 to 6.4	Geogrid	Cohesive soil	Keystone block facing	Clay backfill was poorly compacted. Geogrid layers were misplaced/omitted during construction	The structure failed. Deficiencies in design and construction quality control explain the observed modes of failure	Leonards et al. 1994