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Keynote lecture: Issues in geosynthetic-reinforced soil

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ABSTRACT: Some issues related to geosynthetic-reinforced soil, covering a wide range of applications, are discussed. Generic issues discussed are associated with the mathematical and physical modelling of reinforced soil structures. Issues related to specific structures are also presented: 1. Reinforced embankments over soft soil: monitoring of construction using strain gages and evaluation procedures for required seam strength, 2. Narrow barrier walls: estimation of lateral earth pressures for design, and 3. Geotextile tubes: fabrication and design considering the type of hydraulic fill. Description of the construction of barrier walls and of the performance of tubes, hydraulically filled with clayey slurry, is detailed to help in clarifying the issues. Resolutions to some of the issues are suggested. Areas where further research is needed to resolve the issues are pointed out.

1 INTRODUCTION

Geosynthetics have gained acceptance as reinforcing materials in a variety of earth structures, many of which are critical. This acceptance has evolved rapidly, although geotechnical engineering is an understandably conservative discipline. Partially, it is because of the design tools that have been developed based on sound geotechnical engineering and on proven performance. It is also due to the available variety of properties of this engineered material. Geosynthetics can be made strong under tension, typically ductile, easy to handle and to construct with, and, depending on the type of the polymer, inert and durable in many applications. Consequently, the use of geosynthetics as soil reinforcement is attractive, allowing flexibility in design. The main factor in propagating the use of geosynthetics, however, is its economics. In many cases, use of geosynthetics offers a cost-effective alternative to other solutions.

Several issues, all related (directly or indirectly) to design, still remain, however, and some are raised in this paper. These issues should not infer lack of confidence in using geosynthetics but rather highlight some topics that need further research, clarification, and possibly modification. In fact, areas in geotechnical engineering that are more established than geosynthetic reinforcement are constantly being modified (e.g., slope stability, deep and shallow foundations, ground water flow). Yet, existing design procedures in these areas are routinely and safely being employed in major projects. The issues in geosynthetic reinforcement should, perhaps, be viewed in a similar context.

The issues raised herein vary widely. They cover generic topics such as those related to mathematical and physical modelling of reinforced soil structures. Also covered are topics associated with specific applications. These include the use of strain gages in monitoring the construction of embankments over soft soil (quality-control) and the design value of seam strength (the weak link) in high strength geotextiles. Also discussed are two types of innovative and very little tested structures, thus expanding the scope of this paper. Though these type of structures may not be very visible to the public, their use is likely to increase and become significant. In addition to general issues regarding these structures, some details about their

construction and observed performance are also given. These details place the issues in a proper perspective.

2 MODELLING

Limit equilibrium analysis has been used in the design of earth structures for about 70 years. Its extension to the design of geosynthetic-reinforced soil structures is no surprise. The attractive feature of this analysis is its relatively simple input data. This leads to limited but useful output design information. Furthermore, the modelling of reinforcement can intuitively be comprehended. Practitioners can assess the reasonableness of the results of this analysis based on their experience and through simplified charts or hand calculations. However, there are several unresolved issues regarding limit equilibrium analysis. Some of its results appear paradoxical in a framework of a parametric study. Luckily, these results typically exhibit conservative trends.

The most sophisticated analytical modelling approach is the finite element analysis. It can account for important factors associated with reinforced soil and, typically, it provides information in excess of what is needed for ordinary design. A hinderance in its routine use, however, is the extensive input data required to produce reliable results. It is argued that proper finite element modelling of a problem is not trivial. If this modelling is inadequate, unsafe predictions may be rendered.

The alternative to analytical modelling of a problem is testing of physical models. It is claimed that generalizations stemming out of such models should be made carefully. Small scale models do not necessarily duplicate the behavior of full-scale prototypes. Alternatively, to conduct carefully-controlled full-scale tests, special constraints are imposed by the testing facility (e.g., special facing, testing container of very limited width, etc.). Consequently, when extrapolating from full-scale tests to design, one must account for possibly different field conditions.

2.1 Analysis: Limit Equilibrium

The design of most geosynthetic-reinforced soil structures is based on limiting equilibrium (LE) analysis. This is in concert with established design-practice in geotechnical engineering where the margin of safety of earth structures against collapse is routinely assessed. Seemingly, the inclusion of reinforcement in LE analysis is straightforward. That is, for a test-body assumed to be at the verge of failure, the reinforcement forces are integrated into the limiting equilibrium equations and the global factor of safety then is calculated. After examination of many such test-bodies, the required layout and strength of the reinforcement is determined so that a design safety factor can be attained. Although LE analysis of reinforced soil has progressed significantly in the past 15 years, there are still a few issues that need clarification.

Several different definitions of the factor of safety are currently being used. The one that is consistent with conventional LE analysis, for which design guides and extensive experience have evolved over many years, is defined with respect to the shear strength of the soil:

$$F_s = \frac{\textit{available shear strength of soils}}{\textit{shear strength required for equilibrium}}$$

Subsequently, the design values of shear strength parameters (corresponding to Mohr-Coulomb failure criterion) are $c_d = c/F_s$ and $\phi_d = tan^{-1}[tan(\phi)/F_s]$. For these design parameters, the required reinforcement (i.e., strength and layout) is calculated so that for the most critical feasible case, the structure is at a state of limit equilibrium. The required reinforcement strength then is multiplied by a variety of relevant partial safety factors (e.g., creep, installation damage, aging, uncertainties, etc.) to select an adequate geosynthetic. Since all soil strata are not likely to reach their full strengths simultaneously, it is recommended to use design values (c_d, ϕ_d) that do not exceed the residual strength within each layer. Typically, a F_s of 1.3 applied to peak strength will assure a residual value or less.

Material stress-strain relationships play no role in the LE analysis. However, the analysis implicitly assumes that the geosynthetic and soils will supply their design strengths at the

same instant, although this may depend on the deformation characteristics of the materials. For example, if the reinforcement is very stiff relative to the soil, its strength will be mobilized rapidly, potentially reaching its full design strength before the soil does. This may lead to a premature rupture of the reinforcement, violating the design assumptions and resulting in a global collapse. Selecting a ductile reinforcement (i.e., a typical geosynthetic) together with the residual strengths of soils will assure that if needed, all materials can contribute their design strengths simultaneously, at some unspecified level of structural deformation. The design outcome then is safe against collapse but may be unacceptable deformation-wise.

When including the reinforcement force in the LE formulation, one must assume its inclination. Physically, this inclination may vary between the as-installed (typically horizontal) and parallel to the shear surface. Leshchinsky and Boedeker (1989), Leshchinsky and Perry (1989), and Wright and Duncan (1991) have shown that when granular steep slopes are considered, the inclination has negligible effects on both layout and strength of the reinforcement, as long as a rigorous slope stability analysis is used. However, when cohesive foundation soil is involved, Leshchinsky and Smith (1988a, 1988b) have demonstrated that the assumed inclination may be quite critical. That is, the horizontal inclination may render a required strength of geosynthetic as much as 50% greater than the other extreme. Also, the required layout varies due to different locations of critical slip surfaces. Consequently, in cohesive soils it is recommended to analyze based on the two extreme inclinations, using in each case a different factor of safety. For example, when short-term stability is most critical (e.g., end-of-construction) use F_s equals 1.2 and 1.5 for the horizontal and tangential inclinations, respectively. The stricter values of strength and layout may then be specified in design.

Most available computer codes (e.g., UTEXAS2--Wright and Duncan 1991; STABGM--Duncan et al. 1985) are oriented towards analysis rather then towards design. That is, the reinforcement force is specified implying, *de facto*, that it is an active force and not a reactive one, regardless of the problem. Typically, this force is taken as the allowable tensile resistance of the reinforcement or, if there are layers of reinforcement, anywhere between the allowable maximum value, to a linear distribution with maximum value at the lowest elevation. The resulting F_s then will depend greatly on the assumed active tensile forces and may allow the designer to select geosynthetics with permissible strengths varying between t and $2t$. Such a wide range poses a problem even for the experienced designer. Leshchinsky et al. (1986) suggested a distribution based on a rigid body rotation of a log spiral mechanism. The required tensile strength to attain a prescribed F_s can then be rationally calculated. The results suggested that the tensile reaction in the upper layer is about 60 to 70% of the bottom layer force when granular soil is considered and the reinforcement is uniformly spaced from crest to toe. For cohesive soil, all layers are equally stretched. However, while rigid body motion is consistent with LE analysis, it may overestimate the reinforcement force reaction, potentially resulting in unsafe values.

Considering the frequent use and limitations of LE analysis together with the current state of practice, the following rationale and procedure is recommended for design. Use an analysis that indeed satisfies limit equilibrium equations; i.e., a rigorous analysis. This analysis, for example, can be an extension of Spencer's method (Wright and Duncan 1991) employing a slip surface of general shape. If homogeneous earth structures are considered, the log spiral mechanism is easier to employ. In this case the problem is statically determinate (e.g., Leshchinsky and Boedeker 1989). For the sake of clarity, the presentation hereafter is limited to log spirals and homogeneous, simple slopes--see Figure 1. Extension to complex problems, using a generalized slope stability analysis, is, in principle, straightforward. It is assumed that the reinforcement force will be activated in response to an unstable soil mass possessing c_d and ϕ_d. In slopes it will be activated when the inclination is so steep that slide is likely to develop (e.g., slope steeper than ϕ_d if granular soil is considered); in embankments it will be activated when deep-seated failure is likely to develop through the soft foundation (the logic for this case, which is equivalent, in a sense, to steep slopes, was presented by Leshchinsky 1987, and Leshchinsky and Smith 1988b).

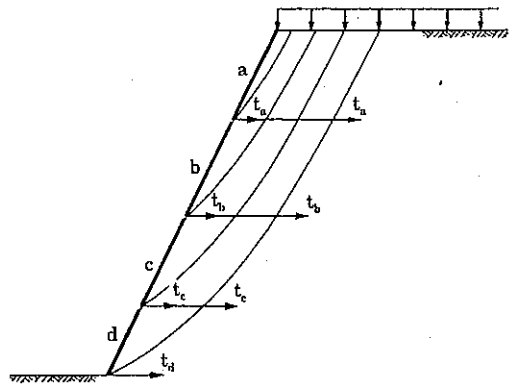

Fig. 1. Mechanism to determine reaction force in reinforcement.

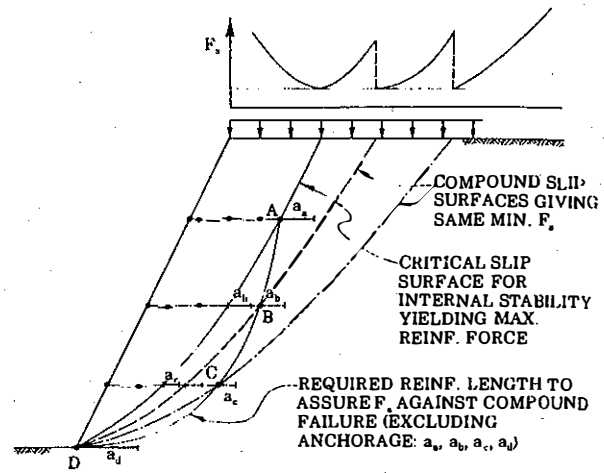

Fig. 2. Approach to assure F_s against compound failure.

Referring to Fig. 1, consider the soil mass leaning against facial unit a. This mass is restrained by the reinforcement reaction t_a. The force t_a can be calculated through a search of a log spiral that produces the maximum reaction. Next, the force acting against unit b is counterbalanced by t_b. In calculating t_b one considers t_a as a known value carrying back the force from facing a and acting at the new slip surface. Once again, the maximum value of t_b is sought through a variation of the log spiral. Repeating this process, one obtains the distribution of reactive forces t_a, t_b, t_c, etc., all supplying the demand for the LE state. Similar rationale was presented by Jewell (1990) and Leshchinsky and Boedeker (1991). Leshchinsky and Boedeker (1989) considered the outer-most log spiral as a boundary between stable soil and an active soil mass. Subsequently, they suggested to embed the geosynthetic in the stable zone so that sufficient pullout resistance (t_a, t_b, etc.) could be attained.

The Leshchinsky and Boedeker (1989) approach is consistent with a tie-back analysis. However, as pointed out by Schmertmann (1991) and Jewell (1990), the resulting layout may not be safe if examined using a conventional slope stability approach; i.e., there are potential slip surfaces extending outside the reinforced zone yielding unacceptable factors of safety. Figure 2 illustrates such compound surfaces, and indicates the analytical procedure to remedy the situation. Log spirals to the right of the previous critical slip surface are sought. The first spiral to the right includes anchorage lengths a_b, a_c and a_d; however a_a is disregarded. Hence, in case of pullout/rupture (i.e., at a LE state) t_b (disregarded at B and beyond), t_c and t_d will develop and therefore, F_s can rigorously be calculated. A surface for which F_s is the same as the internal one, but without t_a and at an extent where t_b is just not needed, is searched for, producing point B. Next, a second log spiral to the right, without t_a and t_b and at an extent where t_c is just not needed, is sought so that F_s will, once again, equal the same value as the internal one. Point C is then located. This process is repeated down to the second layer. The curve $ABCD$ defines the required reinforcement lengths to assure a minimal value of F_s for all possible rotational failures. To simplify calculations (i.e., to avoid consideration of surfaces where a fraction of the pullout resistance is available), the anchorage lengths a_b and a_c are added beyond points B and C, respectively. This simplification is conservative since it "assures" t_b at B and to the left, and t_c at C and to the left. However, since anchorage length for continuous geosynthetic sheets is small relative to its total required length in real design problems, this simplification is reasonably conservative. This approach was suggested by Jewell and Leshchinsky (1991), simplifying Jewell's (1990) more "correct" approach in which the "precise" length of each layer required to yield a given F_s, was calculated.

The anchorage length is calculated to resist the pullout force (i.e., t_a, t_b, etc.) considering the overburden pressure and the parameters defining the interaction soil-geosynthetics.

These parameters are analogous to (c_d, ϕ_d). They are related to (c_d, ϕ_d) through an experimentally determined interaction coefficient C_i. This coefficient is typically determined from pullout tests. However, considering difficulties associated with interpretation of this test results (e.g., how one accounts for progressive failure along the non-uniformly elongating geosynthetic?), it is perhaps simpler to determine C_i from direct shear tests.

Specifying a layout that satisfies a prescribed F_s for rotational failure does not guarantee sufficient resistance against direct slide. This can be determined by using the two-wedge method as proposed by Schmertmann et al. (1987). The required base length, L_{ds}, is calculated so as to assure sufficient resistance against the inter-wedge force P--see Figure 3. The force P is maximum with respect to the inclination θ of the base of Wedge A. The trial-and-error process associated with the two-wedge method satisfies only force equilibrium and is a direct extension of existing practice in evaluating external stability of gravity retaining walls. Note, however, that $\delta=\phi_d$ implying the retained backfill will either settle relative to the reinforced soil above L_{ds} or the reinforced zone will slide slightly as a monolithic block thus allowing δ to develop. Also note in Fig. 3 the term C_{ds}; it signifies the geosynthetic-soil interaction coefficient as determined from a direct shear test. It should be pointed out that if a rigorous slope stability analysis, where a general shape slip surface can automatically be located, will be used,

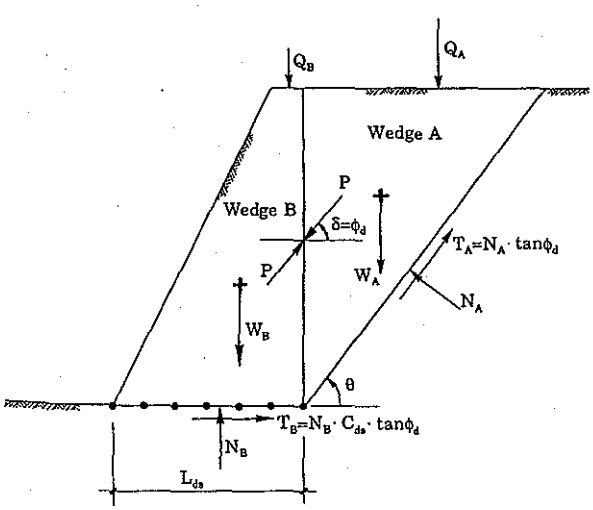

Fig. 3. Mechanism to calculate required L_{ds} to resist direct slide.

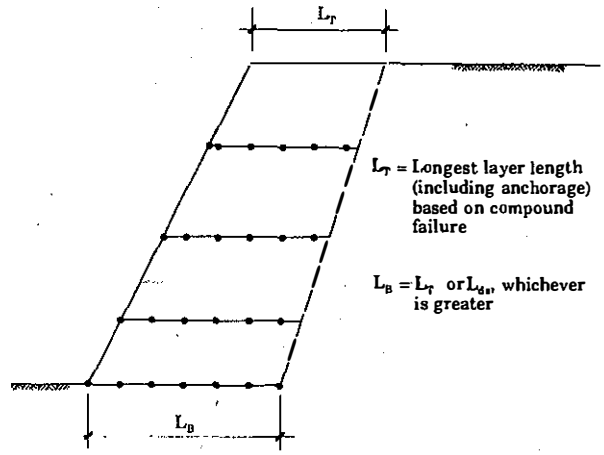

Fig. 4. Practical specification of reinforcement layout.

there will be no need for the direct slide assessment; this mode of potential failure will automatically be tested.

A practical layout may be specified as shown in Fig. 4. The top length, L_T, is based on compound failure; the bottom length, L_B, is either equal to L_T or L_{ds}, whichever is greater. This methodology of layout was suggested by Schmertmann et al. (1987), insuring that all layers equal or exceed their required minimum length while the overall design F_s for the three modes of failure is maintained or exceeded.

It is instructive to conduct a parametric study of the performance of the seemingly reasonable design-oriented analysis. Consider the problem shown in Fig. 5. Ten equally spaced reinforcement layers are used in a 5 m high granular slope. The overall design safety factor, F_s, is 1.3 and ϕ=35°. The soil-geosynthetic interaction coefficients are C_i=C_{ds}=0.8. Three slope inclinations are

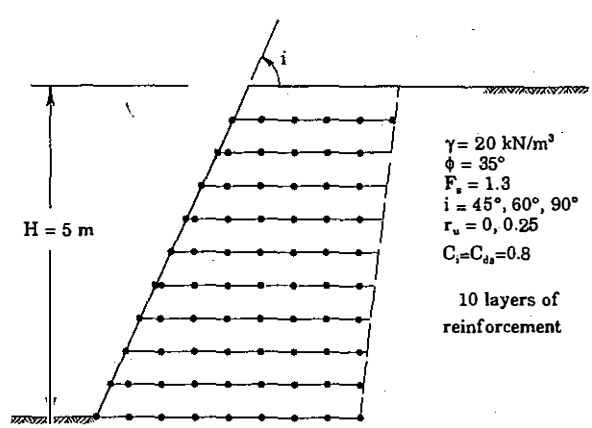

Fig. 5. Data for example problem.

studied (45°, 60° and 90°), each for two different pore-water coefficients (r_u equals 0 and 0.25). The analysis was conducted using a design-oriented computer code (STRATASLOPE) developed by the writer. The geosynthetic forces were assumed to act horizontally. Fig. 6 shows the required reinforcement length to assure F_s=1.3 against compound and internal failure for each of the three slopes at r_u=0. It includes the location of the critical slip surface for internal stability, extending from crest to toe (see also Fig. 1), and the extreme outward compound slip surface (see also Fig. 2). Fig. 7 superimposes all required lengths to resist compound failure (excluding anchorage lengths) at their respective elevations. Fig. 8 represents the required design layout, including lengths to provide anchorage and to resist direct slide, following the scheme shown in Fig. 4. It also shows the traces of critical slip surfaces corresponding to internal stability. Such surfaces are relevant to design based on the tie-back approach. A striking feature appearing in Fig. 8 (or Fig. 7) is that there is little difference in the required area of geosynthetics for either slope inclination. This phenomenon is contrary to intuition where one would expect less reinforcing material as the slope flattens. However, the LE analysis

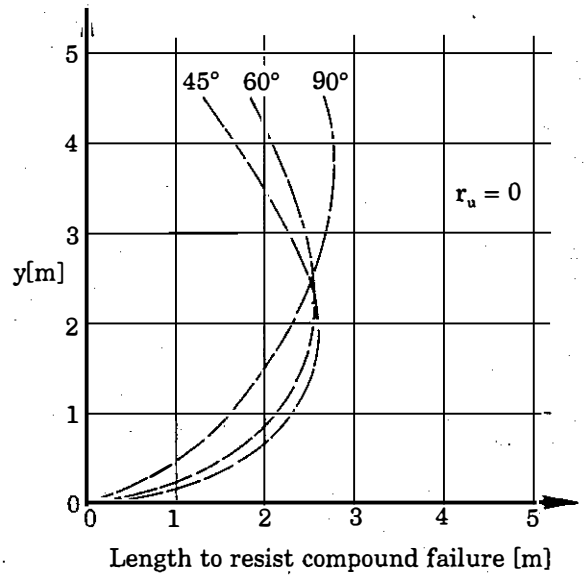

Fig. 7. Lengths of reinforcement at various elevations.

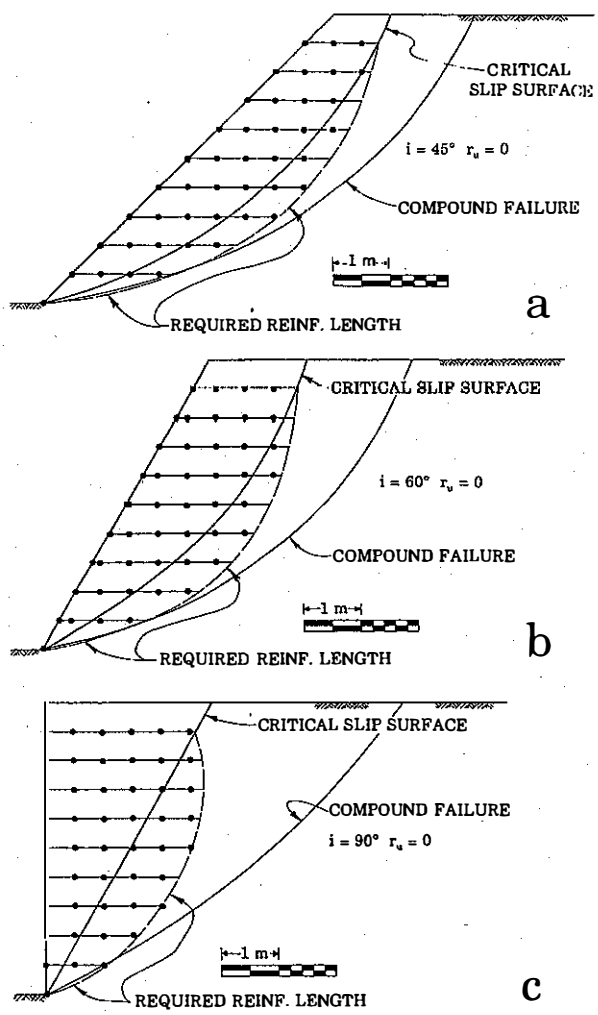

Fig. 6. Reinforcement length to resist compound failure (F_s=1.3): a. *45°*, b. *60°*, and c. *90°*.

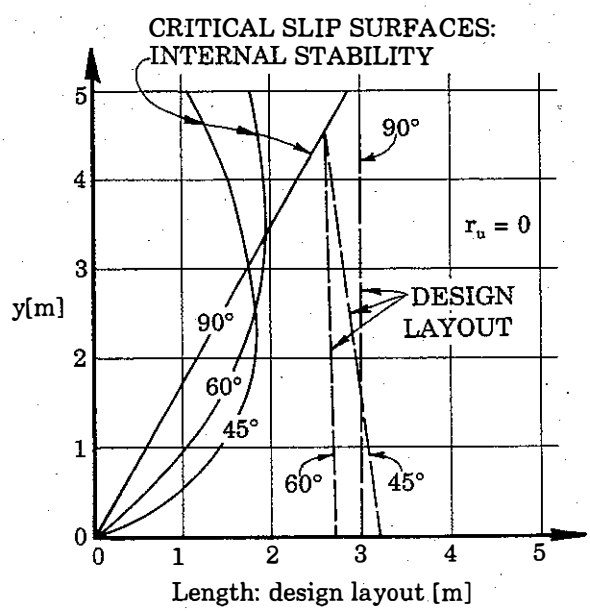

Fig. 8. Practical design layout and trace of critical slip surface at various elevations.

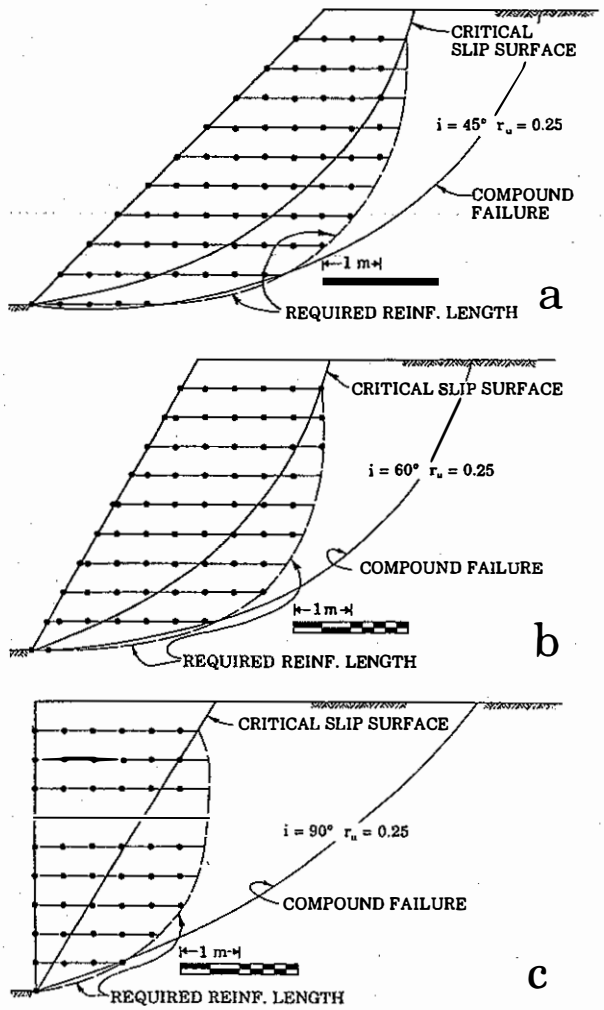

Fig. 9. Reinforcement length to resist compound failure (F_s=1.3): a. *45°*, b. *60°*, and c. *90°*.

indicates that as the slope flattens, the required length (or mass) to resist *direct slide* increases. Consequently, the required length L_B increases while L_T does not change much, thus resulting in an increased overall area of reinforcement. An objective comparison must include the required allowable strength of the reinforcement, a value that is uncoupled from the length but is an outcome of the design. Table 1 provides such a comparison. Clearly, the required strength for the 90° slope is about 4 times the one needed for the 45° slope. In terms of reinforcement area, the 60° slope is the most "efficient." Note that the required area based on tie-back design is consistently about 10 to 12% less than that based on compound failure.

Fig. 9 is equivalent to Fig. 6, but with r_u=0.25. Note that pore-water pressure

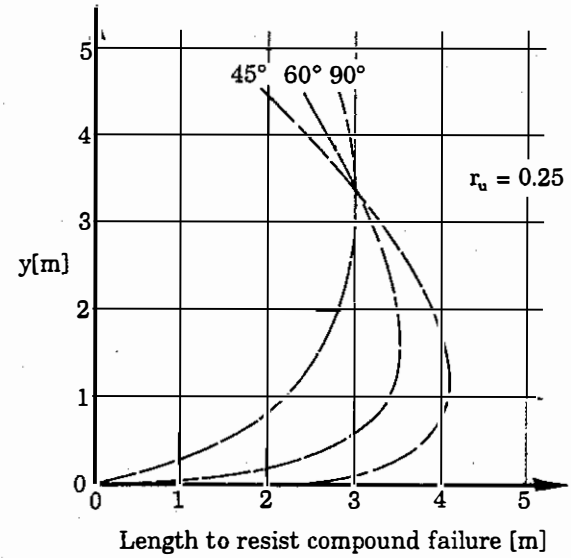

Fig. 10. Lengths of reinforcement at various elevations.

increases the depth and extent of the design-related potential slip surfaces. As the slope flattens, the compound surface penetrates the foundation thus significantly increasing the required reinforcement lengths at lower elevations. This is clearly seen in Fig. 10, which is equivalent to Fig. 7. Fig. 11 (equivalent to Fig. 8) shows that as the slope flattens, the required reinforcement area increases due to both the extent of compound failure, L_T, and the length resisting direct slide, L_B. Table 2 indicates that the required

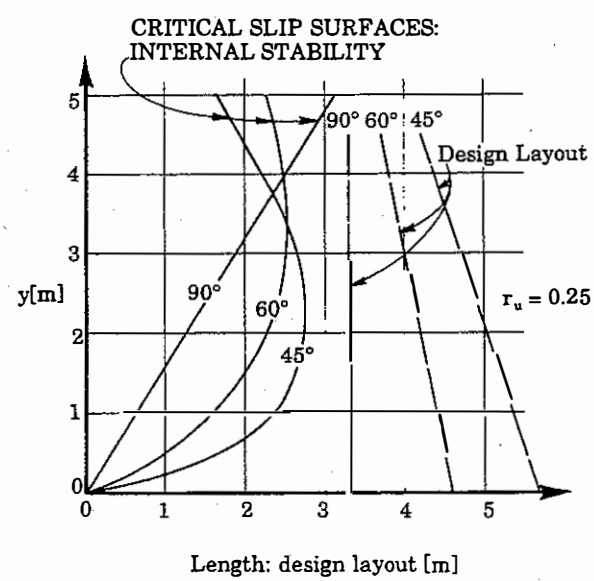

Fig. 11. Practical design layout and trace of critical slip surface at various elevations.

Table 1. LE analysis: design outcome (r_u=0).

i	45°	60°	90°
t_{max} (1) [kN/m]	4.3	7.6	16.9
L_B (2) [m]	3.2	2.7	1.2
L_T (3) [m]	1.9	2.0	2.7
Area (4) [m²/m]	25.5	23.5	27.0
L_T (5) [m]	2.6	2.6	3.0
Area (6) [m²/m]	29.0	26.5	30.0

Table 2. LE analysis: design outcome (r_u=0.25).

i	45°	60°	90°
t_{max} (1) [kN/m]	9.7	13.2	24.6
L_B (2) [m]	5.7	4.6	3.2
L_T (3) [m]	2.9	2.7	3.0
Area (4) [m²/m]	43.0	36.5	31.0
L_T (5) [m]	4.2	3.7	3.3
Area (6) [m²/m]	49.5	41.5	33.0

(1) Required allowable tensile strength of geosynthetic.
(2) Based on direct slide along base (C_{ds}=0.80).
(3) Based on tie-back analysis.
(4) Based on tie-back design approach.
(5) Based on compound failure.
(6) Based on compound failure design approach.

strength for the 90° slope is only about 2.5 times greater than the one for the 45° slope (without pore-water pressure it was 4 times). The percentage difference in required areas, based on tie-back and compound failure designs, remains about the same for all slopes and for r_u=0 and r_u=0.25.

The above example problem demonstrates that although a sensible design approach is combined with a rigorous LE analysis (excluding direct slide), the outcome may be difficult to comprehend when presented in a comparative context. That is, for an increase in pore-water pressure, the required reinforcement area increases as the slope flattens (only up to a certain inclination though). Simultaneously, the difference in required strength decreases. This design outcome implies that as pore-water pressure increases, steeper slopes become more economical reinforcement wise. It should be pointed out that the required length L_T exhibits less of an unreasonable trend based on the tie-back analysis (see Tables 1 and 2).

Reinforced cohesive embankments are increasingly being constructed. Soil mixed with additives, such as lime or cement, is likely to become a common backfill in conjunction with geosynthetic reinforcement. In this case, cohesion becomes a major factor in stability. Therefore, it is worth mentioning that even a small component of cohesion, c_d, in the analysis may have a significant impact on the design outcome, especially if the reinforcement force is specified arbitrarily (i.e., assuming it to be an active force). Consider the problem shown in Fig. 12. For a factor of safety of 1.0, one can show (using Taylor's chart or the log spiral analysis) that the maximum stable height without reinforcement is 3.9 m. The design rationale presented in Figs. 1 through 4 implies that reinforcement is not needed in the upper stable zone (i.e., there is no facial pressure to counterbalance). The design outcome (if spacing is specified at 0.5 m) is shown in Fig. 12; 3 layers, 0.7 m long each (total geosynthetic's area of 2.1 m^2/m), located at the bottom and possessing an allowable strength of 10.1 kN/m. Using the same failure mechanisms but specifying 10 reinforcing layers and assuming an arbitrary tensile resistance force (i.e., taking the reinforcement as an active rather than reactive member), results in the layout shown in Fig. 13. The required allowable strength is 2 or 4 kN/m for an assumed uniform or triangular distribution of tensile resistance, respectively. The reinforcement length is about 3 m and area is 30 m^2/m. Subsequently, the design approach taking the reinforcement as reactive members in a structure requires 2.5 to 5 times stronger reinforcement but 15 times less material as compared to the alternative approach. It should be stated that in case of non-cohesive fill, uniform reinforcement spacing and assumed triangular tensile resistance, the design outcome of the alternative approach will be identical to the one suggested above. However, as the soil

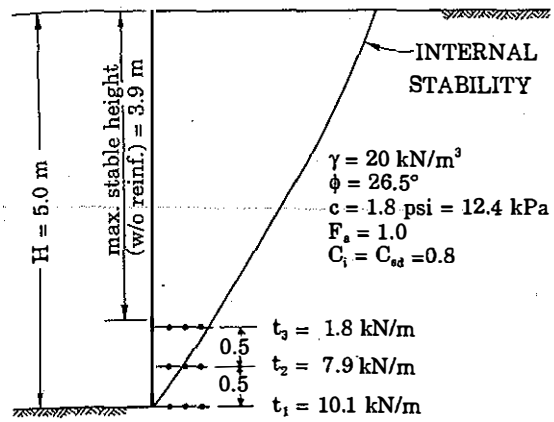

Fig. 12. Layout for a cohesive slope considering *reactive* reinforcement forces.

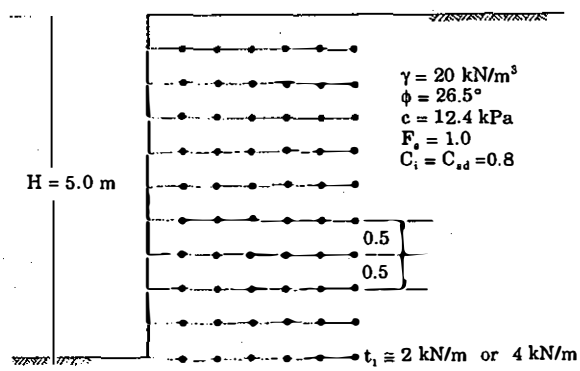

Fig. 13. Layout for a cohesive slope considering *active* reinforcement forces.

becomes more cohesive, the differences obtained in the example will be amplified. In reality, the required strength will be smaller than that predicted in the rational approach since upper layers will likely develop some tensile force, thus reducing the need for force (to satisfy global equilibrium) at the bottom. However, at the current state of knowledge it is generally not known what this reduction would be. Furthermore, the required area for *primary* reinforcement is significantly less for the rational approach. Consequently, being consistent with this approach reduces uncertainties without a significant cost penalty.

2.2 Analysis: Finite Element

Finite element (FE) analysis is a powerful tool having the potential to account for virtually all aspects associated with geosynthetic-reinforced soil problems. These aspects include construction sequence, boundary conditions, reinforcement stiffness, creep, etc. The analysis' output is comprehensive and it includes important information such as structural deformations. The reliability of the output, however, depends mainly on how well the various components of the problem are modelled and how accurate the relevant properties of the materials are defined. The use of the FE analysis may be essential in design when serviceability, at working loads, is a major issue. Also, when reinforcement stiffness plays a major role in the stability of a structure, the FE analysis may provide some insight related to design. An example is a reinforced embankment constructed over soft soil having a stiff crust where relative stiffness of the reinforcement is crucial for safe design. Considering the current design practice of other geotechnical structures though, it seems unlikely that FE will soon become the design tool for most geosynthetic-reinforced soil structures. However, it is certain to be used in research thus enhancing the understanding of reinforced soil behavior and, ultimately, producing simplified design guides useful for practitioners. As an example, if the constituents of a reinforced soil system are modelled adequately, the FE analysis can be used to assess creep of polymers embedded in soil and its effects on structural performance--still a major issue, even in ordinary design.

There have been significant developments in numerical modelling, as well as numerous applications of, the FE analysis in conjunction with reinforced soil during the past 15 years. To name a few, the following list is provided: Herrmann and Al-Yassin (1978), Rowe (1982, 1984), Rowe and Soderman (1984, 1985), Rowe et al. (1984), Rowe and Ho (1988), Rowe and Mylleville (1989), Jones (1988), Bathurst et al. (1992). Matsui and San (1988, 1989, 1990) modelled the problem so that near failure performance could be assessed and, if desired, compared with F_s predictions from a conventional limiting equilibrium analysis. The recent prediction symposium in Denver (Wu et al. 1992), however, demonstrates the

need for further developments. Over ten *Class A* predictions of the performance of two well-defined reinforced walls, all based on FE analysis, were submitted. The range of predictions varied from very conservative to unconservative when compared with actual values such as failure load, displacement field, earth pressure and geotextile strains at various load levels. Generally, this wide range of results may be attributed to different modelling of the problem (e.g., boundary conditions, slip interface elements or no interface at all, facing, soil constitutive relationship, and even numerical discretization). Most importantly, the comparison with actual performance demonstrates that inadequate modelling may be unsafe. Since reinforced problems in the real world are more involved (i.e., less idealized), one can expect then even larger diversity of predictions.

It interesting to point out a parametric study of a hypothetical geosynthetic-reinforced

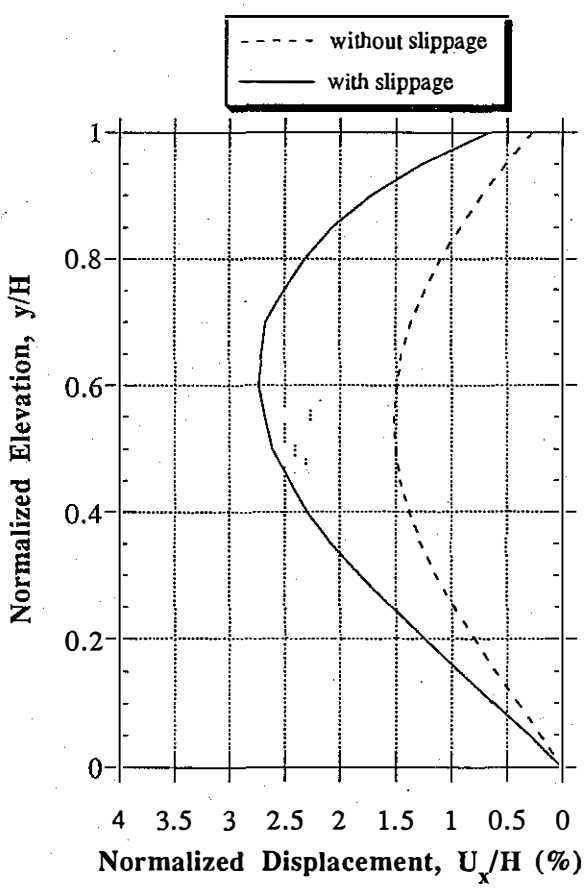

Fig. 14. Effect of *facing-soil* interface modelling on horizontal displacement of facing (after Kaliakin and Xi 1992).

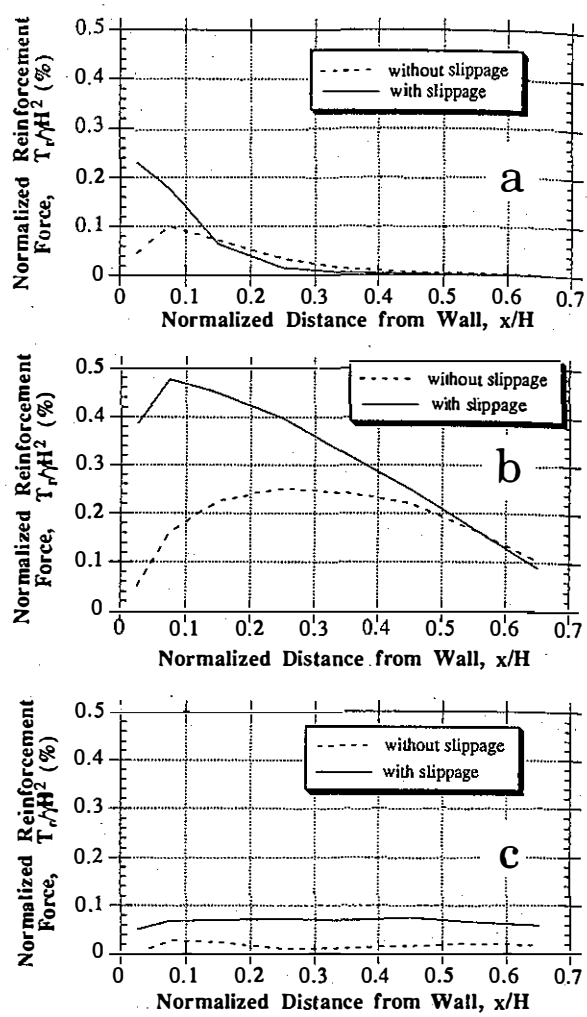

Fig. 15. Effect of *facing-soil* interface modelling on reinforcement force at: a. lowest, b. middle, and c. top layer (after Kaliakin and Xi 1992).

vertical wall conducted by Kaliakin and Xi (1992). For granular backfill they found that using slip or no-slip elements along the geosynthetic interface (i.e., allowing or preventing relative displacement at the soil-reinforcement interface), had minor effects on both outward displacements of the wall and horizontal stress acting against the facing. However, there was some effect on the magnitude and location of tensile force in the reinforcement. However, inclusion of slip interface elements between the backfill and modular facing units has significant effects. Fig. 14 shows that outward displacement can be nearly twice as much if no slippage is allowed. Fig. 15 indicates that the reinforcement force, however, might be about twice if slippage is allowed. Modelling of the

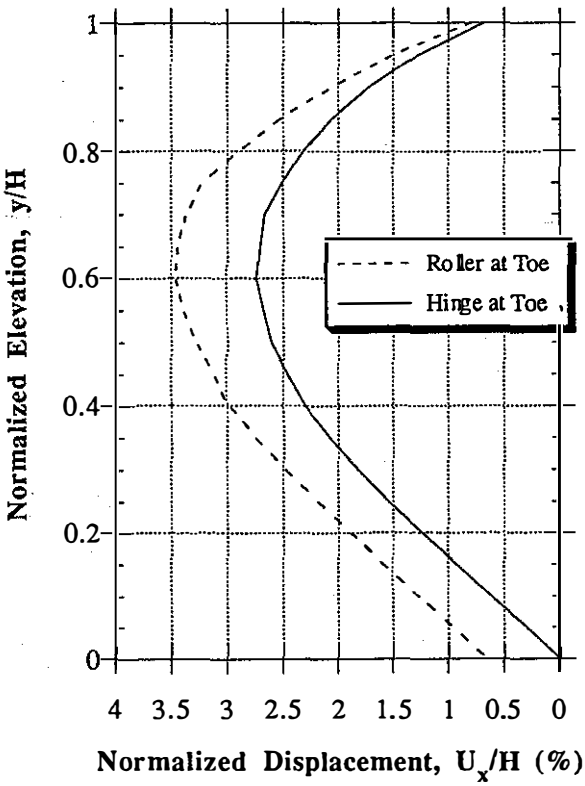

Fig. 16. Effect of *toe* modelling on horizontal displacement of facing (after Kaliakin and Xi 1992).

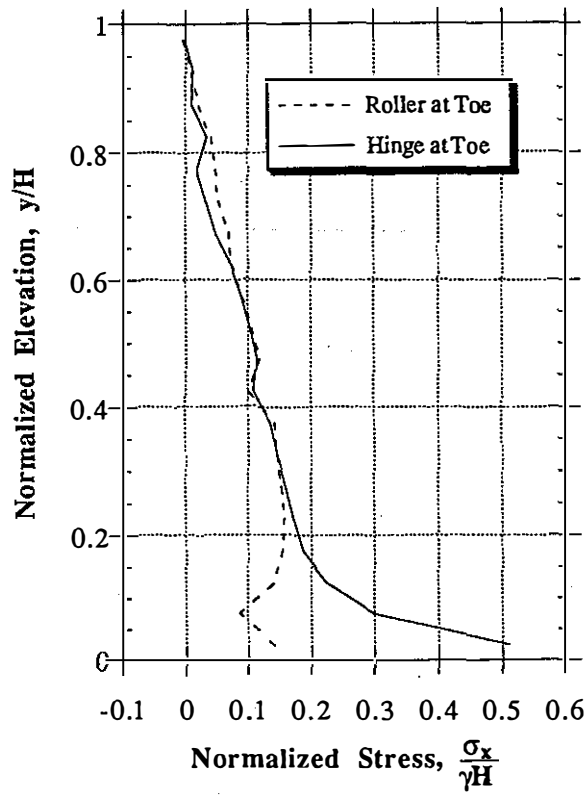

Fig. 17. Effect of *toe* modelling on horizontal soil stress at the facing (after Kaliakin and Xi 1992).

toe boundary condition may be significant. For two extreme toe models (i.e., roller or hinge), Fig. 16 illustrates the differences in predicted facial displacements. Fig. 17 shows that the effect on horizontal stresses against the facing is mainly near the toe. Similarly, Fig. 18 indicates the toe modelling effects on the reinforcement force to be significant up to the wall's mid-height. Simulating the construction process by a single step backfilling underestimates the horizontal movement of the facing (see Fig. 19) as compared to an incremental (real) construction sequence. Also, the single step simulation predicted a significantly lower reinforcement force--see Fig. 20. The Kaliakin and Xi (1992) parametric study contains many more fascinating aspects, all presented in a systematic fashion. It shows the tremendous benefits, as well as difficulties, associated with an application of a sophisticated analysis to geosynthetic-reinforced soil structures. The value of their study, however, is limited to the problem investigated. If, for example, the vertical wall is constructed over soft foundation, the conclusions of their parametric may be different. Furthermore, a parametric study of a different type of structure (say, reinforced embankment over saturated soft clay) is likely to show sensitivity to other modelling aspects (e.g., large vs. small strain FE formulation).

From the discussion in this section one may conclude that FE analysis should not be used as a "black box" in design problems (or research), but rather with understanding of the modelling consequences. Also, detailed knowledge of the properties of the materials involved must be in hand. Otherwise, unsafe predictions may be rendered. In design, results of FE analysis should be double checked against the outcome of an empirically-based approach. If such an approach is not available, attempts to deduce one (conservatively) from a closely related area should be made. A reasonable margin of safety against global failure must be verified using a LE analysis as a measure augmenting the validity of the FE results if those were obtained for working stress conditions.

FE analysis could be used most effectively in conjunction with well-controlled full-scale tests for modification and verification of analysis. It

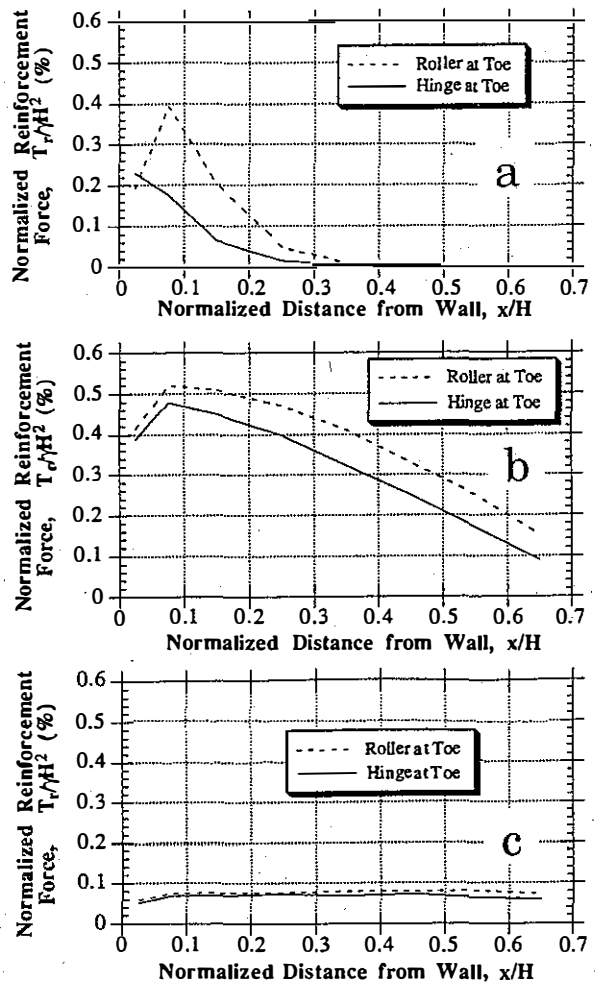

Fig. 18. Effect of *toe* modelling on reinforcement force at: a. lowest, b. middle, and c. top layer (after Kaliakin and Xi 1992).

can then be used inexpensively and with less uncertainties to expand the scope of problems these full-scale tests represent (e.g., to study the effects of stiffness, creep, embedded length, type of fill, boundary conditions, etc.). The end result should not only be a better understanding of reinforced soil behavior, but also simplified and safe design guidelines to be used by practitioners.

2.3 Physical

The least expensive and most popular tests are conducted on small scale models (e.g., Juran and Christopher 1989). Although observations of the performance of such models are useful in a qualitative sense, drawing quantified conclusions may be

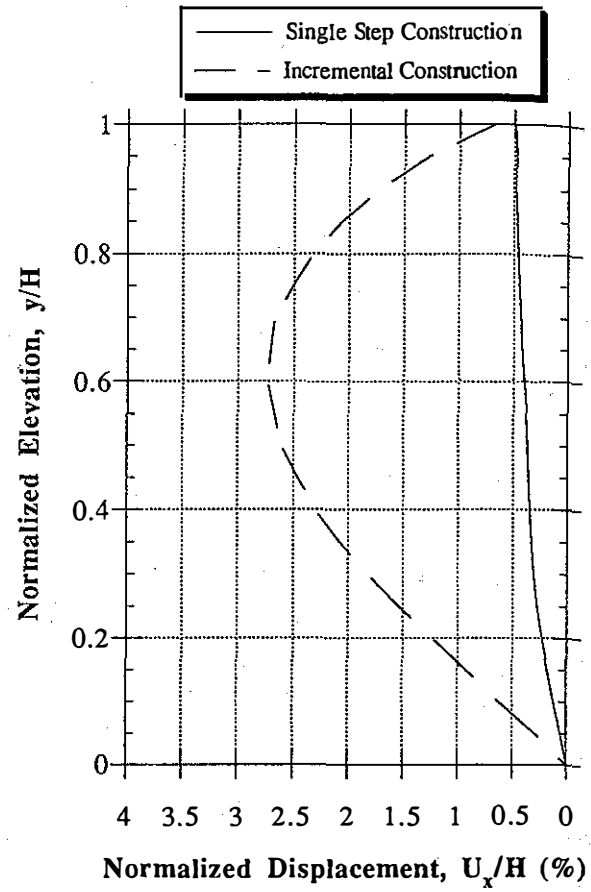

Fig. 19. Effect of *construction* modelling on horizontal displacement of facing (after Kaliakin and Xi 1992).

difficult. For example, mechanisms associated with low soil stresses and geosynthetic properties may be significantly different from those of the prototype.

Models using steel pins as a substitute for granular backfill (e.g., Leshchinsky and Lambert 1991) have the advantage of an increased body force resulting in more realistic "soil" stresses (e.g., a 1 *m* high model corresponds to a prototype about 4 *m* high). Furthermore, the model is easily prepared and is not subjected to unknown end effects (e.g., friction with side-walls of the box containing the model). However, there is a question about the consequences of substituting soil with two-dimensional metal pins. Certainly, the soil-geosynthetic interaction is different from metal pin-geosynthetic. It should be noted, though, that this type of model is suitable to check the validity of predicted failure by the LE analysis. Uniform metal pins behavior is identical to the idealized material behavior assumed in this analysis; i.e., failure

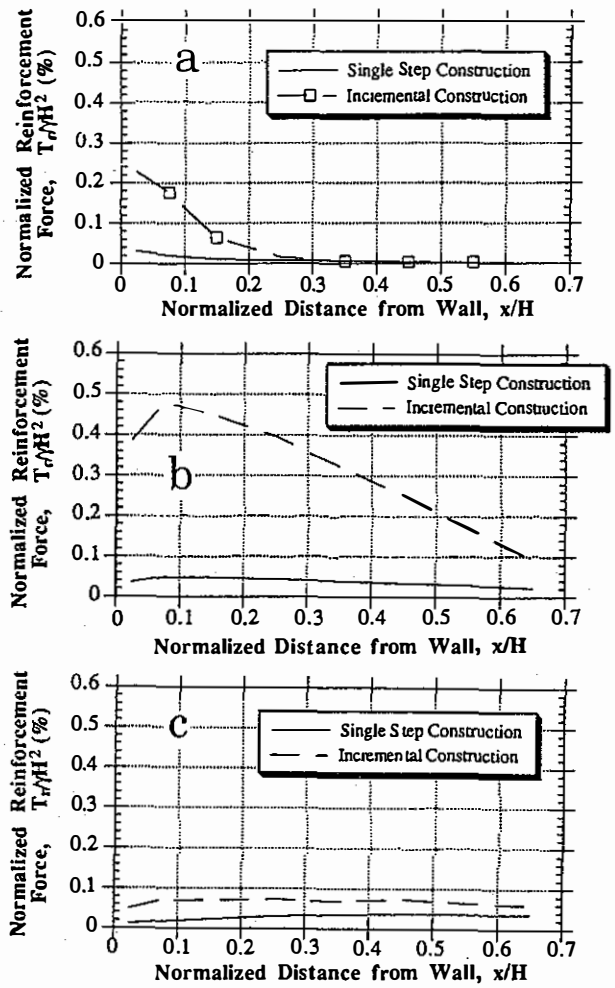

Fig. 20. Effect of *construction* modelling on reinforcement force at: a. lowest, b. middle, and c. top layer (after Kaliakin and Xi 1992).

developed instantaneously; constant ϕ, isotropic and well defined along the slip surface. Also, the location of the slip surface can easily be traced (see Fig. 21). Verification, however, is limited to assessing the validity of the LE analysis within the context of its own simplifying assumptions. It does not necessarily mean that real soil structures exhibit the same performance.

Centrifugal models presumably have stress and strain fields that are identical to the prototype and yet the models are n times smaller (n=ratio between centrifugal acceleration and g). The similarity of stress/strain is a tremendous advantage when non-linear and inelastic materials, such as soils and geosynthetics, are involved. This and the availability of sophisticated centrifuges

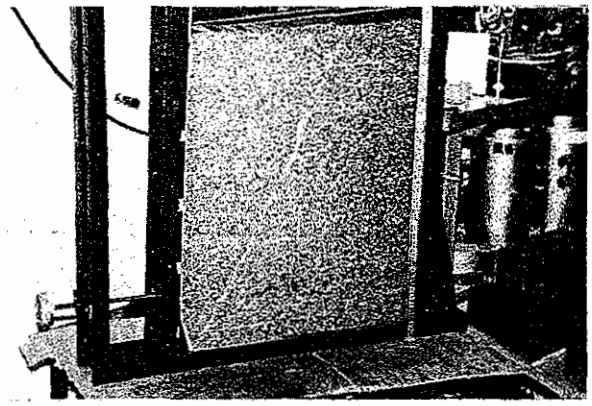

Fig. 21. Appearance of slip surface in models utilizing steel pins.

make centrifugal testing of models increasingly popular (e.g., Jaber et al. 1990). Limitations of this technique, however, should be pointed out. Tatsuoka et al. (1991) have demonstrated unsafe predictions by centrifugal models and attributed these to progressive failure and its relation to shear band formation and particle size. As pointed out by Law et al. (1992), scaled modelling of the prototype's geosynthetic in terms of both ultimate strength and stress-strain relation is important. One can also add the required modelling of creep properties, which further complicates the problem. Hence, finding a geosynthetic that simulates a particular prototype is not simple. Another issue has to do with simulation of interface interaction. Milligan et al. (1990) have shown, using photoelastic techniques and glass beads, that the medium around the reinforcement is strained as the reinforcement deforms. Irsyam and Hryciw (1991) used wax to find the influence zone of a moving inclusion embedded within Ottawa sand. The writer used a modified pullout box, having transparent side-walls, to study the displacements induced in Ottawa sand as a result of tensile force transferred to an embedded geogrid. The displacement field was measured through photogrammetric means. Fig. 22 illustrates schematically the pattern of observed displacements, indicating induced strains 2 to 5 *cm* away from the geogrid. The induced strain due to geosynthetic-soil interaction implies that a model in which the reinforcement spacing is 2 to 5 *cm* (typical to centrifugal models) does not necessarily duplicate the behavior of a full-scale prototype unless the soil particles are also scaled. That

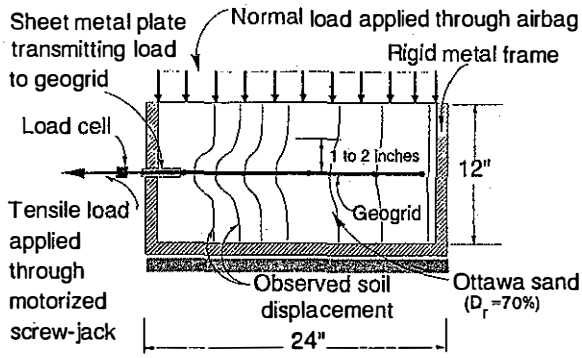

Fig. 22. Observed displacement field in soil due to movement of a confined-geogrid.

is, in the model there is likely to be a strong interaction of reinforcement layers through arching, resulting in load transfer (i.e., shedding). The reinforcement force distribution in the prototype, however, may vary from the model due to lesser interaction of far spaced reinforcing layers, yielding different structural performance and smaller overall stability. For example, the extensive experimental work by Law et al. (1992) predicted a wide range of results as compared to a prototype. Most of their predictions appear unsafe when compared with the collapse load of the full-scale prototype tested by Wu (1992). Perhaps this deviation can partially be explained by the 2 *inch* spacing in the model, versus 10 *inch* spacing in the prototype (i.e., n=5).

Despite the above discussion, the writer thinks the centrifuge to be a powerful tool in studying geosynthetic-reinforced soil. However, rather than trying to directly simulate the performance of a full-scale prototype, it should be used to verify and modify advanced analyses (such as the FE) which are capable of dealing with the true (i.e., not significantly idealized) problem. Once the validity of an analysis is ascertained, while accounting for factors such as interfaces, and hence interlayer interaction, the analysis can then be applied with increased confidence to study the prototype behavior. This process may be very cost effective, especially the in the context of research. If centrifugal models are used to verify simplified analysis methodologies (such as LE), however, care should be taken in interpreting the results. These models may overestimate stability compared to the simplified analyses, thus potentially giving false confidence regarding the results.

Finally, there is no alternative to carefully-controlled full-scaled tests. Although such tests are expensive, they are essential for developing founded (i.e., non-speculative) design methods, as well as for verification of analysis. Examples of significant full-scale tests have been reported by Bathurst et al. (1988a, 1988b), Bathurst and Benjamin (1990), Wu (1992), Adib et al. (1990), Fannin and Hermann (1990), Murata et al. (1991). It should be pointed out, however, that to achieve well-controlled tests, such full-scale models are subjected to conditions that do not necessarily exist in a typical reinforced structure. For example, they are typically contained in a narrow box, having special facial units and subjected to high surcharge load (i.e., boundary conditions that may differ significantly from reality). Sometimes they include instrumentation that might alter the structural behavior at the location of the sensor and subsequently, the significance of the measured values (e.g., strain gages attached using stiff adhesive). Consequently, when extrapolating to design of real structures, one must account for the structural differences and, perhaps, consider the worst case scenario. In any event, full-scale test modelling is the most useful approach to verify analytical models, to understand the reinforced structure's behavior and to develop design procedures with confidence.

3 REINFORCED EMBANKMENTS OVER SOFT SOIL

Population growth and environmental constraints necessitate construction of embankments over soft soils at an increased rate. The past 15 years indicate that utilization of reinforcement in this construction is cost-effective. However, there are some issues associated with this construction, three of which are discussed herein. One issue deals with quality-control of construction; a procedure is suggested in which the reinforcement is accounted for in the most useful fashion. That is, its strains are monitored thus directly indicating the stability of the embankment. Another issue is related to design method in which the required seam strength is estimated. It is suggested that a rational method to predict the length of an

anticipated mud wave is needed. The last issue deals with the low efficiency of sewn seams. This makes the seam the weak link, requiring the selection of a very high-strength reinforcement. An alternative way for traditional sewing, such as use of special glues, may provide an economical solution.

3.1 Construction Monitoring: Strain Gages

A major problem in designing *unreinforced* embankments over soft soil is the accurate characterization of the foundation material. Moreover, the pore-water pressures that tend to develop during construction are difficult to predict. These pressures, however, may cause catastrophic failure. Consequently, it is customary in large projects to monitor the foundation and embankment performance during construction. Typically, piezometers, inclinometers and settlement plates are installed and monitored. These devices' output then is compared with design predictions (with emphasis on stability) and construction activities are adjusted, if deemed necessary. However, the analysis used in design is quite speculative, (even if the devices' readings are used for its "calibration,") and therefore, its predictions include an element of uncertainty. As a result, there are frequent disputes between the contractor and the designer, where one wants to accelerate the construction while the other orders a slow down.

Construction of *reinforced* embankments over soft soil follows a similar layout of sensing devices. This coincides with existing practice for unreinforced embankments since the uncertainties concerning the foundation are similar. However, it seems that the most important sensors, that is, strain gages, are frequently not included. Since the stability of the constructed embankment hinges upon the tensile strength of the geosynthetic, knowledge of the geosynthetic strains is a direct and immediate measure of stability. For example, if the measured maximum strain is, say, 2%, the construction can proceed at an increased rate. If the maximum strain is, say, 5%, the design strength of the geosynthetic has been exceeded and therefore, placement of additional soil layers should be ceased until the foundation gains sufficient strength due to consolidation. If, however, the maximum strain is, say, 10%, the reinforcement may break and a deep-seated failure is likely to follow; hence preventive actions should be taken. Strain measurement as part of construction monitoring also has an indirect benefit. That is, it helps the designer to gain experience and hence, improve subsequent designs. Such experience is invaluable in geotechnical engineering.

Embankments, where reinforcement is *crucial*, are typically constructed over very soft clays (c_u is between 50 to 200 lb/ft^2) and therefore, require high-strength (tensile strength in excess of 1000 lb/ft) geosynthetics for stability. Attaching strain gages to such geosynthetics is rather simple (e.g., Fowler and Leshchinsky 1990). It is recommended to use a silicone rubber as adhesive and sealant. Due to its softness, the effect of this adhesive on the geosynthetic behavior at the measurement location is negligible, thus allowing for strain readings that reflect the true geosynthetic response. Using the same material as sealant will assure long term performance of strain gages under water, as well as provide protection during field installation of the geosynthetic. The writer was involved in two complex field installations of strain gaged geotextiles (Schimelfenyg et al. 1990, and Fritzinger 1990), and reported the lessons learned (Fowler and Leshchinsky 1990). Fig. 23 shows an overview of the installed strain gages in the Wilmington Harbor project. Similarly, Fig. 24 shows the gages in the New Bedford project. Generally, inclusion of strain gages does not overly complicate the field installation; however, it requires coordination with the contractor.

When soft elastic silicone adhesive is used, there is a question whether the strain gage deforms together with the geosynthetic surface, thus reflecting its true strain. That is, the silicone cementation may be insufficient to prevent relative movement between the elongating geosynthetic and the adhered strain gage. Leshchinsky and Fowler (1990) reported the results of a study attempting to answer this question. Two different types of polyester geotextile were used in their study. A strain gage was attached to one side of each specimen while marker points were marked on its other side. Each geotextile specimen was placed in a set of roller grips and then subjected to a wide-width tensile test. A simple photogrammetric technique was used in which displacements of marker points along

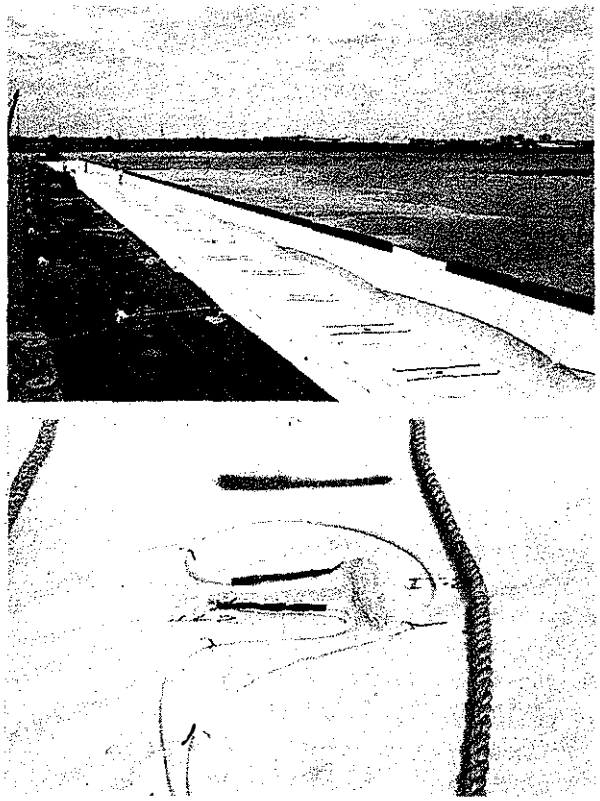

Fig. 23. View of installed strain gages in Wilmington Harbor project.

Fig. 24. View of installed strain gages in New Bedford project.

the deforming specimen were measured from photographs. The corresponding strains were then calculated and compared with the appropriate strain gage output. Figs. 25a and 26a show the stress-strain curves for the Wilmington geotextile, each representing an average relationship of six independent tests using the strain gage output. Figs. 25a and 26a correspond to warp and fill directions, respectively. Figs. 25b and 26b indicate that the calculated strains are nearly identical in both photogrammetric and strain gage measurements. The conclusion was that the bonding obtained using silicone is adequate, at least for woven polyester geotextiles. However, one has to substantiate this conclusion for other types of polymers or geosynthetics before using the silicone adhesive in the field.

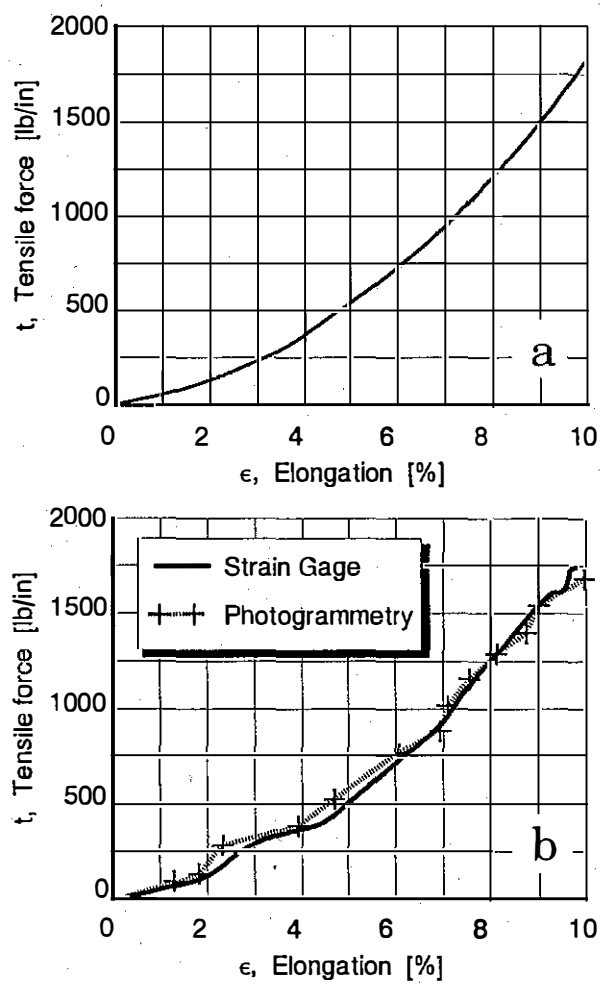

Fig. 25. Geotextile load-elongation curve (Wilmington project) in warp direction: a. average, and b. individual test comparing strain gage and photogrammetric outputs.

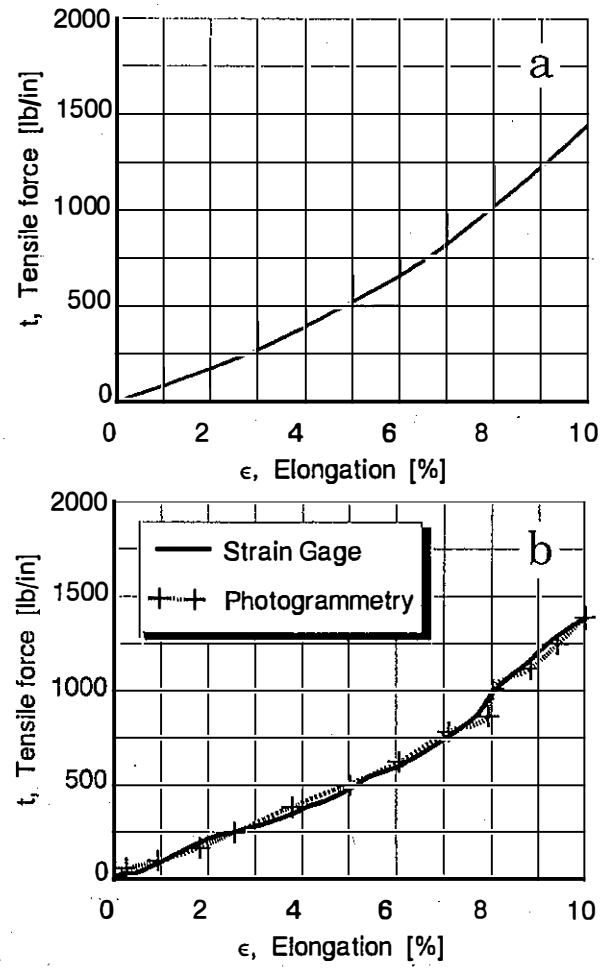

Fig. 26. Geotextile load-elongation curve (Wilmington project) in *fill* direction: a. average, and b. individual test comparing strain gage and photogrammetric outputs.

Aside from the possible problems associated with attaching the strain gage to the geosynthetic, there is the issue of monitoring its output. Leshchinsky and Fowler (1990) suggested the use an off-the-shelf 4½-digit ohm-meter, with an accuracy of 1/100 Ω, to measure the change in the strain gage resistance, ΔR. To determine the strain corresponding to the measured ΔR, the relationship $\varepsilon=\Delta R/(GF \times R_0)$, available in experimental stress analysis textbooks, was used. In this equation, ε=gage strain, GF=gage factor provided by the manufacturer, R_0=initial gage resistance, and ΔR=measured change in strain gage resistance due to elongation. Typical values of GF=2.08, and R_0=120 Ω give $\varepsilon=0.4\Delta R$, where ε is in percent and ΔR is in [Ω]. The ohm-meter reading provides strain within 0.1% of its exact value for a temperature range between 10 to 40° *C*. This accuracy in geosynthetic applications is sufficient for all practical purposes. Measurements of strain gages resistance (and hence, strains) were conducted successfully by un-trained personal in Wilmington (for about 2 years) and New Bedford (for about half a year). Fifty strain gages were read within less than 15 minutes. To facilitate the readings at these sites, the strain gage wires were all connected to a high-quality rotary switch. By turning this switch, the ohm-meter was rapidly engaged with the desired gage.

3.2 Seam Strength: Very Soft Foundation

It is difficult to avoid the formation of mud waves during construction of embankments over very soft soil. Such waves, in turn, tend to drag the geosynthetic sheet from its position. To prevent this relocation, firm attachment of adjacent geosynthetic panels may be required. Fowler (1989) has detailed a procedure through which the required strength of the seam (attachment) could be estimated. It is based on the scheme shown in Fig. 27. Multiplying the residual undrained shear strength of the soft foundation (typically around 50 lb/ft^2 taken also as adhesion) by the effective length along which the mud wave drags the bottom face of the geosynthetic, yields the required minimal strength (of seam or geosynthetic), per unit width, in the fill direction. The actual required strength must be the larger value of either the one obtained from the drag force or the value obtained from a stability analysis considering the thickness of the working table, height of soil piles dumped by trucks, and the equipment live and dead loads.

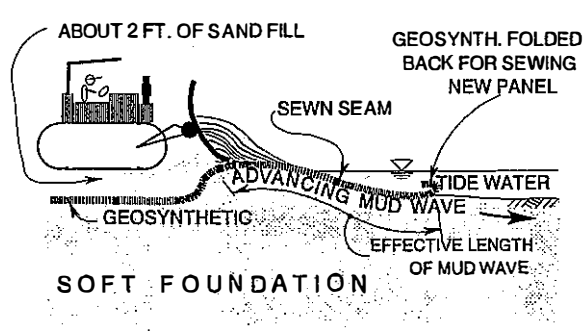

Fig. 27. Mud wave exerting drag force on sewn geotextile panels.

Frequently, the drag force is the factor determining the required geosynthetic strength in the fill direction. Observing Fig. 27, however, one realizes that prediction of the drag force is as good as the prediction of the effective length of the mud wave. Clearly, a methodology in which this length can reasonably be determined is needed. This need is amplified by the fact that currently, seams are sewn and their strength is the weak link in high-strength geotextiles. That is, due to the sewn seam's low efficiency, a geosynthetic with about twice the strength in the fill direction is needed to achieve a prescribed sewn seam strength. As an example consider the Wilmington project. The construction specifications called for a seam strength of 800 *lb/in*. The load-elongation of the geotextile used is shown in Fig. 25a for the warp direction; for the fill direction (Fig. 26a) the ultimate strength was about 1500 *lb/in* (Leshchinsky and Fowler 1990). Since the specified value could not consistently be achieved and in an attempt to remedy the problem, a contractor specializing in sewing approached the writer in July 1987 to conduct a comprehensive experimental study to optimize the stitching. Following the proposed *ASTM* Test Method for Seam Strength of Sewn Geotextiles (current designation is *D 4884-90*), using roller grips and varying stitch type, tightness, density and number of rows, the obtained seam efficiency was between 35 and 45%. Testing another type of polyester woven geotextile with a different weaving technique, increased efficiency, under ideal laboratory conditions, to about 50 to 65%. Consequently, it appears that two issues related to seam strength need immediate attention:

1. A reliable method to estimate the required seam strength (or, alternatively, the effective length of the anticipated mud wave).
2. An improved method of seaming so that efficiency will approach 100%. Geosynthetic strength then will not be dictated by the weak link (i.e., seam strength).

Resolution of these two issues will help to promote a cost-effective solution for the construction of reinforced embankments over soft soils.

4 RAPIDLY CONSTRUCTED STRUCTURES

Innovative construction using geosynthetic reinforcement is frequently encountered in many areas of geotechnical engineering. Typically, a soil structure is designed based on engineering intuition. The successful performance of the structure creates new applications, triggering research that results in reliable analysis and design. However, this evolution is slower when less visible structures are concerned, although their applications may be economically significant. Two such types of structures are described here, including their construction process. Issues with regard to the design of these structures are discussed, suggesting further studies may significantly increase their economics and use.

4.1 Barrier Walls

Barrier walls may be constructed to reduce noise, to absorb the impact of falling rocks in mountainous terrain, to provide flood protection, to retrofit unstable slopes, etc. Typically, these walls are low in height (3 to 4 *m*) and in cost. Emergency needs, as well as low costs, require rapid and simple construction. Subsequently, though these barriers are useful, their aesthetics become secondary. If aesthetics remain unimportant, such walls may be temporary or "permanent." Three different barricades are discussed: double-faced geotextile wall, geotextile-wrapped sandbag wall, and geocell wall.

Woven or nonwoven geotextile sheets can be used to construct the double-faced geotextile wall. To ease construction, each sheet wraps about a 1 *ft* high layer of loosely placed sandy soil (Fig. 28). Two sets of wooden forms are needed (Fig. 29). After the setup of the first set of forms, the geotextile sheet is laid inside; the crib is then filled up with soil and the sheet is folded back (Fig. 30). The second set of forms is placed on top of the just completed layer. The geotextile is laid now inside the crib defined by the second set, the space is filled up with sandy soil, and the sheet is folded back. At this stage, the weight of the second layer is sufficient to interlock through friction the two backfolded portions of the layer underneath. Consequently, the layer underneath is self supported, thus allowing the removal of the

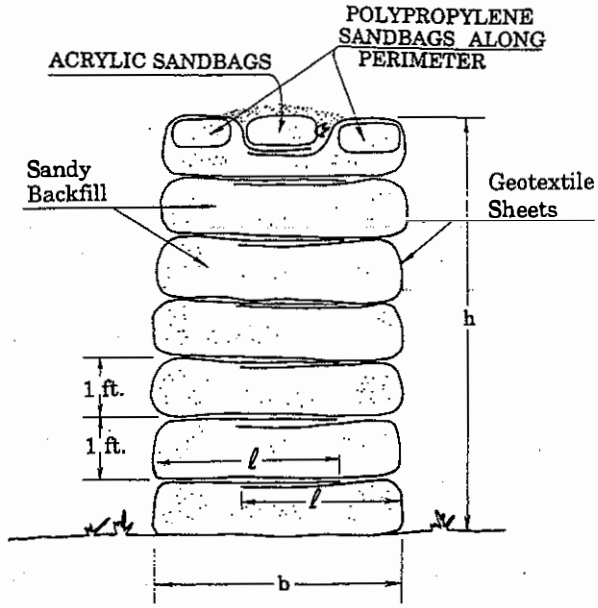

Fig. 28. Double-faced geotextile wall: cross section.

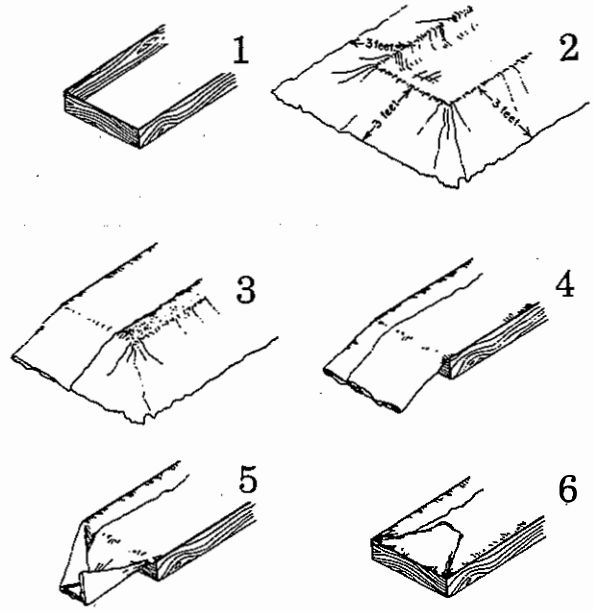

Fig. 30. Schematics of a layer construction.

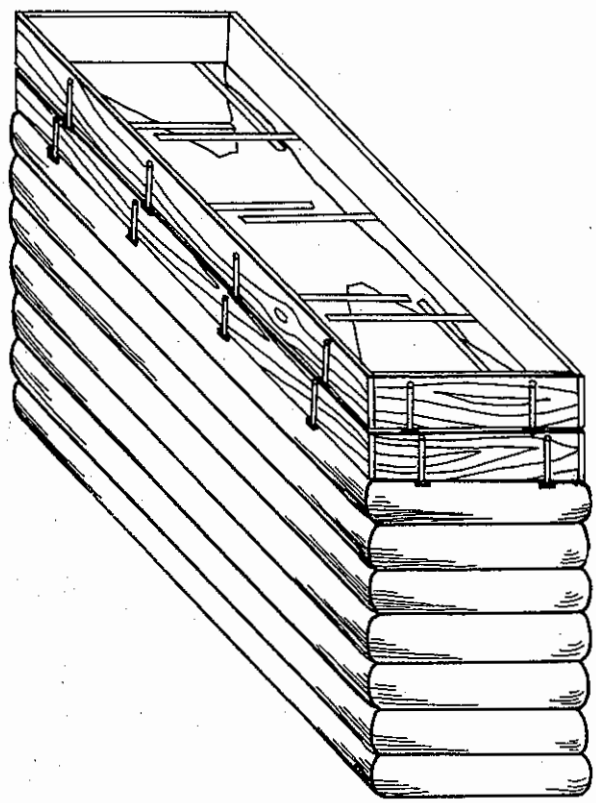

Fig. 29. Arrangement of forms.

first set of forms. This set can be placed now on top, and the process of construction continued up to the desired height. The forms are supported temporarily by removable and reusable metal L-shaped brackets (Fig. 31). If survival over the long run is important, the geotextile portions exposed to UV can be painted using water or oil based paint (Fig. 32).

Fig. 33 illustrates the geotextile-wrapped sandbag wall. Two layers of the sandbags are first arranged, in a staggered fashion (Fig. 34), along the perimeter over a spread-out geotextile sheet. The crib defined by the stacked sandbags is filled up with sandy soil (Fig. 35), followed by the backfolding of the geotextile sheet (Fig. 36). After spreading another geotextile sheet on top of the completed layer, the process of filling up the crib and backfolding is repeated. This sequence is continued until the desired height is achieved. The exposed geotextile can be painted if UV protection is necessary. Forms are not needed and the sandbags provide extra protection in case a tear develops at the face. Because of the filling of sandbags, however, construction time is significantly prolonged.

Geocell material that can efficiently be used for walls is made of HDPE strips, approximately 8 *inch* high. The strips are welded together at 13 *inch* intervals. When expanded, a honeycomb structure is created (Fig. 37). To construct the wall, the first layer of the geocell is expanded in between temporary stakes (Fig. 38). The cells are filled up with a slightly tamped sand. Next, two additional layers of geocell are placed on top

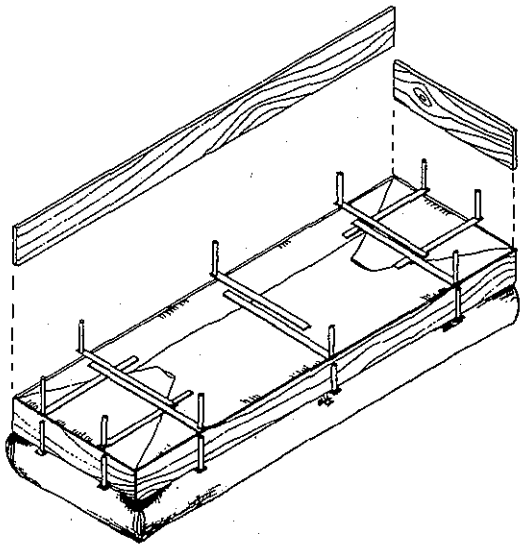

Fig. 31. Arrangement of metal brackets to support upper forms.

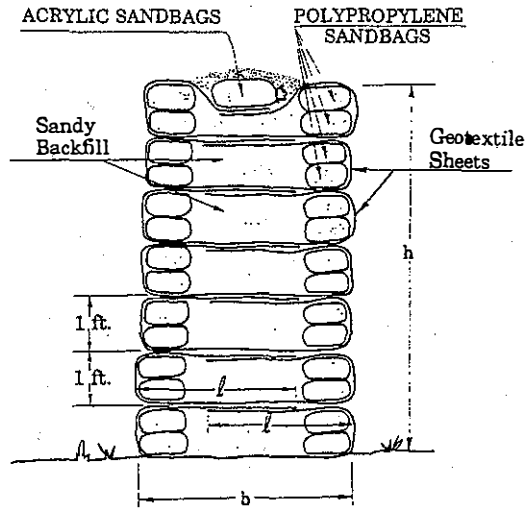

Fig. 33. Geotextile-wrapped sandbag wall: cross section.

(Fig. 39) and locked into position. The locking minimizes sand flow through inter-layer openings and assures the construction of a straight wall. To achieve this locking, the geocell mattress has first to be notched as shown in Fig. 40 using a band saw. The cells of the two layers are filled up, and the process is repeated until the desired height is achieved (Fig. 41). The geocell material is UV stabilized and therefore, may not need protection against sun light. It is shipped collapsed requiring little storage space.

The writer has constructed these types of walls at the Waterways Experiment Station, US Army Corps of Engineers, and conducted a general comparison for a segment measuring 6 *ft* high, 3 *ft* wide and 20 *ft* long. Table 3 was assembled considering availability of three

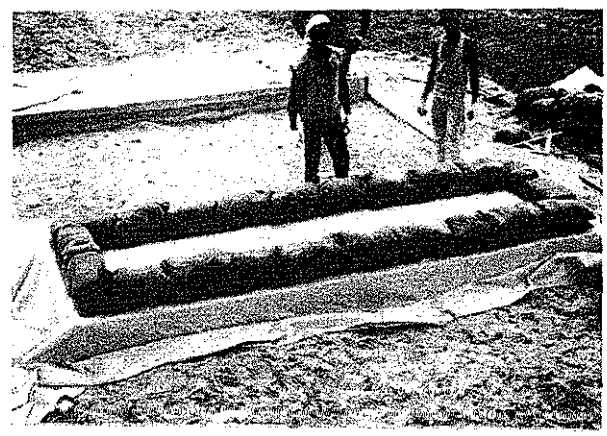

Fig. 34. Staggered sandbags along perimeter.

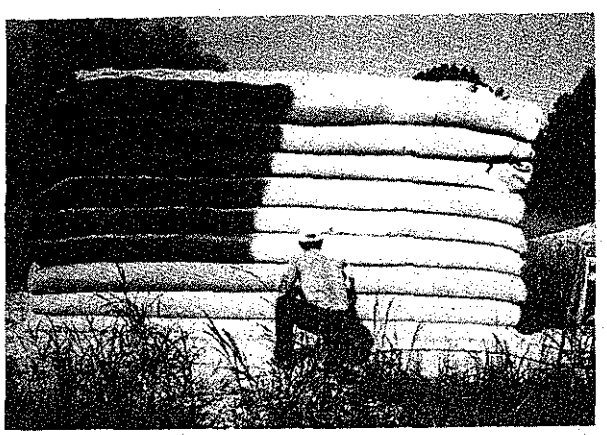

Fig. 32. Spray-painted geotextile.

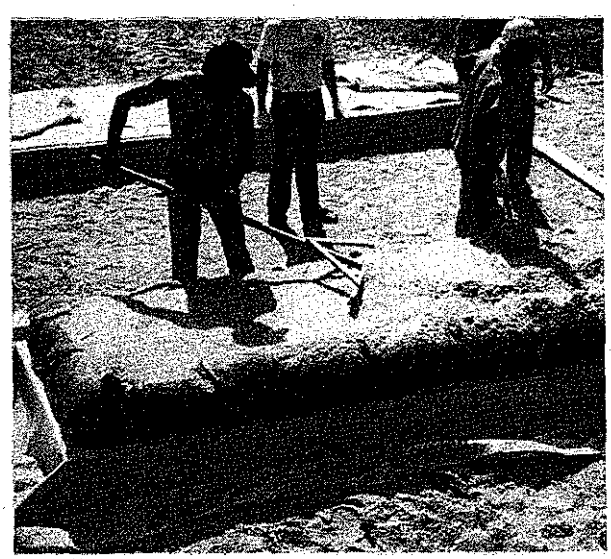

Fig. 35. Sand leveled in sandbagged crib.

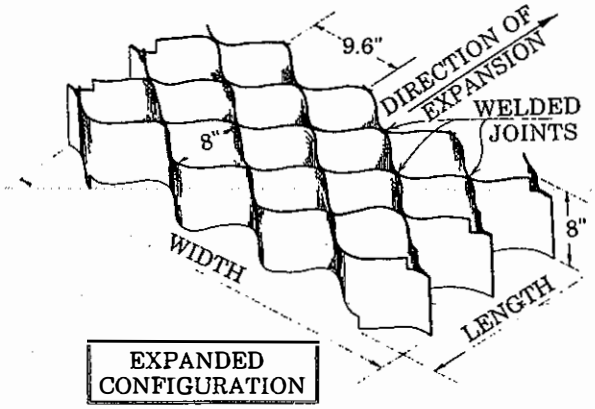

Fig. 36. Back folding of geotextile.

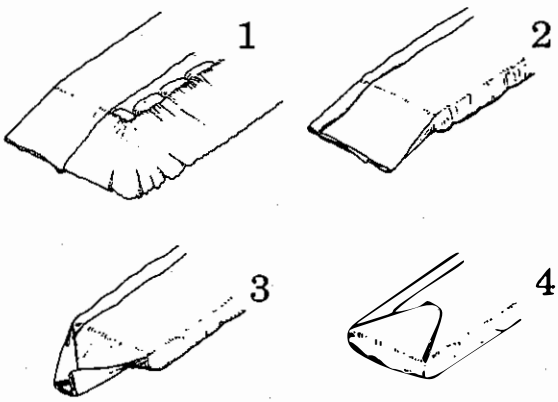

Fig. 37. Expanded notched geocell.

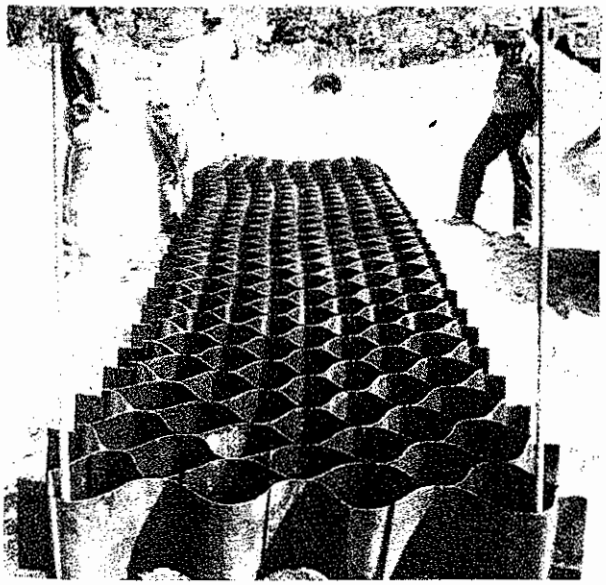

Fig. 38. Metal rods to secure expanded geocell in place.

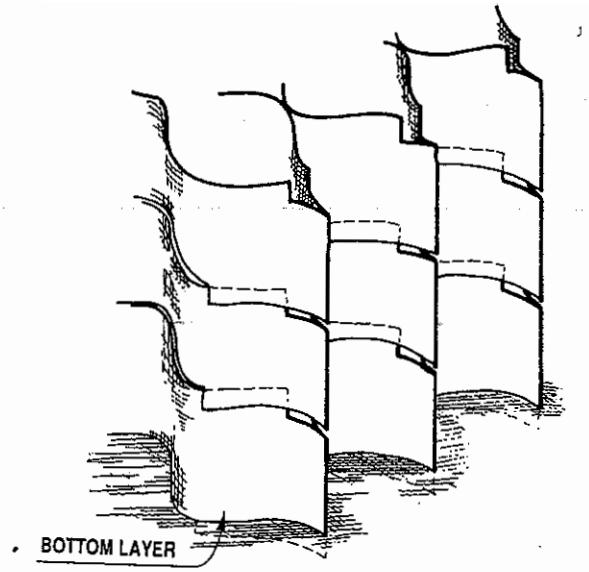

Fig. 39. Interlocked geocell layers.

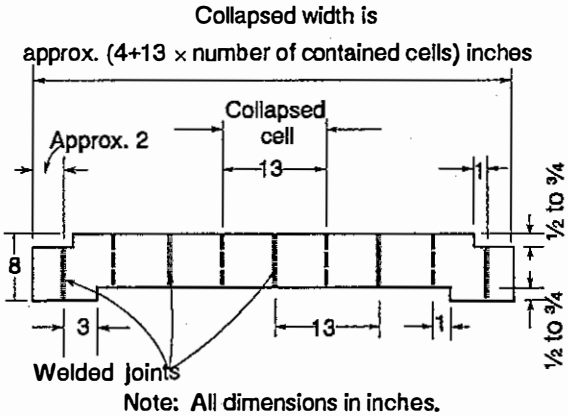

Fig. 40. Notching details.

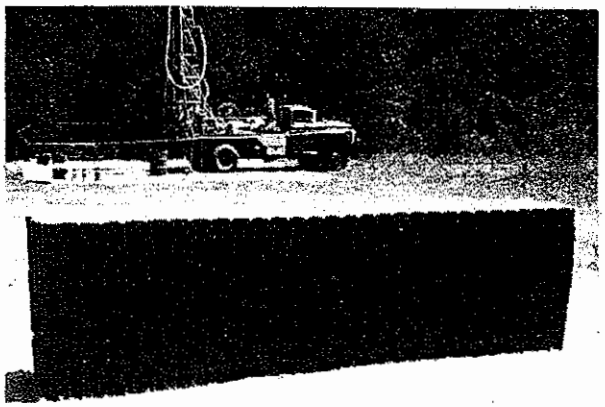

Fig. 41. Finished geocell wall (3 ft. wide by 6 ft. high by 20 ft. long).

laborers and a front-end loader. Compared with an equivalent barrier made of sandbags only, geosynthetic walls are up to 3 times cheaper and can be built 8 times quicker. Construction material (except soil) takes up to 8 times less storage room and may last 10 times longer.

These walls are useful in emergencies and cost-effective in many other applications. However, very little research has been conducted to study their behavior and hence, improve their design. Since the geotextile walls are rather narrow relative to their height, there is a question whether the full Rankine's pressure develops as shown in Fig. 42, especially if they do not retain soil. This pressure, in turn, determines the required strength, as well as the length of backfolding, ℓ. For example, if Coulomb's analysis is used, the wedge will not extend to the surface and will not pass through all layers, thus it may produce a significantly different value than Rankine's. Furthermore, since the geotextile layers are closely spaced and are quite free to displace at the face of the wall, there is a likelihood of load shedding resulting in greatly reduced lateral pressures (i.e., as low as half of those predicted by Rankine's).

Similar issues exist with the geocell wall. Although the strength of the composite

Table 3. General comparison for a 6 by 3 by 20 feet wall.

	GEOTEXTILE WALL		GEOTEXTILE-WRAPPED SANDBAGS WALL		GEOCELL WALL
	Woven	Nonwoven	Woven	Nonwoven	
Quantity of geosynthetic [ft^2]	1848	1848	1848	1848	540
Cost of geosynthetic [$]	100-200 (130-230)(4)	100-200 (130-230)(4)	100-200 (350-450)(4)	100-200 (350-450)(4)	600
Weight of geosynthetic [lbs]	50-70	70-140	50-70	70-140	370
Bulk volume(1) of stored geosynthetic [ft^3]	5-16	20-80	5-16	20-80	14
Workability(2) with geosynthetic	good	very good	good	very good	excellent
No. of sandbags	50	50	420	420	none
Accessories	forms	forms	sandbags	sandbags	none
UV protection	required	required	required	required	none
Significance of face tear	some	little	non- critical	non-critical	none
Construction(3) time [minutes]	250	250	900	900	180

(1) Volume of geotextiles includes voids in roll. Geocell collapsed dimensions are 5.5'× 5"× 8" per layer.

(2) If woven geotextile surface texture is smooth, workability rating declines to good; otherwise it is very good.

(3) Measures for UV protection are not included in the indicated construction time. Time includes filling up of sandbags. Without sandbags filling, actual construction time is about the same for each of the three walls.

(4) Cost includes sandbags.

HDPE/soil is likely to be sufficient for a 4 *m* high wall, the question is whether the welded joint long-term strength at the outer cells is adequate for the same height. Fig. 43 shows a methodology to estimate the required allowable welded joint strength. Once again, k_a based on Rankine's analysis is used; however, lateral earth pressures in a column of cells, 9.6 by 8 *inches* in cross section, are likely to be much smaller, mainly because of the friction with the side-walls of the tall cells. More studies, analytical and experimental, are needed to understand the interaction between the cells and the confined soil.

Despite the analysis issues raised, experience indicates that the approaches depicted in Figs. 42 and 43 yield safe results. It is felt that although the approach for the geocell wall is rational, it yields extremely conservative results, mainly since the soil is *not* treated as a confined material but rather as a homogeneous mass. Further research is likely to produce surprising results showing geocell barriers (and retaining walls) can be constructed safely to significant heights. Their cost-effectiveness can be enhanced through a configuration resembling crib-walls.

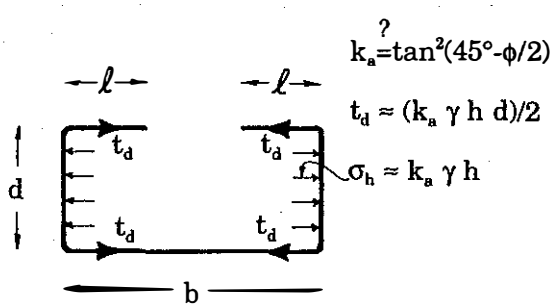

Fig. 42. Approximated lateral earth pressure against bottom facing.

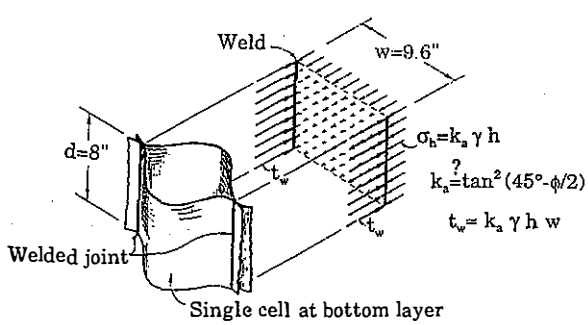

Fig. 43. Force in welded joint calculated from approximated lateral earth pressure.

4.2 Geotextile Tubes

Construction in environmentally sensitive areas (e.g., wetlands) requires techniques causing minimum disturbance and damage. One such technique can be achieved with the aide of dikes made of geotextile tubes. The dikes are placed manually, may retain water on one side while allowing construction on the other, and over time, allow vegetation growth over themselves. These tubes can also be used to construct levees to contain dredged material (Fig. 44), to form a "working table" for embankments over very soft soil, to construct groins to control beach erosion, etc.

The tubes are made of sewn geotextile sheets. Inlet openings on top (Fig. 44) allow for the attachment of a pipe that transports hydraulic fill into the tube. If the fill is sandy, these inlets should be located closely (say, less than 10 *m* apart) to assure a uniform fill up of the tube. If clayey hydraulic fill (i.e., slurry) is used, the inlets may be located over 150 *m* apart, provided the slurry velocity reaching the tube is sufficient. Some case histories are reported by Bogossian et al. (1982), Perrier (1986) and Ockels (1991).

The writer's experience is limited to clayey hydraulic fill. Four 500 *ft* long geotextile tubes were filled with 100% clay (*LL*=120%, *PL*=32% and *PI*=88%), in a dredged material containment site located on Gaillard Island, Theodore Ship Channel, Mobile Harbor, Alabama. This experimental project was carried out by the Mobile District, Corps of Engineers and WES Geotechnical Laboratory. The tubes were fabricated from two 14 *ft* wide

Fig. 44. Geotextile tube filled with slurrified clay serving as an experimental levee.

woven geotextile sheets, laid on top of each other and sewn along their longitudinal edges. The geotextile in each tube had a wide-width tensile strength of 400 *lb/in* in the warp direction and, at least 250 *lb/in* in the fill direction. The geotextiles' equivalent opening size corresponded to sieve No. 70 to 100. Slurry was pumped through an 8 *inch* diameter branch pipeline, connected through a flexible strap to the geotextile inlet opening (Fig. 44). Each tube reached its full height after only about 90 to 120 minutes of net pumping time. The tubes had an elliptical shape, 4 to 5 *ft* height and 10 to 12 *ft* wide.

Two tubes were lined with a nonwoven geotextile to reduce the loss of fines. However, it was quickly realized that even with large openings of the woven geotextile, the fine clay particles clog or blind the fabric within a few minutes. Hence, inner liners may not be necessary for preventing the loss of fines. In case of a seam defect, though, such lining may add an inexpensive extra layer of protection.

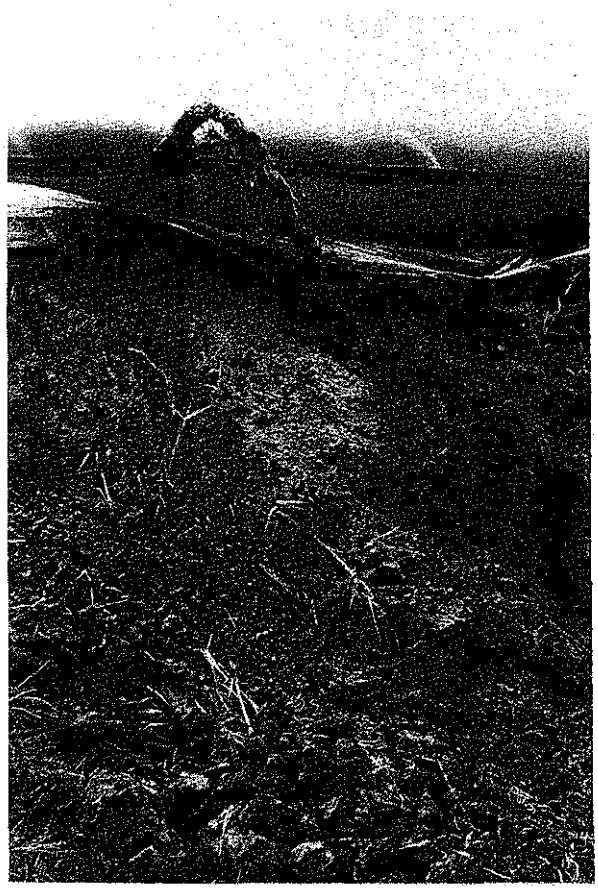

Fig. 45. Slurry flow through defective seam.

That is, seam failures (Fig. 45) may have significant consequences, considering the tube is initially filled with slurry.

Samples taken from within the tube shown in Fig. 44 indicated the water content and total unit weight near the inlet was 214% and 1.25 *gr/cc*, respectively; 250 *ft* from the inlet it was 284% and 1.19 *gr/cc*; and 500 *ft* away it was 308% and 1.18 *gr/cc*. A month later, the slurry turned into a soft clay and the respective water contents and total unit weights were 127% and 1.29 *gr/cc*, 153% and 1.34 *gr/cc*, 286% and 1.19 *gr/cc*. Clearly, more solids tend to accumulate near the inlet. The average height loss during that month was about 50%. However, a second pumping forced the tube to return quickly to its original height.

Eight equally spaced strain gages were installed along the circumference of one of the tubes, immediately near the inlet connection. At full height all measured strains were less than 2%, indicating the geotextile yarns had just realigned and the fabric was hardly stressed. The validity of this measurement, however, is limited to slurrified clay fill.

The main issue with the design of geotextile tube seems to be insufficient experimental data to substantiate the selected required seam strength and, hence, the geotextile strength. Currently available scientific data is scarce. Additional data, for different hydraulic fill materials and different geotextiles should allow for the development of a rational design for this innovative hydraulic/geotechnical structure. Because of the significance of the seam, perhaps a liner should be used in critical applications to make-up for potentially defective local stitch.

5 CONCLUSION

Several issues in geosynthetic-reinforced soil have been discussed. Some of these are:

1. Limit equilibrium analysis may result in larger required total area of reinforcement as the slope becomes flatter, depending mainly on ϕ and pore-water pressure. This seems to be contrary to intuition.

2. In most limit equilibrium analyses the reinforcement contribution is considered as an active force. That is, the forces in the reinforcing layers are taken as known vectors in the equilibrium calculations. However, a

rational approach in design calculations is to take it as a stabilizing element, reacting to (i.e., supplying) the demand created by the applied loading.

3. The layout and required strength of reinforcement determined from limit equilibrium computations are very sensitive to cohesion when active versus reactive reinforcement force is considered in the analysis. The results in the active case cannot be classified as either conservative or unconservative relative to the reactive one. Since the *reactive* approach is sensible in design, application of the *active* approach should be conducted with extreme care in cohesive soil.

4. The finite element method is a powerful analytical tool. Predictions made using this method may vary widely, however, depending on how the reinforced problem is modelled. This variation may produce conservative or unconservative predictions. Consequently, the finite element analysis may be a dangerous tool if used by an inexperienced analyst or without a check of its predictions against results from a simplified analysis.

5. While a large centrifuge is a useful facility to test meaningful physical models, one should cautiously deduce a prototype behavior based simply on similitude. In the model, the reinforcement layers are closely spaced. Consequently, there is load transfer from layer to layer through soil-arching. This phenomenon, which greatly enhances the model performance, may not exist in the prototype. To overcome this, one needs to scale down also the soil particles in the model--a major and sometimes prohibitive task. Centrifugal models are ideal to verify analytical models that can account for interface interaction, boundary conditions, material heterogeneities, etc. Later, the analytical models can be used with increased confidence to study prototypes behavior.

6. The standard instrumentation to monitor construction of embankments over soft soil is frequently prescribed also for reinforced embankments. However, the geosynthetic's performance is rarely monitored, although it can be done reliably using strain gages. The output from such gages is most useful in the framework of quality-control, providing direct information about the embankment stability.

7. The required seam strength of geotextile laid over very soft soil is based on the tensile force needed for stability or on the tensile force due to drag of a potential mud wave, whichever is greater. A procedure to predict the length of a mud wave and hence, drag force, needs to be established. This may have significant influence on the selection of geotextiles, which sometimes is dictated by the low efficiency of sewn seams.

8. It is not clear what values of lateral earth pressure should be used in case the width of a reinforced barrier, not retaining soil behind the reinforced zone, is narrower than the full Coulomb or Rankine wedge. It is also not clear what are those pressures when the soil is contained within narrow and tall cells, even if the barrier serves as a retaining wall.

9. Hydraulically filled geotextile tubes may be useful in rapid construction of low dikes with minimal disturbance of the environment. However, there is not sufficient experimental data to refine a general design procedure for this type of structure.

Despite the above issues, geosynthetic-reinforced earth structures are successfully and economically being built. Failures are scarce. It appears, therefore, that current design procedures follow the conservative aspects of the critical issues, exactly as they should.

Geosynthetics are materials engineered by design. They can be manipulated during manufacturing to possess a desired specific property and thus match most reinforcement needs. As a result, there are many more reinforcing applications that are likely to come about in the future. This prospect makes research and development in geosynthetic-reinforced soil even more exciting.

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