**Technical Paper by D. Leshchinsky, H. Ling and
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UNIFIED DESIGN APPROACH TO GEOSYNTHETIC REINFORCED SLOPES AND SEGMENTAL WALLS

ABSTRACT: Geosynthetic reinforced slopes and walls are being increasingly constructed to perform as permanent structures. Facing blocks are used to improve the aesthetic appearance of walls and to simplify their construction. This paper presents a unified design approach, based on limit equilibrium analysis, that considers the various aspects of stability of reinforced soil structures including the stabilizing effects of facing blocks. Possible instabilities due to tied-back failure, compound failure, direct sliding, and overturning are considered. The design procedure is validated using the test results of two full-scale walls. The results of parametric studies indicate that facing interblock friction significantly reduces the required geosynthetic length and strength for a near-vertical wall. This effect diminishes as the slope angle reduces. Even though the soil-block friction reduces the required geosynthetic strength, it increases the required length of geosynthetic. Examples, and general guidelines for appropriate selection of design material properties and safety factors, are discussed.

KEYWORDS: Geosynthetic reinforced slopes, Segmental walls, Limit equilibrium analysis, Facing block, Parametric study.

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PUBLICATION: *Geosynthetics International* is published by the Industrial Fabrics Association International, 345 Cedar St., Suite 800, St. Paul, MN 55101, USA, Telephone: 1/612-222-2508, Telefax: 1/612-222-8215. *Geosynthetics International* is registered under ISSN 1072-6349.

DATES: Original manuscript received 5 May 1995, revised version received and accepted 10 September 1995. Discussion open until 1 May 1996.

REFERENCE: D. Leshchinsky, H. Ling and G. Hanks, 1995, "Unified Design Approach to Geosynthetic Reinforced Slopes and Segmental Walls", *Geosynthetics International*, Vol. 2, No. 5, pp. 845-881.

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1 INTRODUCTION

Geosynthetic reinforced walls and steep slopes are often more cost-effective than alternative structures. Full-scale tests, however, indicate that the performance of near vertical reinforced walls is significantly better than predicted by simplified limit equilibrium (LE) analysis (Bathurst et al. 1987; Wu 1991; Tatsuoka 1992). That is, analytical predictions for walls are overly conservative. Conversely, predictions for reinforced steep slopes are reasonably conservative (Fannin and Hermann 1990; Christopher and Leshchinsky 1991) although, in principle, the same type of analysis is used for both steep slopes and walls. In LE analyses of slopes, the stabilizing effects of the facia are ignored.

In segmental walls, facing blocks are used as part of the construction. Ultimately, the blocks provide an aesthetically pleasing appearance to the completed structure. The blocks have both high interblock friction and soil-block friction; however, they may be quite expensive. These factors combined with the overly conservative predictions of current analyses create the engineering and economic incentives to include the potential stabilizing effects of the facing blocks in the analysis so as to reduce the required reinforcement length and strength. Bathurst and Simac (1994) considered some of the effects of friction between the facing blocks and soil in the design of near-vertical segmental walls based on the method of design and analysis proposed by the National Concrete Masonry Association (NCMA) in the United States (Simac et al. 1993).

The objective of this paper is to present a unified design philosophy and analysis that is equally applicable to reinforced slopes and segmental walls. The developed design-oriented analytical approach is based on well established procedures in geotechnical engineering. A parametric study implies that facing blocks have significant effects on the required reinforcement strength and length when near-vertical slopes are considered. However, these effects diminish as the slope flattens to about 60° from the horizontal. Instructive example problems demonstrate the effects of typical facing blocks.

2 DESIGN-ORIENTED ANALYSES

2.1 General

Limit equilibrium (LE) analysis has been used for decades in the design of earth structures. Consequently, its extension to the design of geosynthetic reinforced soil structures is quite natural. Attractive features of LE analysis include experience of practitioners with its application, simple input data, useful (though limited) output design information, tangible modeling of reinforcement, and results that can be checked for "reasonableness" through simplified charts or hand calculations. The main disadvantage of LE analysis is its inability to deal directly with displacements and hence, the serviceability issue. It is assumed in practice, though, that prescribed minimum values of safety factors will not only produce a structure possessing an acceptable margin of safety against collapse (i.e. limit state), but that they will also render acceptable displacements and deformations at normal working load conditions.

In principle, inclusion of reinforcement in a LE analysis is a straightforward process (i.e. the reinforcement tensile force is integrated directly into the limiting equilibrium

equations to assess its effects on stability; however, the inclination of this force must be assumed). Physically, reinforcement inclination may vary between the as-installed (typically horizontal) and parallel to the slip surface. Leshchinsky and Boedeker (1989) and Wright and Duncan (1991) have demonstrated that for a granular backfill, this inclination has negligible effects on both required strength and layout of reinforcement. Consequently, the dilemma of the inclination of force can practically be resolved by assuming it to be horizontal. The end result then is reasonably conservative for cohesionless material (i.e. typical backfill). A more significant problem in LE analysis of reinforced soil is the need to know the mobilized tensile force of each reinforcement layer at the verge of imminent collapse. This force may vary between zero and the ultimate strength of the reinforcement. Assuming the actual force is known in advance, as typically done in analysis-oriented computer packages such as STABGM (Duncan et al. 1985), implies the reinforcement force is actually an *active* force, regardless whether it is needed or not. The designer then needs to assume the active force for each reinforcement layer. The design outcome may be greatly influenced by this assumption. Trying to superimpose the facia effects on the assumed active force in the reinforcement layers is then even more challenging. As a result, facia effects are ignored altogether in most LE analysis, although experience indicates it may have significant influence on wall stability (Wu 1991; Tatsuoka 1992).

The following LE analyses deal with various aspects of potential instabilities considering cohesionless backfill soil. It includes: a rational methodology to estimate the required reinforcement tensile resistance; an assessment of potential rotational failures emerging beyond the reinforced zone; an evaluation of possible direct sliding of the reinforced zone; and an estimate of overturning of facia via examination of length of reinforcement. In limit cases (i.e. when reinforcement is not necessary), the analyses results will degenerate to results obtained from conventional LE analysis of gravity walls or rigorous slope stability analysis, depending on the problem and its prevailing modes of failure.

2.2 Internal Stability Analysis

The internal stability (or tied-back) analysis is used to determine the required tensile resistance of each reinforcement layer. Specifying reinforcement layers having an allowable strength that exceeds the calculated required resistance will ensure that (in an analytical sense) the reinforced mass is safe against internal collapse due to its own weight combined with surcharge loads. Development of tensile resistance of the reinforcement is made possible through a tied-back ("load transfer") mechanism in which sufficient anchorage of the reinforcement into a stable soil zone is provided.

The notation and convention used in the analyses presented are shown in Figure 1. Note that the front end of the slope is defined by facing blocks. The reinforcement layers are attached to the facing. When no facia is present (i.e. the case of a conventional reinforced steep slope), the dimensions and related properties of the blocks are ignored. The crest is subjected to a surcharge load, Q . The reinforced soil is cohesionless.

By definition, cohesionless steep slopes are steeper than their angle of repose (i.e. $i > \phi_d$, where i is the wall/slope inclination from the horizontal and ϕ_d is the design angle of friction). In a steep slope, therefore, the force in each reinforcement layer is activated (i.e. *reactive* force) by an unstable soil mass to ensure stability. This reactive force must

Figure 1. Notation and conventions used in analyses.

restore a LE state at each level. To assess the location of the probable unstable mass at each level and subsequently, the required reactive force, a log spiral failure mechanism is chosen. This mechanism agrees with experimental observations of internal failure surfaces (Leshchinsky and Lambert 1991) and is frequently used in a variety of geotechnical problems.

It should be pointed out that a planar surface, commonly used in a tied-back analysis of walls, can be viewed as a segment of a log spiral that has its pole at infinity. Hence, a log spiral mechanism should degenerate to a planar surface in case such a surface is indeed more critical. Finally, a log spiral mechanism makes the problem statically determinate. To realize this, refer to Figure 2. For an assumed surface (i.e. for any three independent parameters defining a log spiral, e.g. for x_c , y_c , and A), the moment equilibrium equation written about the pole can be explicitly written without resorting to static assumptions:

$$\sum M_P = W(x_{CG} - x_c) - \sum_{j=1}^n t_j(y_c - Y_j) = 0 \quad (1)$$

The moment about point P is independent of the $\sigma(\xi)$ distribution over the log spiral. Using Equation 1 one can verify whether the system shown in Figure 2 is stable for ϕ_d

Figure 2. Log spiral slip surface and its statical implications.

(Notes: $\psi = \tan\phi_d$; A = constant of log spiral; ϕ_d = design friction angle; ξ = angle of the log spiral; CG = center of gravity; $\sigma(\xi)dI$ = vertical stress along the length dI of the log spiral slip surface; and $\tau(\xi)dI$ = shear stress along the length dI of the log spiral slip surface.)

when examining all possible log spirals (compare the driving versus resisting moment terms in the moment equilibrium equation).

The procedure for calculating the required tensile resistance of each reinforcement layer is outlined in Figure 3. At this stage the interblock shear resistance is ignored. The friction angle between the reinforced soil and the facing blocks is denoted as δ_f . In Step 1, the soil mass leaning against Sv_1 is considered. The moment equilibrium equation (see Equation 1) is used to find the critical log spiral producing t_{nmax} using the free-body diagram shown. The resulting value of t_n counterbalances the horizontal pressure against Sv_1 . The vertical force component due to lateral earth pressure and interface friction between the wall and the reinforced soil is $t_n \tan(\delta_f)$. This force is carried down-

Figure 3. Scheme for calculating tensile reaction, t_n , in reinforcement.

(Notes: δ_I = friction angle between reinforced soil and facing blocks; and Sv_n = tributary area for layer n .)

wards by the facing blocks. Consequently, the reactive force t_n of layer n is calculated. In essence, the resultant force of t_n and $t_n \tan(\delta_1)$ is inclined at δ_1 to the horizontal and is completely analogous to Coulomb's force in retaining walls. However, here a more generic sliding mechanism is considered (log spiral rather than a plane). That is, when the planar surface is more critical, the log spiral degenerates to a plane yielding results identical to those of Coulomb. The tributary area, Sv_1 , was chosen to extend down to a level just above layer $(n - 1)$ for several reasons. This results in the largest possible tributary area for layer n without activating the layer below. Additionally, the block-reinforcement interface is typically weaker than the block-block interface and is thus likely to serve as a "toe" allowing the most critical log spiral for layer n to emerge.

In Step 2, the force against Sv_2 is calculated. The tributary area Sv_2 extends from layer $(n - 1)$ to layer $(n - 2)$. Using the moment equilibrium equation, $t_{(n-1)max}$, required to resist the horizontal force component of the soil mass against Sv_2 , is computed. When calculating t_{n-1} , the quantities t_n and $t_n \tan(\delta_1)$ have already been determined in Step 1 and therefore, are considered known in magnitude and point of action. Hence, the LE reactive force in layer $(n - 1)$, t_{n-1} , is the only unknown to be determined from the moment equilibrium equation. The vertical force component due to lateral earth pressure against Sv_2 and friction between blocks and reinforced soil, $t_{n-1} \tan(\delta_1)$, is carried downwards by the facing blocks. Figure 3 shows that by repeating this process, the reactive forces of all reinforcing layers, down to t_1 , are calculated while producing a LE state at each reinforcement level.

The outer-most critical log spiral defines the outer surface of the mass retained by layer 1. In a conventional tied-back analysis, it signifies the extent of the active zone (i.e. it is the boundary between the active soil mass and stable soil). Reinforcement layers are anchored into the stable soil to ensure their capacity to develop the calculated tensile reaction, t_j (see Figure 4). The next section, however, demonstrates that the "stable" soil may not be defined by this outer-most surface.

So far, the scheme for calculating the reinforcement tensile reaction has ignored the interblock shear strength. This may be overly conservative and even paradoxical. For example, the required tensile resistance will be the same even if a stable conventional (gravity or cantilever) retaining wall is considered; that is, the reinforcement force must counterbalance the horizontal component of the active lateral earth pressure regardless of whether a conventional stable wall, possibly made of heavy stacked blocks, is already sustaining this pressure. To avoid such a situation, the following scheme has been utilized:

1. Use the procedure illustrated in Figure 3 to calculate t_n through t_i . This procedure implies that the horizontal earth force from the crest (layer n) to tributary layer i is resisted by a resultant horizontal force equal to the summation of t_j where j is varied between n and i .
2. Figure 5a defines the interblock shear strength, Vu_i , as used in the present work. With or without the effects of interblock strength, the same overall horizontal earth force must be resisted at each level. Hence, in lieu of the interblock shear strength, the required reinforcement contribution to resist the horizontal earth force is:

$$\sum_{j=n}^i t_j = Vu_i + \sum_{j=n}^i \bar{t}_j \quad (2)$$

Figure 4. Tied-back analysis: tensile reaction in all reinforcement layers is transferred into the soil adjacent to active zone.

where $\bar{t}_j$ = corrected tensile reaction of layer j accounting for interblock strength; and Vu_i = interblock shear strength. Equation 2 can be rewritten more conveniently as:

$$\bar{t}_i = \sum_{j=n}^i t_j - \sum_{j=n}^{i+1} \bar{t}_j - Vu_i \quad (3)$$

Equation 3 shows that the corrected values of reinforcement force resistance should be calculated from the top layer, n , down to layer 1. This is explicitly illustrated in Figure 5b.

Accounting for facia effects is based on the premise that the total horizontal earth force from the crest to any layer i is a result of "active" lateral pressure having the same resultant force regardless of the facing. Generally, the facing has an effect on the lateral earth pressure distribution (i.e. pressure will be greater if the facing does not yield enough to mobilize the full soil strength); however, the active condition assumption is reasonable for segmental walls and steep slopes and is consistent with LE analysis. Hence, the scheme for calculating t_j shown in Figure 3 remains valid. Now, if at the bottom of the tributary area for layer n there is available interblock shear force resistance, Vu_n , then, based on the premise, the required resisting force for this layer is only $\bar{t}_n = t_n - Vu_n$. If the interblock resistance exceeds the horizontal component of the lateral

Figure 5. Interblock shear strength: (a) definition of interblock shear strength; (b) corrected reinforcement force due to interblock shear strength.

(Notes: N = force due to self-weight of stacked blocks + induced force due to friction of soil-blocks; Vu = interblock shear strength; and δ_3 = friction angle between block and reinforcement layer; Vu_i is taken at the interblock interface with layer ($i-1$) as dictated by the tributary area for each layer in Figure 3 (Vu_1 is at elevation zero).)

earth force, t_n , then no layer strength will be needed in practice. The resisting force required at layer $(n - 1)$ is $(t_n + t_{n-1})$ as calculated before. However, now $\bar{t}_n$ and $Vu_{(n-1)}$ are resisting some of the previously calculated load; hence, horizontal force equilibrium will be achieved when $\bar{t}_{n-1} = t_n + t_{n-1} - \bar{t}_n - Vu_{(n-1)}$. The process continues down to $\bar{t}_1$; at this level the outer-most log spiral in Figure 4 is counterbalanced by $\bar{t}_1 + \bar{t}_2 + \dots + \bar{t}_n + Vu_1$ (see Figure 5b). Note that at each level the full thrust of the lateral force is equilibrated while accounting explicitly only for Vu at that level; the Vu values of layers above have just decreased the required reinforcement *reaction* force of these layers. The end result is a stable system at each level.

Note that the net reaction force in the reinforcement, $\bar{t}_j$, is formally calculated at the face of the slope (see Figure 3 and Figure 5). This implies that the strength of the connection between the reinforcement and the facing unit (that retains the soil “slice” - Figure 3) should be equal to a minimum of $\bar{t}_j$. In the case where the connection strength is less than $\bar{t}_j$, the reinforcement will detach from the facing, resulting in direct sliding along interblock surface j (i.e. the disconnected reinforcement cannot react to resist the force acting over the area S_{Vj}).

The aforementioned procedure can be viewed in the context of conventional retaining walls. In such monolithic walls, the horizontal force component of the lateral earth pressure is sustained through resistance to sliding along the base of the wall. If this resistance is insufficient, an increase can be achieved by a tied-back procedure in which the unbalanced portion of the lateral load is transferred back into the stable soil (i.e. tied-back). The scheme in Figure 5b describes exactly the same procedure; however, it examines the capacity of facing blocks to carry existing lateral loads at each level, starting at the crest. Hence, the calculated reinforcement reactions ensure facia stability at each reinforcement elevation. In the case of a conventional wall, which is stable against direct sliding, the scheme implies that no tied-back reinforcement is necessary. In fact, as the height of such a wall increases, the scheme will indicate at what height tied-back reinforcement is needed to attain horizontal force equilibrium. The equivalent case for a segmental wall is that there exists a possibility that no reinforcement will be needed to carry lateral earth pressure near the crest. Simply, the interblock shear resistance in that zone is sufficient to counterbalance the horizontal load and thus prevent direct sliding.

Finally, note that Vu is due to the friction between a block and the reinforcement material as expressed by the angle δ_3 (Figure 5a). The frictional force, $N \tan(\delta_3)$, depends on the interblock normal force N . This force is due to self-weight of the column of blocks and the vertical force component induced by the friction between the reinforced soil and blocks. This induced force, at each elevation, was calculated as discussed previously (Figure 3) and its value is $(t_n + t_{n-1} + \dots + t_j) \tan(\delta_1)$. Of course, for δ_1 and δ_3 to develop, a small relative movement between soil-block and block-block, respectively, is required. The design values of these parameters will depend on the feasibility of this movement occurring.

2.3 Compound Stability Analysis

The tied-back analysis provides the required tensile resistance at the level of each reinforcement layer. It also yields the trace of the outer-most log spiral defining the “active” zone, a notion commonly employed in conventional wall design. In reinforced soil

structures, the ability of the reinforcement to develop the required tensile resistance depends also on its pullout resistance (i.e. resistance due to anchorage into the stable soil zone). If the boundary of this stable zone is defined by the “active” zone, then the possibility of potential slip surfaces, passing to the right of the outer-most log spiral in Figure 4, outside or within the effective anchorage length, will not be critical. However, such potential surfaces may render reduced pullout resistance since the effective anchorage length is shortened. That is, the reduced tensile resistance capacity along these surfaces could potentially produce a globally unstable system. Consequently, a slope stability analysis approach is used to determine the required reinforcement length so that compound failures (i.e. surfaces extending into the unreinforced soil zone) will not likely occur. The objective of the compound analysis is to find the minimum length of each reinforcement layer needed to ensure adequate stability against rotational failures.

Note that the compound stability analysis presented in this paper is a modification of the analysis introduced by Leshchinsky (1992). That is, here the allowable strength of the reinforcement, as specified in an actual structure, is used whereas before only the calculated tied-back reaction, t_j , was utilized. As a result, for strong reinforcement, shorter lengths will be required according to the modified compound analysis. In the previous analysis, however, specified reinforcement which was stronger than t_j had no effect on the required lengths.

Tied-back analyses give the required reinforcement strength at each level. In actual practice, specified reinforcement layers will have allowable strengths in excess of that required (i.e. $t_{(allowable)j} \geq \bar{t}_j$). The end result is that globally, the tied-back analysis requires only m reinforcement layers extending outside the “active” zone (Figure 4) and into the stable soil. That is, the stronger than required m layers will maintain stability of the active mass. Internally, however, layers $(m + 1)$ through n are also needed to ensure local stability as implied in the scheme presented in Figure 3. To calculate the minimum number of layers, m , the following equation is used:

$$\sum_{j=1}^m t_{(allowable)j} \geq \sum_{j=1}^n \bar{t}_j \quad (4)$$

Note that m is the number of layers, counting from the bottom, capable of developing a total tensile resistance equal to (or slightly exceeding) the net total force as obtained from the internal stability analysis (Figure 5b). When $m = n$ the compound analysis degenerates to that introduced by Leshchinsky (1992). The m layers are selected as bottom layers and may contribute their full allowable strength to global stability (i.e. slope stability) when compound stability of the reinforced system is examined.

Embedding the reinforcement immediately to the right of the outer-most log spiral obtained in the tied-back analysis (refer to Figure 6), so that $t_{allowable}$ for layer 1 through m can develop through pullout resistance, ensures that, in an average sense, the mobilized friction angle, ϕ_m , along this log spiral is equal to, or slightly less than, ϕ_d . The upper layers $(m + 1)$ through n (see points A, B and C in Figure 6) are not needed for the global stability of the “active” mass and therefore, from a theoretical viewpoint, could be ignored at points A, B and C. Note that the mobilized friction angle, ϕ_m , represents the required angle to produce a limit equilibrium state while accounting for the allowable reinforcement strength. Hence, when $\phi_m < \phi_d$, a fictitious situation is ana-

Figure 6. Scheme for computing reinforcement length required to resist compound failure. (Notes: ϕ_d = design friction angle; and ϕ_m = mobilized friction angle needed to produce a LE state for an arbitrarily specified log spiral.)

lyzed: i.e. the system is stable (not in a LE state) since the available soil strength, as expressed by ϕ_d , is larger than needed. Only when $\phi_m = \phi_d$ is a LE state achieved.

At this stage of analysis, reinforcement layers 1 through m are lengthened to a test body defined by an arbitrary log spiral extending between the toe and the crest, to the right of the outer-most log spiral (Figure 6). Each layer beyond the slip surface is embedded so that the calculated $t_{allowablej}$ can be developed; ϕ_m for this surface will be smaller than ϕ_d used in design (i.e. for this layout, the outermost surface from tied-back analysis is most critical). The upper reinforcement layer is truncated in a numerical sense (i.e. $t_m = 0$), and the moment equilibrium equation for the arbitrary log spiral (see Equation 1; include the effect of base resistance $V u_1$ if facia is considered) is used to check whether $\phi_m = \phi_d$. If $\phi_m = \phi_d$, layer m is sufficiently long (see point D in Figure 6); otherwise, lengthen this layer and repeat calculations until a satisfactory length is found. A satisfactory length implies that the critical log spiral passing through point D

yields a stable system for the design friction angle, ϕ_d ; all feasible log spirals between this one and the one from a tied-back analysis have $\phi_m < \phi_d$ indicating they represent less critical mechanisms (note that the strength of layers 1 through m is available between these two log spirals).

The process is repeated to find the required length of layer $(m - 1)$ (Figure 6). Since the layers above were already “truncated”, they no longer contribute tensile resistance to deeper slip surfaces. Once the process has been repeated for all layers down to layer 1, the length of all layers (curve DEFGH in Figure 6) required to ensure that ϕ_m does not exceed ϕ_d for all possible log spiral failure surfaces, is found. To be slightly conservative (and to avoid consideration of surfaces where only a fraction of the pullout resistance is available), all anchorage lengths can be specified beyond points D, E, F, and G. This simplification is conservative since, contrary to the compound analysis procedure, it ensures the following: $t_{(allowable)m}$ at point D (not zero resistance at D);

$t_{(allowable)m-1}$ at point E (not zero resistance at E); and so on. However, since the anchorage length for planar geosynthetic sheets is small relative to its total required length in practical problems, this simplification is reasonably conservative. Even though this simplification is not essential, it will greatly simplify any computational process.

Compound critical slip surfaces emerging above the toe are possible and consequently, the procedure in Figure 6 should be formally repeated for slip surfaces emerging through the facing blocks. Subsequently, layers previously truncated are lengthened if necessary to ensure that $\phi_m \leq \phi_d$. Due to practical considerations, however, a uniform or linearly varying reinforcement length is typically specified and as a result, most layers are longer, and some are stronger, than optimally needed. For simple layouts, repetition of the procedure in Figure 6 for slip surfaces emerging at the slope surface is not likely to affect the specified length. It should be pointed out that the outlined compound analysis procedure can result in an efficient and design-oriented computation process which calculates the optimal length of reinforcement (e.g. Leshchinsky 1995, program ReSlope).

Finally, at points A through G, the anchorage length is calculated to resist the pullout force (i.e. $\bar{t}_n, \dots, \bar{t}_{m+1}, t_{(allowable)m}, \dots, t_{(allowable)1}$). This force is multiplied by a factor of safety for pullout, FS_{po} , which typically has a value of 1.5. When choosing a value for FS_{po} the overburden pressure along the anchored portion and a parameter defining the shear strength of the soil-reinforcement interface are considered. This parameter, C_i , called the “interaction coefficient”, relates the reinforcement-soil interface strength to the reinforced soil strength $\tan(\phi_d)$. The interaction coefficient is typically determined from a pullout test. Adding the anchorage length to the length needed to resist compound failure allows the designer to determine the longest length required to resist tied-back and compound failures. This length could be specified for construction as demonstrated in Section 3.

2.4 Direct Sliding Analysis

Specifying a reinforcement layout that satisfies a prescribed ϕ_d against rotational failure on the basis of a compound analysis (Figure 6) does not guarantee sufficient resistance against direct sliding of the reinforced zone along its base, or along any layer of reinforcement. The reinforcement length required to ensure stability against failure due

to direct sliding, L_{ds} , is determined from a force LE analysis; i.e. the two-part wedge method.

Figure 7 shows the notation used in defining the geometry and forces in the two-part wedge analysis. First, an initial value of L_{ds} is assumed. Then, for an assumed interwedge force inclination, δ_2 , the maximum value of the interwedge force, P , is found by varying θ while solving the force equilibrium equations for the active Wedge A. This interwedge force signifies the resultant of the lateral earth pressure exerted by the retained soil on the reinforced soil. Next, the vertical force equilibrium equation for Wedge B is solved considering the frictional force between the reinforced soil and facing blocks (i.e. $\sum t_i \tan \delta_1$) and the vertical force component of the lateral thrust of the active wedge (i.e. $P_{max} \sin \delta_2$). Subsequently, the reaction N_B is obtained and the base sliding resisting force of Wedge B, T_B , is calculated. Note that the shear strength of the facing block at elevation zero, Vu_1 , should be added to T_B as a force resisting sliding. The force Vu_1 , defined in Figure 5, is a function of the weight of the vertical column of blocks and the frictional force induced (i.e. transferred) by the reinforced soil into the blocks, $\sum t_i \tan \delta_1$, calculated in the internal stability analysis.

When calculating T_B , the coefficient C_{ds} is used (C_{ds} = the interaction coefficient between the reinforcement and soil as determined from a direct shear test). Two such coefficients are needed: one for the reinforcement-reinforced soil interface and one for the reinforcement-foundation soil interface.

Figure 7. Two-part wedge mechanism for direct sliding analysis.

(Notes: *If layer 1 is above the foundation, check also for possible sliding along the reinforced soil-foundation soil interface (then take $C_{ds} = 1$ and the soil parameters giving the largest value of L_{ds}). Otherwise, check for possible sliding above and below bottom layer (i.e. along the interface with reinforced soil and with foundation soil).)

At this stage, the actual factor of safety against direct sliding, FS_{ds} , is calculated using the following equation:

$$FS_{ds} = \frac{T_B + Vu_t}{P \cos \delta_2} \quad (5)$$

This value of FS_{ds} is then compared with the specified value. In case of an unsatisfactory value, the value of L_{ds} is changed and the process is repeated. This calculation procedure is ideally suited for computers (Leshchinsky 1995). Specifying the reinforcement length L_{ds} ensures satisfactory stability against direct sliding.

The assumed value of δ_2 may have a significant influence on the outcome of the analysis, especially as the wall face inclination decreases. Normally, the value can vary between 0 and ϕ_d . Selecting $\delta_2 > 0$ implies that the retained soil will either settle relative to the reinforced soil above, L_{ds} , or the reinforced soil will slide slightly as a monolithic block thus allowing interwedge friction to develop. For non-vertical slopes ($i \leq 70^\circ$), the effect of reinforcement layers, some of which typically intersect the interwedge interface, are ignored, and therefore selecting a value of δ_2 between $(2/3)\phi_d$ and ϕ_d should not be viewed as an unconservative choice. In fact, Schmertmann et al. (1987) used the value $\delta_2 = \phi_d$ in a two-part wedge analysis. For walls, selecting a large value for δ_2 is at least as justifiable as specifying δ_1 greater than zero.

The direct sliding analysis can be viewed as an extension of the existing practice for evaluating the sliding resistance of conventional gravity walls. When the interblock shear resistance at the bottom, Vu_t , is very large, the value of L_{ds} produced by the analysis will approach zero, yielding a Coulomb wedge that is retained by a stable gravity wall.

2.5 Overturning Analysis

Normally, overturning is not a concern associated with reinforced soil walls and is even less of a concern in steep slopes. However, when facia effects are significant, the tied-back, compound and direct sliding analyses may indicate that very short, or no reinforcement, is required. There is then the possibility that overturning will occur in the same fashion as in a conventional gravity wall, though it may first occur at some height above the toe. Installing reinforcement layers with minimal length, at a given spacing, will prevent such a potential failure.

Figure 8 shows the mechanism utilized to calculate the required layer height to resist overturning. Similar to the direct sliding analysis, P_{max} between Wedges A and B is calculated after assuming an initial value for the required geosynthetic length to resist overturning, L_{ot} . The interwedge force, P , is assumed to act at $\bar{Y} = 1/3$ the interwedge height for a surcharge-free problem; otherwise, a conservative value of $1/2$ the height is used. Next, moments generated by Wedge B (due to self-weight, surcharge and the vertical and horizontal components of P) and the facia are calculated about point O. The factor of safety against overturning is then calculated using the following equation:

$$FS_{ot} = \frac{M_R}{M_D} \quad (6)$$

Figure 8. Mechanism to calculate required layer length to resist overturning.

(Notes: FS_{ot} = factor of safety against overturning; M_R and M_D = resisting and driving moments, respectively, of Wedge B and facia (i.e. reinforced zone) calculated about point O.)

Figure 9. Simplified layout for reinforcement length.

(Notes: $L_B = L_T$ when based on the longest length from compound failure analysis, direct sliding analysis or overturning analysis; or, L_B = longest length as above, and L_T = length from compound analysis.)

and compared with the prescribed value. If necessary, a new value for L_{ot} is assumed and the computational process is then repeated until a satisfactory FS_{ot} value is obtained. Formally, if the upper reinforcement is significantly shorter than L_{ot} , the overturning stability of higher layers should be checked as well.

The overturning analysis is a direct extension of the conventional analysis for retaining walls. In the limit, when sufficiently wide and heavy facing blocks are used, the analysis outcome will indicate that the required L_{ot} is zero; Wedge A will then be directly retained by a stable gravity wall comprised solely of stacked facing blocks.

2.6 Specified Layout

A practical layout of reinforcement is shown in Figure 9. To simplify construction, all layers can be specified to have equal length. This length is selected as the longest value obtained from the four analyses. Alternatively, a safe layout will be L_B and L_T at the bottom and top, respectively, where L_B is the longest length from all four analyses, and L_T is the length obtained from compound and tied-back failure analyses. Intermediate layers should be specified based on a linear interpolation between L_T and L_B . A layout comprised of varying lengths of reinforcement is more economical but it may complicate construction. Specifying each layer length based on its longest length from all stability analyses will produce an optimal structure; however, an impractical layout is likely to result.

3 MATERIAL PROPERTIES, FACTORS OF SAFETY, AND DESIGN CONSIDERATIONS

Limit equilibrium analysis deals, by definition, with a structure that is at the onset of failure. Adequate factors of safety ensure that an acceptable margin of safety exists against such states of failure. It is therefore implicitly assumed in a LE analysis that the different materials involved (i.e. geosynthetic, soil and interblock interface) will all contribute their full strength when the assumed state of failure occurs. For materials reaching a constant shear strength (i.e. materials exhibiting plastic behavior) after a small deformation, such an assumption is correct; however, the materials in the reinforced soil problem may not possess this idealized plasticity. Consequently, the following guide is given for selecting material properties for analysis.

Slip surface development within a soil is a progressive phenomenon, especially in reinforced soil where the reinforcement layers delay the formation of a surface in the vicinity of the reinforcement (e.g. Murata et al. 1991). The slip surface does not develop simultaneously along its full length and thus, the peak shear strength of the compacted backfill is not simultaneously mobilized. That is, when a slip surface fully develops within the soil, ϕ along this surface will vary between peak and residual values. Consequently, it is recommended to use a design value of ϕ_d that does not exceed the residual strength of the soil. The value of the friction angle typically reported by commercial laboratories corresponds to a peak shear value. In this case, a reduction factor, RD , of 1.3 will usually ensure that the design friction angle is at or below its residual value (i.e. $\phi_d = \tan^{-1}(\tan\phi_{peak}/RD)$). It is recognized that by using residual values (and not the as-placed peak strength), the gain in soil strength due to compaction is basically ignored

in the analysis and thus, has an overall conservative impact on the designed structure. Consequently, the complex issue of progressive failure is then avoided while assuring results “on the safe side”. Furthermore, compaction will reduce structural deformations at working loads, and most importantly it will prevent potential shear of reinforcement layers at the reinforcement-block connection due to excessive compression of the reinforced soil relative to the facing. Hence, use of the residual strength in analysis should not undermine the significance of compaction of backfill. It should be stated that the common practice of selecting $\phi_d = 30^\circ$ to 33° very likely represents residual strength values that are at or below residual strength values of typical granular soils used for backfill.

The LE analysis assumes that the reinforcement and soil will supply their design strengths at the same instant, regardless of deformation characteristics. Even though the use of the residual strength of the soil will ensure, at least, this strength at large deformations, this may not be the case with the reinforcement. 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 reaches its full design strength. This may lead to a premature rupture of the reinforcement, violating the premise of the analysis and potentially resulting in local or global collapse. However, since most geosynthetics are ductile (typically having a rupture strain greater than 15%), large strains will develop locally if overstressed and thus, permit the soil to deform and mobilize its strength as assumed in the analysis. To allow for some overstressing of the reinforcement without breakage, a factor of safety to account for uncertainties in local overstressing of reinforcement is recommended. This factor should increase the required reinforcement tensile strength at each level calculated in the LE analysis. Typical values used in practice for the factor of safety for uncertainties range from 1.3 to 1.5. This larger reinforcement strength value should be available throughout the design life of the structure. Consequently, partial factors of safety for construction damage, creep, and biological and chemical degradation (Koerner 1994), should also be considered when selecting an adequate reinforcement strength.

The friction angle, δ_3 , of the block-block interface or the block-reinforcement interface should correspond to residual strength values of the interface (similar to ϕ_d). Following the schemes in Figures 3 and 5, it is apparent that the friction at the block-geosynthetic interface is utilized in defining the tributary area. Typically, this interface is weaker than the block-block interface. In the case where the interblock δ_3 is smaller, however, the smaller value should be used in the analysis. This ensures that the results are safe within the context of analysis.

Equations 5 and 6 define the factors of safety against direct sliding and overturning. Typical values used in design are $FS_{ds} = 1.5$ and $FS_{ot} = 2.0$. When calculating the anchorage length to resist pullout, a factor of safety which increases the force required by the tied-back and compound analyses, should be used. The anchorage length is then calculated for the increased load. A typical value is $FS_{po} = 1.5$.

Looking at Figure 3, it is evident that if the lowest reinforcement layer is placed at the toe, its reactive tensile force will be zero. (Typically, the lowest reinforcement layer is placed on top of the first block.) Furthermore, maximum spacing of layers is typically restricted to about 1 m. This restriction is not related to the analysis, but has to do with constructability. That is, a high stack of blocks that is not tied back, will deform outward

during compaction and thus, adversely affect the geometry of the wall. It may also cause bulging of facing blocks after construction if high walls are considered.

If a large spacing of primary reinforcement is required due to practical constraints, intermediate layers should still be specified. These layers should be at least 1 m in length. Finally, for very large interblock strengths there is a possibility that the required strength of the upper reinforcement layers will be zero (e.g. $t_b = 0$). Yet, to improve constructability and prevent overturning of the upper blocks, reinforcement layers should still be specified. The length of these layers may be dictated by the active layers below. In this case, the required length obtained from tied-back or compound analysis is controlled by the lower layers and therefore, may be significantly shorter (see Figure 9 for the effects on specified length of reinforcement).

4 PARAMETRIC STUDY

Limit equilibrium analyses yield the required strength and length of reinforcement. To realize the effects of each analysis, as well as the parameters involved, a brief parametric study was conducted. This study was restricted to a nondimensional analysis so that the observations could be generalized. Since the introduction of nondimensional facing blocks complicates the presentation of results, their weight was set to zero; hence, the effect of facia dimensions was eliminated. However, the effects of soil-block interface friction, δ_i , and subsequently, interblock friction, δ_s , are considered. It should be noted that ignoring the self-weight of the blocks decreases the interblock shear strength, V_u , and hence reduces the contribution of the facia to the stability of the structure.

Figure 10 shows the effects of δ_i , δ_s , ϕ_d and wall face inclination, i , on the reinforcement strength as obtained from a tied-back analysis. The strength is normalized by dividing the summation of required tensile resistance of all reinforcing layers, $\Sigma \bar{t}_i$, by $(\gamma H^2/2)$. In essence, the ratio defined by this normalization is analogous to the horizontal component of Coulomb's lateral earth pressure coefficient. To calculate the required tensile force from this chart, the overburden pressure, calculated at the middle of a selected tributary area of a reinforcing layer, is multiplied by its tributary area and by the coefficient K_h which can be obtained from the chart.

Clearly, when $\delta_i = 0$, the interblock friction, δ_s , has no effect on the required strength (recall that the self-weight of the block is ignored). The values of K_h are then identical to the classical Coulomb values for a vertical slope. In fact, all values of K_h for $\delta_s = 0$ coincide with Coulomb's horizontal coefficients when using the appropriate value of soil-block friction, δ_i , and for vertical walls. However, for flatter slopes (or walls), Coulomb's planar mechanism is no longer critical and therefore, it renders smaller K_h values than the log spiral surface. The results of Figure 10 show that interblock friction is most pronounced for vertical walls. For example, for $\delta_i = 20^\circ$, the required tensile resistance will decrease by about 15% as δ_s increases from 0 to 15° , and it will decrease by about 25% for the case $\delta_i = 0$, which is currently used in design.

Nondimensional results pertaining to required reinforcement length were calculated assuming equally spaced layers. In an optimal design, however, the spacing may vary to render the most economical structure. Since the required length in a practical layout depends also on the elevation of the layers, nondimensional charts (Figure 11) are of

Figure 10. Effects of δ_1 , δ_3 , ϕ_d and face inclination, i , on required reinforcement strength.

(Notes: Block self-weight is set to zero; δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; K_h = normalized strength of reinforcement; $\bar{t}_j$ = required reinforcement strength based on tied-back analysis; γ = unit weight of soil; and H = height of slope.)

limited use for a specific case; however, they are instructive in the framework of a parametric study. For the development of Figure 11, strong reinforcement layers were specified for the bottom layers in the compound stability analysis. That is, less than half of the specified layers located at the bottom were needed to resist a compound failure (see Figure 6 for realization of the significance of this choice). Consequently, the required length for the compound failure, L_c/H , is short and is dictated by the tied-back length (Figure 6) (i.e. the tied-back ("active") zone is deeper than the zone defined by the compound failure). Figure 11 shows there is a slight increase in reinforcement length as δ_1 increases. This is expected because similar to Coulomb's wedge, the extent of the slip surface increases as the soil-block friction increases. Note, however, that increases in interblock friction, δ_3 , reverses this trend. In fact, for very large interblock shear resistance (e.g. when the self-weight of the blocks is considered in the analysis), the required length may decrease significantly, becoming shorter than for the case where $\delta_1 = 0$. The example problem presented in Section 5 (Figure 17) demonstrates this phenomenon. Once again, the effects on length diminish as the inclination flattens.

Figure 12 indicates that the effects of δ_1 and δ_3 on the required length based on direct sliding analysis are very small. The slight increase in required length as δ_1 increases can be attributed to the decrease in vertical force in Wedge B due to transfer of load to the facing, through friction. This transferred load, in turn, generates sliding resistance according to the specified interblock friction, δ_3 , which in Figure 12, is always smaller than ϕ_d . Consequently, the direct sliding resistance of Wedge B combined with the interblock shear resistance, is smaller than could be generated by Wedge B alone if facing

Figure 11. Effects of δ_1 , δ_3 , ϕ_d and face inclination, i , on maximum reinforcement length, L_c .

(Notes: δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; and L_c = maximum required length obtained from compound failure or tied-back analysis; and H = height of slope.)

is ignored. It should be pointed out, however, that the reinforcement length will decrease when the self-weight of the blocks is included. Note that the factor of safety chosen was $FS_{ds} = 1.5$ and that the interwedge force inclination used was $\delta_2 = \phi_d$; smaller values of δ_2 result in longer reinforcement lengths.

Figure 13 illustrates the effects of design parameters on the required reinforcement length to resist overturning. Since the reinforced soil and the facing are viewed as one monolithic block, δ_1 and δ_3 have no effect on overturning (i.e. they generate internal forces only). Note that $\delta_2 = \phi_d$ was used meaning that the interwedge force has a stabilizing effect (vertical force component), as well as a destabilizing influence (horizontal component).

Hanks and Leshchinsky (1995) have introduced many more charts that are similar to Figures 10 through 13. They have demonstrated how to use these charts in the context of design.

Figures 14 and 15 are intended to demonstrate the effects of a pseudo-static force (i.e. "seismic" force) on reinforced vertical walls. The seismic coefficient, C_s , reflects horizontal force components that are directly proportional to gravity (i.e. weight) and surcharge loads. The various limit equilibrium analyses, which were presented schematically in Section 2, were modified to include the horizontal "seismic" force components accordingly. Such a modification, without facia effects, are found in a study by Leshchinsky (1995). Note that for a rather large coefficient, $C_s = 0.20$, there is a marked increase in the required tensile resistance, regardless of δ_1 and δ_3 . Since seismic forces act over a very short period of time, this increase does not necessarily mean that a stron-

Figure 12. Effects of δ_1 , δ_3 , ϕ_d and face inclination, i , on required reinforcement length to resist direct sliding ($FS_{ds} = 1.5$, $\delta_2 = \phi_d$).

(Notes: δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; and L_{ds} = required reinforcement length to resist direct sliding; H = height of slope; FS_{ds} = factor of safety against direct sliding; and δ_2 = interwedge force inclination.)

ger reinforcement should ultimately be selected. That is, a smaller reduction of ultimate strength due to creep is needed. The required lengths, obtained from compound/tied-back failure analysis, direct sliding analysis and overturning analysis, all increase significantly under seismic loading. The effect of δ_1 and δ_3 on reinforcement lengths are similar to those exhibited in Figures 11 through 13. The “seismic” lengths will have a major influence on the amount of reinforcement needed; yet, because of relaxing creep constraints, it may not affect the selection of a geosynthetic based on its ultimate strength.

5 EXAMPLE PROBLEMS

To demonstrate the effect of facing blocks, two examples each containing four cases, with the same simple geometry, soil properties, reinforcement spacings, and facing dimensions, are selected. The effects of soil-block friction, δ_1 , and interblock friction δ_3 , separately and combined, are illustrated by the results of the calculations in the two problem sets.

Example 1. In all cases the interwedge force inclination is assumed to be $\delta_2 = \phi_d$ and the direct sliding coefficient and interaction coefficients are taken to be $C_{ds} = 1.0$ and

Figure 13. Effects of δ_1 , δ_3 , ϕ_d and face inclination, i , on required reinforcement length to resist overturning ($FS_{ot} = 2.0$, $\delta_2 = \phi_d$).

(Notes: δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; L_{ot} = required reinforcement length to resist overturning; H = height of slope; FS_{ot} = factor of safety against overturning; and δ_2 = interwedge force inclination.)

Figure 14. Effects of δ_1 , δ_3 , ϕ_d and the seismic coefficient, C_s , on required strength and tied-back length ($i = 90^\circ$).

(Notes: δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; H = height of wall; i = face inclination; K_h = normalized strength of reinforcement; $\bar{t}_j$ = required reinforcement strength based on tied-back analysis; and γ = unit weight of soil.)

Figure 15. Effects of δ_1 , δ_3 , ϕ_d and the seismic coefficient, C_s , on required reinforcement length against direct sliding and overturning ($i = 90^\circ$, $\delta_2 = \phi_d$).

(Notes: δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; L_{ot} = required reinforcement length to resist overturning; L_{ds} = length of reinforcement to resist direct sliding; H = height of wall; FS_{ot} = factor of safety against overturning; FS_{ds} = factor of safety against direct sliding; and δ_2 = interwedge force inclination.)

$C_i = 0.8$, respectively. The allowable strength of the reinforcement was $t_{allowable} = 19.5$ kN/m. The assumed values for δ_1 and δ_3 are shown in the accompanying figures. Figure 16 shows the length of the reinforcement layers required to ensure stability against compound or tied-back failure (see Figure 6 for rationale). As can be seen, the length of reinforcement increases as δ_1 increases, that is, the soil-block friction tends to produce deeper slip surfaces. This phenomenon is the same as the one exhibited by Coulomb's solution. Conversely, the required length decreases as δ_3 increases. In fact, given the same values of friction, the effects of the interblock friction are more significant than those of soil-block friction. Note in Figure 16 that the total number of layers, m , needed to resist compound failure (see Equation 4 or Figure 6) is as follows: $m = 4$ for Case 1; $m = 3$ for Cases 2 and 3; and $m = 2$ for Case 4.

END OF EXAMPLE 1

Example 2. The value of parameters δ_2 , ϕ_d , C_{ds} , C_i , $t_{allowable}$, δ_1 and δ_3 are the same as in Example 1. Figure 17 shows the specified (practical) layout and the required allowable strength $\bar{t}_i$ of the reinforcement. Note that in all cases, the compound/tied-back failure (Figure 16) dictates the required reinforcement length. This is the typical case for vertical walls; direct sliding and overturning analyses require shorter reinforcement lengths for commonly used factors of safety. Note in Figure 17 that the surfaces in the

Soil: $\gamma = 18 \text{ kN/m}^3$, $\phi_d = \delta_2 = 32^\circ$
 Blocks: $\gamma = 24 \text{ kN/m}^3$, height = 0.2 m, depth = 0.3 m
 $C_{ds} = 1.0$, $C_i = 0.8$; $FS_{ot} = 2.0$, $FS_{ds} = 1.5$, $FS_{po} = 1.5$

Figure 16. Example 1: length of reinforcement required to resist compound or tied-back failure.

(Notes: γ = unit weight; δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; H = height of wall; FS_{ot} = factor of safety against overturning; FS_{po} = factor of safety against pullout; FS_{ds} = factor of safety against direct sliding; t^* = available (allowable) reinforcement strength (utilized in resisting compound failure); $\bar{t}_j$ = required reinforcement strength based on tied-back analysis; C_{ds} = interaction coefficient, determined from direct shear test; C_i = interaction coefficient determined from pullout test; and δ_2 = interwedge force inclination.)

Soil: $\gamma = 18 \text{ kN/m}^3$, $\phi_d = \delta_2 = 32^\circ$

Blocks: $\gamma = 24 \text{ kN/m}^3$, height = 0.2 m, depth = 0.3 m

$C_{ds} = 1.0$, $C_i = 0.8$; $FS_{ot} = 2.0$, $FS_{ds} = 1.5$, $FS_{po} = 1.5$, $t_{allowable} = 19.5 \text{ kN/m}$

Figure 17. Example 2: final required length of reinforcement following the simplified procedure in Figure 9 and showing the effects of facia.

(Notes: γ = unit weight; δ_1 = soil-block friction; δ_3 = interblock friction; ϕ_d = design angle of friction; H = height of wall; FS_{ot} = factor of safety against overturning; FS_{po} = factor of safety against pullout; FS_{ds} = factor of safety against direct sliding; t_j = required reinforcement strength based on tied-back analysis; C_{ds} = interaction coefficient between reinforcement and soil, determined from direct shear test; C_i = interaction coefficient determined from pullout test; and δ_2 = interwedge force inclination.)

tied-back analysis degenerate to planar surfaces. This phenomenon is typical for near vertical walls in which the critical log spiral tends to be defined by a pole located far away from the slope (wall). In fact, the critical surface then degenerates to Coulomb's solution with the same δ_1 . Figure 17 also shows the effect of δ_3 on both the required length and strength of reinforcement. Using reasonable design values, the length to height ratio drops from $L/H = 0.64$ to $L/H = 0.56$ when changing $\delta_1 = \delta_3 = 0^\circ$ to $\delta_1 = \delta_3 = 20^\circ$, respectively (approximately a 12% shorter required length). More significant is the drop in required strength of the most highly stressed layer based on the tied-back analysis; approximate 40% decrease. Furthermore, the required strength of the two upper layers is zero (i.e. the interblock strength is sufficient to provide the support needed to retain the soil without reinforcement tied-back). Even though this implies that the top layers do not have to be installed (i.e. no reinforcement from elevation 3.8 m to crest), practical constraints that prevent outward movement of facing during construction dictate a maximum allowable spacing that may be less than 1 m. Moreover, at the top, local overturning of upper blocks may occur. Hence, top reinforcement layers should be specified, though a weaker and shorter reinforcement may be utilized.

END OF EXAMPLE 2

Overall, the non-conservative effects of interblock friction are more significant than that of the soil-block friction. The existence of this interblock frictional force in laboratory models and full-scale reinforced walls may explain the consistent conservatism of the measured values of tensile forces in reinforcement layers when compared with predictions from commonly used limit equilibrium analyses.

6 VERIFICATION

6.1 Overview

Some of the presented design-orientated procedure has been validated using laboratory model test results (Leshchinsky and Lambert 1991). In this study, the following additional verification, against full-scale test results, was conducted: a slope having facing similar to a wrap-around type (Norwegian Wall); and a segmental wall (FHWA Wall). The verification compares the required total geosynthetic force at the end of construction (i.e. self-weight loading) and during surcharge loading. Note that for a wrap-around wall, a tributary area, different from a segmental wall (Figure 3), is utilized; the area covered by the wrapped facing for each layer (Figure 18). The rationale for such a tributary area is consistent with that presented for segmental walls (Figure 3) and is presented by Leshchinsky (1995).

6.2 Norwegian Wall

Fannin and Herman (1990) conducted full-scale tests on different sections of a 4.8 m high wall with a slope of 1H:2V (1 horizontal: 2 vertical). The results for two of the sections (Sections J and N) are reported. The backfill soil was a uniformly graded me-

Figure 18. Tributary area in wrap-around wall as compared to segmental wall.

Figure 19. Configuration of Norwegian Wall: (a) Section J; (b) Section N.

dium to fine sand, compacted to a unit weight of 17 kN/m^3 . The plane strain residual angle of friction of this soil is reported to be 38° . In Section J (Figure 19a), primary and intermediate geogrid reinforcement, of different lengths and spacings, were used. However, in Section N (Figure 19b), primary geogrid reinforcement of equal lengths were used. The force in each geosynthetic layer was measured directly using load cells. Only the facing of these sections was constructed of a wire mesh, which is considered equivalent to a wrap-around wall (i.e. $\delta_1 = \delta_3 = 0^\circ$). Following construction, the wall was surcharged at both sections with soil placed to a depth of 3 m. Since no details were given, it was assumed that the slope of this surcharge fill was 2H:1V.

The predicted tied-back failure surfaces were contained within the reinforced zone (Figures 19a and b). Table 1 compares the measured and predicted results of geosynthetic force under self-weight loading. For Section J, the total measured geosynthetic force, including that of the intermediate reinforcement layers, was reported to be 15.8 kN/m . Thus, the calculated total required tied-back force (16.8 kN/m), using the program ReSlope (Leshchinsky 1995), is in good agreement with the measured value. For Section N, as shown in Table 2, the calculated total force (17.0 kN/m) is also very close to the measured total forces (15.3 kN/m). In these walls, geogrids of relatively large strength were used (i.e. all measured strains were small indicating that the reinforcement was much stronger than needed). Consequently, all compound failure surfaces would be close to the tied-back failure surface, and therefore, be contained entirely within the reinforced zone. That is, the actual uniform reinforcement length was much longer than the required truncated length indicated in Figure 6; hence, for the layout used in the field, all compound surfaces will intersect all reinforcement layers. The compound analysis rationale implies that all layers will carry equal load (i.e. 3.36 kN/m each), exactly as would be implied by a conventional slope stability analysis (in Figure 6 the allowable strength of each specified layer is used in the LE stability analysis). Since the actual layout is not the same as required in Figure 6, back analysis using the presented design-oriented analysis can, at best, suggest a probable range of feasible values. The probable required force for each layer is then expected to lie between the required force against tied-back and compound failures (i.e. between the value needed to ensure local and global stability). In design, the maximum value of the probable range is specified. It seems that a better agreement of geosynthetic force in each layer is obtained for Section N, where the unaccounted effects of intermediate layers were not present, when compared to Section J. During surcharge loading, the total required forces for Section J and N were calculated to be 20.8 kN/m and 21.1 kN/m , respectively. These values agree well with the measured values: 21.0 kN/m and 22.22 kN/m , respectively.

Note that formally, the LE analysis has to be verified against structures at the limit state. For very steep granular slopes, it is assumed that sufficient soil movement occurs to allow the mobilization of the friction angle. At this stage, the stability of the reinforced structure is dependent upon the available strength of the reinforcement. The force in the reinforcement then leads to the minimum required strength (i.e. for this minimum, a system in a LE state will be produced). The minimum strength of each layer is the measured value and, for comparison purposes, the calculated value. Consequently, the presented comparison is valid. Since the reinforcement used was stronger than that required for a LE state, the available factor of safety with respect to geosynthetic strength can be estimated.

Table 1. Reinforcement forces in Norwegian Wall under self-weight loading: Section J.

Layer no. j	Elevation (m)	Calculated*			Measured**
		Tied-back failure t_j (kN/m)	Compound failure $\Sigma t_j/n$ (kN/m)	Probable range of required force (kN/m)	Reinforcement force (kN/m)
1	0.0	4.1	3.36	3.36 - 4.1	1.10
2	0.6	3.5	3.36	3.36 - 3.5	2.13
3	1.2	5.2	3.36	3.36 - 5.2	2.4
5	2.4	3.0	3.36	3.0 - 3.36	2.03
7	3.6	1.0	3.36	1.0 - 3.36	1.83
Σt_j		16.8	16.8	-	9.49 (15.8)***

Notes: * Calculated ignoring intermediate layers; ** values were not corrected to account for loads carried by intermediate layers; and *** Σt_j = sum of reinforcement forces, corrected by Fannin and Hermann (1990) to account for contribution from the intermediate reinforcement layers.

Table 2. Reinforcement forces in Norwegian Wall under self-weight loading: Section N.

Layer no. j	Elevation (m)	Calculated			Measured
		Tied-back failure t_j (kN/m)	Compound failure $\Sigma t_j/n$ (kN/m)	Probable range of required force (kN/m)	Reinforcement force (kN/m)
1	0.0	4.1	2.13	2.13 - 4.1	1.06
2	0.6	3.5	2.13	2.13 - 3.5	2.25
3	1.2	2.9	2.13	2.13 - 2.9	2.01
4	1.8	2.4	2.13	2.13 - 2.4	2.34
5	2.4	1.8	2.13	1.8 - 2.13	2.00
6	3.0	1.3	2.13	1.3 - 2.13	1.46
7	3.6	0.8	2.13	0.8 - 2.13	1.92
8	4.2	0.2	2.13	0.2 - 2.13	2.26
Σt_j		17.0	17.0	-	15.3

Figure 20. Configuration of the FHWA Wall.

6.3 FHWA Wall

Bathurst et al. (1993) reported the results of a near-vertical wall having a segmental block facing (Figure 20). The wall was 6.1 m high. The unit weight of the backfill soil was 20.1 kN/m³ and it had an internal friction angle of 40°. Each segmental block weighed 47.5 kg and had dimensions of 200 mm × 400 mm × 600 mm (height × depth × width). At the end of construction, it was surcharged using 2.1 m of fill. The strains measured in the geogrid were used by the writers to estimate the forces based on the reported wide-width tensile test results. Note that confinement effects and soil interaction were ignored when converting measured strains to force. Min et al. (1995) have demonstrated that the modulus of a geogrid is an intrinsic material property; hence, for a given strain, there corresponds only one force that can be determined directly from the load-elongation curve. The measured strains were smaller than 1% indicating the geogrid used was much stronger than that required. Since information on the block-soil and interblock interaction was not available, analyses were conducted for $\delta_1 = \delta_3 = 0^\circ$ and $\delta_1 = 40^\circ$, and $\delta_3 = 20^\circ$. The weight of the segmental blocks was included.

Figure 20 shows that the tied-back failure surfaces for self-weight and surcharge loadings are well within the reinforced zone. Once again, due to strong reinforcement, all compound failures are forced to be near the tied-back failure surface and thus pass through all reinforcement layers. The calculated and measured results are shown in Tables 3 and 4 for self-weight and surcharge loadings, respectively. It is seen that if the interblock and block-soil interface shear capacities are neglected, the total required reinforcement force is more than 4 to 5 times greater than measured or as calculated using $\delta_1 = 40^\circ$, $\delta_3 = 20^\circ$. This demonstrates the overconservatism of design methods that fail to account for the facia effects. Since values of δ_1 and δ_3 are not available, further discussion cannot be conducted in this paper. Furthermore, there are other potential factors other than the facing that may have influenced the test results; and consequently, additional verification is needed.

Table 3. Reinforcement force in FHWA Wall under self-weight.

Layer no. j	Elevation (m)	Calculated			Measured
		Tied-back failure t_j (kN/m)	Compound failure $\Sigma t_j/n$ (kN/m)	Probable range of required force (kN/m)	Reinforcement force (kN/m)
1	0.2	2.3* (5.2)**	1.98*	1.98 - 2.3*	0.7
2	0.8	5.0 (14.7)	1.98	1.98 - 5.0	2.4
3	1.4	4.2 (13.1)	1.98	1.98 - 4.2	2.6
4	2.0	3.4 (11.5)	1.98	1.98 - 3.4	2.8
5	2.6	0.9 (10.0)	1.98	0.9 - 1.98	2.3
6	3.4	0.0 (10.8)	1.98	0 - 1.98	1.8
7	4.4	0.0 (9.6)	1.98	0 - 1.98	1.2
8	5.2	0.0 (6.3)	1.98	0 - 1.98	0.5
Σt_j		15.8 (81.2)	15.8	-	14.3

 Notes: * $\delta_I = 40^\circ$ and $\delta_3 = 20^\circ$; and ** $\delta_I = \delta_3 = 0^\circ$.

Table 4. Reinforcement force in FHWA Wall under surcharge.

Layer no. j	Elevation (m)	Calculated			Measured
		Tied-back failure t_j (kN/m)	Compound failure $\Sigma t_j/n$ (kN/m)	Probable range of required force (kN/m)	Reinforcement force (kN/m)
1	0.2	3.1* (6.7)**	4.4	3.1 - 4.4	1.2
2	0.8	7.5 (19.3)	4.4	4.4 - 7.5	4.3
3	1.4	6.7 (17.4)	4.4	4.4 - 6.7	4.4
4	2.0	5.8 (15.7)	4.4	4.4 - 5.8	4.5
5	2.6	4.9 (14.2)	4.4	4.4 - 4.9	4.4
6	3.4	5.1 (15.8)	4.4	4.4 - 5.1	4.3
7	4.4	2.0 (14.6)	4.4	2.0 - 4.4	3.1
8	5.2	0.0 (9.6)	4.4	0 - 4.4	1.6
Σt_j		35.1 (113.3)	35.1	-	27.8

 Notes: * $\delta_I = 40^\circ$ and $\delta_3 = 20^\circ$; and ** $\delta_I = \delta_3 = 0^\circ$.

7 CONCLUSIONS

Different limit equilibrium analyses were modified to account for facing block effects. These analyses provide the required reinforcement strength and length, considering several possible modes of failure. The design-oriented analyses are consistent with each other and are equally valid for steep slopes and walls. When the facing blocks are sufficiently large, the analyses indicate that no reinforcement is needed (i.e. a stable gravity wall is produced). When flat slopes are considered, the analyses will show that no reinforcement is needed. The analyses were partially verified against full-scale test results and proved satisfactory.

Although not explicitly demonstrated in this paper, the writers have verified that for vertical walls having the same design parameters, the internal stability analysis yields identical results to the National Concrete and Masonry Association (NCMA) Manual (Simac et al. 1993); i.e. same required strength and same outer-most slip surface. In fact, the required strength diverges from the NCMA method by approximately 5% when the slope angle is about 75° (i.e. the NCMA results are less critical). For slope angles less than 75° this divergence increases rapidly. This shows that while the NCMA approach is adequate for walls, the presented approach, which utilizes log spiral failure mechanisms, is more general thus making it suitable for slopes as well as for walls. Consequently, a uniform design for walls and slopes has been introduced.

A parametric study indicates that facia effects are most significant when very steep granular slopes are considered; i.e. walls. When only steep slopes are analyzed, the effects of block-soil friction and interblock friction are small. The example problems demonstrate that interblock friction has a significant influence on the design of walls: i.e. shorter and weaker reinforcement is required. The soil-block friction decreases the required strength, but increases the required reinforcement length. The analyses were partially verified against full-scale test results and proved satisfactory.

Finally, knowing that the premise of the analyses is a limiting equilibrium state in which all strength components contribute full resistance simultaneously, a methodology for selecting strength parameters is recommended. These parameters should correspond to residual strength conditions.

ACKNOWLEDGEMENTS

The design approach presented in the paper is the result of an evolutionary process over a period of ten years. The merging of reinforced slopes and walls within the same design framework was made possible in part by funding from the Waterways Experiment Station (WES), US Army Corps of Engineers (Contract No. DACW39-94-C-0073) and from the Delaware Transportation Institute (DTI). The DTI is an institute sponsored by the University of Delaware and the Delaware Department of Transportation (DelDOT). The writers gratefully acknowledge this support.

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NOTATIONS

Basic SI units are given in parentheses.

A	=	constant of log spiral (dimensionless)
C_{ds}	=	interaction coefficient between reinforcement and soil, determined from direct shear test (dimensionless)
C_i	=	interaction coefficient relating the reinforced soil interface strength to the reinforced soil strength determined from pullout test (dimensionless)
C_s	=	seismic coefficient (dimensionless)
FS_{ds}	=	factor of safety against direct sliding (dimensionless)
FS_{ot}	=	factor of safety against overturning (dimensionless)
FS_{po}	=	factor of safety against pullout (dimensionless)
H	=	height of wall/slope (m)
i	=	wall/slope inclination from horizontal ($^{\circ}$)
K_h	=	normalized strength of reinforced layer = $\Sigma \bar{t}_j / (0.5\gamma H^2)$ (dimensionless)

L_B	=	longest reinforcement length determined from compound, tied-back, direct sliding and overturning analyses, specified at the wall/slope bottom (m)
L_c	=	maximum required reinforcement length obtained from compound failure or tied-back analysis (m)
L_{ds}	=	reinforcement length required to ensure stability against failure due to direct sliding (m)
L_{ot}	=	reinforcement length required to resist overturning (m)
L_T	=	reinforcement length obtained from compound and tied-back failure analyses, specified at the wall/slope crest (m)
L_1	=	width of horizontal portion of backfill surface (Figure 1) (m)
L_2	=	length of sloped portion of backfill surface (Figure 1) (m)
M_D	=	driving moments (N-m/m)
M_R	=	resisting moments (N-m/m)
M_P	=	moment about the point P (Figure 2) (N-m/m)
m	=	number of layers required to ensure compound stability (dimensionless)
N	=	force due to self-weight of stacked blocks and the induced force due to friction between the soil and blocks (N/m)
N_A	=	reaction force of retained soil, Wedge A (Figure 7) (N/m)
N_B	=	base reaction force of reinforced soil, Wedge B (Figure 7) (N/m)
n	=	total number of reinforcement layers (dimensionless)
P	=	interwedge force (N/m)
P_{max}	=	maximum interwedge force (Figure 7) (N/m)
Q	=	surcharge load (N/m ²)
RD	=	reduction factor (dimensionless)
r_1	=	$A \exp(\psi \xi_1)$, log spiral radius at ξ_1 (Figure 2) (m)
r_2	=	$A \exp(-\psi \xi_2)$, log spiral radius at ξ_2 (Figure 2) (m)
SV_n	=	tributary area, n (dimensionless)
T_A	=	sliding resistance force of retained soil, Wedge A (Figure 7) (N/m)
T_B	=	base sliding resistance force of reinforced soil, Wedge B (Figure 7) (N/m)
$t_{allowable}$	=	allowable reinforcement strength (N/m)
t_{nmax}	=	maximum tensile force in reinforcement layer n (N/m)
t_j	=	tensile reactive force in reinforcement layer j (N/m)
$\bar{t}_j$	=	reactive force in reinforcement layer j corrected for interblock shear strength (N/m)
Vu	=	interblock shear strength (N/m)
W	=	self-weight of soil mass (N/m)
W_A	=	self-weight of retained soil, Wedge A (Figure 7) (N/m)
W_B	=	self-weight of reinforced soil, Wedge B (Figure 7) (N/m)

x	=	distance in horizontal direction (m)
$\bar{Y}$	=	height at which interwedge force acts (m)
Y_j	=	vertical distance from slope base to reinforcement layer j (Figure 2) (m)
y	=	distance in vertical direction (m)
β	=	slope of backfill from horizontal (Figure 1) ($^\circ$)
ξ	=	log spiral angle (Figure 2) ($^\circ$)
δ_1	=	friction angle between reinforced soil and facing block ($^\circ$)
δ_2	=	interwedge force inclination (Figure 7) ($^\circ$)
δ_3	=	friction angle between facing block and reinforcement layer, or between block and block, whichever is smaller ($^\circ$)
ϕ	=	soil internal angle of friction ($^\circ$)
ϕ_d	=	design angle of friction (residual value is recommended) ($^\circ$)
ϕ_m	=	mobilized soil friction angle ($^\circ$)
ϕ_{peak}	=	peak soil friction angle ($^\circ$)
γ	=	unit weight of soil (N/m ³)
θ_{cr}	=	slope angle of retained soil Wedge A producing P_{max} (Figure 7) ($^\circ$)
$\sigma(\xi)$	=	normal stress over slip surface (Figure 2) (N/m ²)
$\tau(\xi)$	=	shear stress along slip surface (Figure 2) (N/m ²)
ψ	=	$\tan\phi_d$ (Figure 2) (dimensionless)