Some Issues in Geosynthetic Reinforced Walls and Slopes

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ABSTRACT: Current design of geosynthetic reinforced soil is well-established, rendering safe and economical structures. However, there are some issues that need attention so as to improve the economics or to avoid pitfalls. This paper presents three such issues suggesting possible solutions and commentary. The first issue deals with the artificial definition of reinforced walls and reinforced slopes. The distinction is based on an arbitrary slope angle. Such division results in two incompatible design methodologies. For reinforced walls the required strength of reinforcement is as much as twice as that needed for slopes; however, the trade off is simpler and more transparent calculations. With the availability of computer codes and with entrusting geotechnical engineers (as opposed to structural engineers) to design walls, it is expected that the less conservative and more consistent approach for reinforced slopes will be adopted for walls. One possible approach is using the safety map approach. The second issue deals with the current seismic design of geosynthetic reinforced walls. This design actually inhibits the use of such walls in seismic areas. However, field experience indicates that such walls actually behave very well under seismic loads. Its inherent flexibility produces a 'shock absorbing' type of structure that can dissipate seismic energy. Presented are the results of large scale shake table tests demonstrating the performance of geosynthetic reinforced retention structures. An alternative pseudostatic design approach, including reduced seismic coefficients, is proposed as a conclusion. The third issue deals with observations of 'smaller than expected' field measured load in geosynthetic reinforcement. These measurements have resulted in 'calibration' of a new design methodology that completely ignores statics and entirely relies on statistics. While the motivation to improve very conservative designs is understandable, the alternative of completely discarding the principles of static may result in unsafe structures. It is shown that a clear and simple explanation for the apparent conservatism is due to apparent cohesion which is generated by soil matrix suction. Without this cohesion, which is likely to disappear during the life span of the reinforced structure, the statistically-based approach yields a structure that is globally unsafe. Hence, in the context of design, the statistical approach without a benchmark based on statics, is unsafe. To get the full perspective of all three issues, references giving further elaboration are provided.

1. INTRODUCTION

Current design of geosynthetic reinforced soil is well-established, rendering safe and economical structures. Many thousands of reinforced structures validate this statement. However, there are some issues that need attention so as to improve the economics or to avoid pitfalls. This paper presents three such issues suggesting possible solutions and commentary. The first issue deals with the artificial definition of reinforced walls and reinforced slopes. The second issue deals with the current seismic design of geosynthetic reinforced walls. The third issue deals with observations of 'smaller than expected' field measured load in geosynthetic reinforcement. To get the full perspective of all three issues, references giving further elaboration are provided.

2. ISSUE: CATEGORIZATION OF REINFORCED WALLS AND SLOPES

2.1 Background

AASHTO as well as other national codes base their design calculations for geosynthetic reinforced walls on lateral earth pressure. The attractive feature of lateral earth pressure theory (e.g., Rankine) is its simplicity. The calculations are such that no computer is needed (an important factor when the approach was formalized in the 70's). Its application is limited to walls. Moreover, even for simple geometry, reinforced walls grossly violate the conditions for which Rankine's lateral earth pressure theory is valid (e.g., the presence of reinforcement layers changes the principal stress orientation within the mass). Careful attempts to measure it on the back of the facing blocks (e.g., Ling et al. 2005) demonstrate its elusive value. Clearly, the lateral earth pressure approach cannot be extrapolated to slopes and multitiered walls as its association with planar slip surfaces quickly becomes invalid (unconservative). The alternative to lateral earth pressures is the use of limit equilibrium (LE). This analysis quantifies the margin of safety of the system against a state of imminent failure. It has been used successfully for design of major geotechnical structures for decades. One limitation of limit equilibrium in conjunction with reinforced soil applications is its inability to predict the force distribution among layers (e.g., Leshchinsky et al, 2010a). However, experience with geosynthetic-reinforced earth structures shows that its maximum mobilized strength for all layers is approximately uniform (e.g., Leshchinsky et al, 2010b). Hence, the practice of geosynthetic-reinforced slope design allows the use of properly modified slope stability analysis to account for the reinforcement effects while assuming that each reinforcement layer will contribute its long term strength (e.g., Elias 2001). In fact, comparison between the global stability predicted by a common limit equilibrium analysis (Bishop Method) and a continuum mechanics-based analysis which account for local conditions shows good agreement (e.g., Leshchinsky and Han 2004).

2.2 The Problem

Current design methods (e.g., AASHTO 2002; BSI 1995) consider geosynthetic reinforced walls as structures with a batter less than 20° whereas geosynthetic reinforced slopes are inclined at an angle of 70° or less. Obviously, such distinction is arbitrary as there is nothing magical with the 70° boundary. Further, design of reinforced walls commonly utilizes lateral earth pressures whereas design of reinforced slopes uses LE (slope stability) type of analysis. Consequently, there is an abrupt change in the required reinforcement strength and length when reaching the arbitrary boundary between so-called reinforced walls and slopes. No rational explanation for this incompatible behavior, basically designing the same structure, exists. As an example, the maximum force in reinforcement based on lateral earth pressures can be as much as two times that of LE.

It should be noted that other aspects related to design of slopes and walls are extremely different. The amount of fines FHWA allows for walls is limited to 15% whereas for slopes it is 50%. This has major cost implications. Seismic design for walls is by far more severe for walls than for slopes. It is no wonder that in some regions within the US some producers use slopes inclined at 69.5°, taking advantage on arbitrarily, seemingly irrational rules.

Selection of the magic boundary of 70° has to do with the well-known fact that planar surface to define the active wedge is reasonably accurate for slopes of 70° and higher. Hence, rather than using computerized optimization as done in slope stability analysis, simple optimal equation such as that of Rankine's is rendered.

Though such approach is reasonable in terms of total reinforcement force, it poorly estimates the force for an individual layer; in lower layers it overestimates the required force by as much as a factor of two. Furthermore, it resulted in what is called internal stability (needed to determine the reinforcement strength, the strength of its connection to the facing, and the pullout resistive length) and external stability (considering the reinforced soil as a coherent mass that must not slide, overturn, or fail in bearing capacity mode). However, what was a reasonable approximation in the 70's is no longer reasonable with the availability of computerized slope stability analysis. In such analysis, there is no need for prescribed inclination of active wedge, and no need for internal and external stability analyses.

It is interesting to note that many design methodologies based on lateral earth pressure prescribe the length for reinforcement to be at least 0.7H (L/H=0.7). The likely explanation for this 'magic' number has to do with the development of design of MSE walls introduced by the Reinforced Earth Company. Since design is based on iterative calculations and since much of it was done by hand in the 70's, a convenient benchmark to start the calculations was L/H=0.7. Such selection ensured rapid convergence to the optimal length of reinforcement. It also eliminated major hand calculations errors as for a typical backfill (where ϕ was assumed as 30°), this ratio is likely to be close to the final value. Hence, as a matter of 'insurance', the L/H ratio was set to 0.7 even of calculations would show smaller required value. This ratio now is being used in many codes as if it is an absolute number stemming from some mathematical calculations. While its value is likely appropriate as a benchmark for metallic reinforcement, it is questionable whether it should be imposed on geosynthetic reinforcement. Computerized calculations show that for high quality backfill this L/H ratio can be as low as 0.5, substantially shorter than 0.7.

2.3 Solution

Han and Leshchinsky (2006) presented a unified solution that is applicable for both 'slopes' and 'walls'. It uses a safety map where the factor of safety on the soil shear strength is set to constant value (say, Fs=1.3). For that Fs, the required strength of reinforcement is determined at each location. It renders the required connection strength, the maximum required strength of each layer, and the required length of each layer to ensure sufficient anchorage. It considers multilayer interaction and is applicable for any slope angle. The analysis is as simple and kept in the context of LE. The details provided by Han and Leshchinsky (2006) are sufficient for one to develop a general computer program so as to enable the application of this rational approach to any problem. It will require intensive computations; however, this is inexpensive and nowadays can be done rapidly. Consequently, while the framework for this unified approach was provided by Han and Leshchinsky (2006), there is a need for special computer program that can be used to implement this approach. Without a comprehensive unified approach, the irrational division to reinforced slopes and walls will likely continue, yielding further patches which are even more irrational (e.g., K-stiffness method; see the third issue).

3. ISSUE: SEISMIC DESIGN

3.1 Background

Ideally, the design of any structure subjected to earthquakes should be based on tolerable recoverable and/or permanent displacements. This approach is difficult to implement for reasons such as a lack of acceptable criteria for tolerable displacements, highly random nature of seismic excitation, inaccurate identification of in situ soil constitutive behavior, and numerical difficulties in predicting displacements within the matrix soil-geosynthetic. The state-of-the-art in seismic slope stability analysis is not yet sufficiently developed to entirely replace the current design practice.

FHWA/AASHTO design of MSE walls is based on hybrid approach (e.g. FHWA 2001). It directly adopts the Reinforced Erath company approach which is based on numerical work as well as an approach for gravity walls. First, the Peak Ground Acceleration (PGA) for design is amplified by a factor of (1.45-PGA) while limiting PGA to maximum value of 0.3g. For PGA>0.3g a displacement analysis is recommended (no guidelines how to do such an analysis are provided). Then internal stability is conducted. The mass of the 'active wedge' is used to find the lateral pseudo static load. This load is then divided amongst all reinforcement layer based on their actual anchored length resisting pullout. For external stability, half of the added pseudostatic load (i.e., ΔPae/2) due to the retained soil plus a pseudostatic load representing the inertia of portion of the reinforced mass are added when assessing resistance to sliding, overturning (or eccentricity) and bearing capacity. This external pseudostatic load is acting at 0.6H while the static load is acting at H/3. Essentially, the AASHTO approach is based on lateral earth pressures. It is made up of patching of unrelated approaches to render a unified seismic approach for design of all reinforced walls. To a large extent it is irrational but nevertheless it represents a first formal step. Generally, experience shows that AASHTO approach hinders the acceptance of geosynthetic reinforced walls in seismic zones by either rendering overly conservative designs or being complicated to apply.

The alternative to lateral earth pressures would be a slope stability approach. Such an approach offers immediate integration of reinforced wall design with existing sound practice. Design of slopes is typically based on limit equilibrium (LE) stability analysis. Pseudostatic slope stability analysis assumes an equivalent seismic coefficient, typically in the horizontal direction, which results in additional force components in the LE equations, all proportional to gravity. Specifying the seismic coefficient as peak ground acceleration (PGA) is likely overly conservative as it considers the maximum seismic forces permanent rather than momentary. The objective of this study was to quantify a reasonable reduction factor (RF) on the PGA for geocell retention structures. Reduced factors can then be integrated with well-established LE analysis to conduct seismic and static design. Implications of this research are relevant o geosynthetic reinforced walls.

3.2 Shake Table Testing

Details of this work are presented by Leshchinsky et al (2009) and Ling et al (2009). The shake table utilized is located at the Japan National Research Institute of Agricultural Engineering, Tsukuba City. It can excite gross maximum payload of 500kN to vertical and/or horizontal acceleration of 1g; maximum accelerations for lighter payloads can be larger than 1g. The metal testing box containing the geocell retention systems was 2m wide, 6m long, and 3m tall. To minimize reflection of waves from the side and rear of the metal box, expanded polystyrene (EPS) boards, 5cm thick, were placed against the testing box walls. To reduce friction with the sidewalls, greased plastic sheeting was placed against the EPS.

In all tests, an amplified time record of the 1995 Kobe earthquake was applied to the shake table. The Kobe record used had horizontal PGA of 0.59g and a vertical PGA of 0.34g. The peak horizontal and vertical accelerations did not occur simultaneously. Table 1 shows the applied peak accelerations in four different tests.
There were either two or three loading stages. In the first loading stage, the Kobe record was attenuated in an attempt to verify whether excessive movements occurred. An hour later the second loading stage was applied, amplifying the Kobe record. In Tests 3 and 4, a third excitation was applied, this time reaching the capacity of the shake table. The third stage nearly doubled the Kobe recorded acceleration. Stage 2 was aimed at developing an active wedge; it was hoped that the third stage would bring about collapse. The geometry of each tested retention system is shown in Figure 1 (a-d).

Table 1. Applied Peak Acceleration in Four Tests

Recorded Peak Ground Acceleration (PGA) in the Field	Test No.	Applied Peak Acceleration at Base of Shake Table					
		Horizontal			Vertical		
		Loading Stage:			Loading Stage:		
		1	2	3	1	2	3
Horizontal: 0.59g Vertical: 0.34g	1	*0.46g*	*0.92g*	*N/A*	*0.21g*	*0.42g*	*N/A*
	2	*0.48g*	*0.94g*	*N/A*	*0.20g*	*0.39g*	*N/A*
	3	*0.47g*	*0.95g*	*1.22g*	*0.20g*	*0.37g*	*0.48g*
	4	*0.41g*	*0.87g*	*1.21g*	*0.18g*	*0.34g*	*0.50g*

In Tests 1-3, the retention system was 2.8m high; in Test 4 it was 2.7m. All retention systems were constructed over a 0.2m-thick foundation soil. The HDPE geocells, resembling a honeycomb structure, were 0.2m high with internal aperture of approximately 0.21m by 0.21m. The average face inclination of the systems was 2(v):1(h). The top geocell layer was 2.52m long, much longer than all layers below. This top layer was infilled with compacted gravel. It was assumed that long top layer made of geocell would inhibit crack or even slip surface formation immediately below this layer. Indeed, tests indicated that while numerous small and shallow tension cracks initiated at the crest due to seismic excitation, none was observed immediately below the long top geocell layer in any of the tests, thus supporting the initial assumption.

Tests 1-2 represented flexible gravity walls and Tests 3-4 utilized geocell as reinforcement and facing. In terms of economics, the systems in Tests 3 and 4 are about the same. In Test 4 the layout of geocell resembled that of traditional geogrid reinforcement while still acting as 3-D element. Generally, the polyethylene geocell used in the tests cannot be used as reinforcement for sizeable structures since it has low long-term tensile strength. As tested, only sufficient short-term properties were needed to resist the seismic loading. However, the lessons should indicate the needed product improvements in developing a geocell made for reinforcement as well as produce a simple design methodology.

The backfill soil behind the facing and in the 0.2m-thick foundation was fine uniform sand (Median Grain Size = 0.27mm; 0.35% passing sieve #200; Uniformity Coefficient = 2). The backfill was compacted to 90% of Standard Proctor at a moisture content of 16% yielding a dry unit weight of 13.5 kN/m^3 or moist unit weight of 15.6 kN/m^3. Compaction was done by a handheld vibratory compactor. Drained triaxial tests yielded peak strength of ϕ= 38°. Unit weight of the compacted gravel was 19.9 kN/m^3.

Thin white seams of sand were placed every about 0.4m within the backfill material. Upon completion of each test, the backfill was carefully excavated to observe dislocations of these seams so that traces of slip surfaces could be identified. In addition, each test was comprehensively instrumented including pressure transducers, laser displacement gages, accelerometers, and strain gages (Ling et al, 2009).

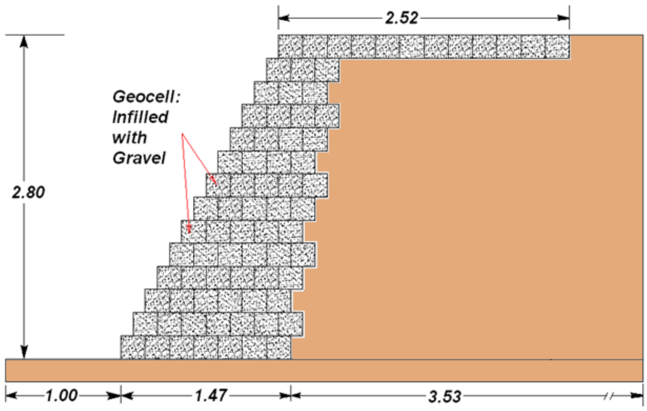

Figure 1a. Test 1: Gravity Wall with Gravel Infilled Geocell

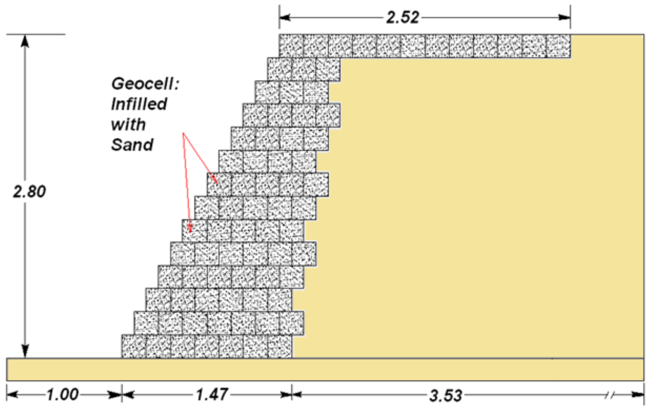

Figure 1b. Test 2: Gravity Wall with Sand Infilled Geocell

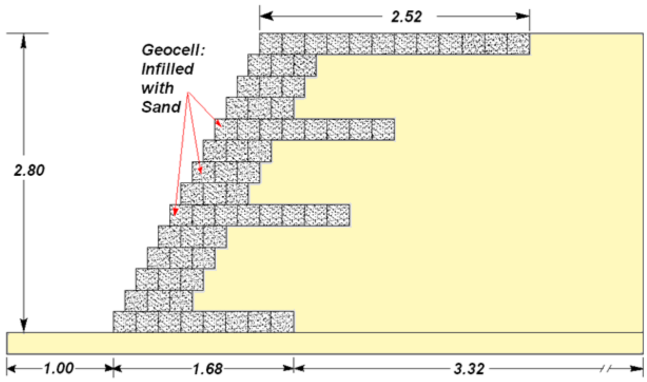

Figure 1c. Test 3: Geocell Reinforcement Infilled with Sand

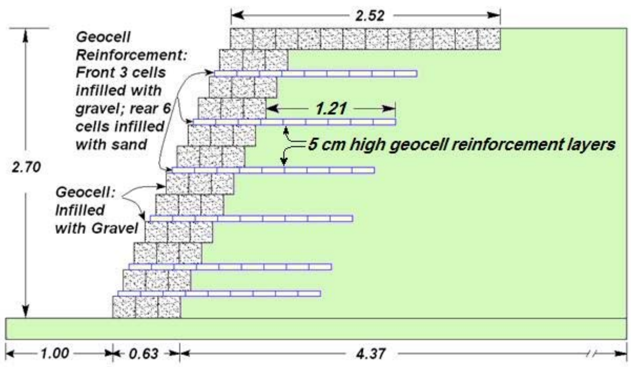

Figure 1d. Test 4: 5 cm High Geocell Reinforcement Infilled with Sand

Figure 1. Configuration of Tested Systems

3.3 Results and Interpretation

Accelerometers embedded within the backfill soil and facing, at several elevations, indicate that magnification of base acceleration was negligibly small. This may not be surprising with flexible retention systems as they deform during shaking, dissipating energy and acting as shock absorbers.

Table 2 shows the measured maximum displacements in each one of the tests. Note that displacements were not uniform and, therefore, the term maximum represents a rather narrow zone where it occurs. Also note that for Tests 1 and 2, the maximum applied acceleration was significantly lower than that for Tests 3 and 4 (see Table 1). Overall, considering the severity of the applied seismic excitation, the recorded values do not imply a catastrophic failure (e.g., see Figure 2 for typical post-shaking appearance).

Table 2. Measured Maximum Displacements

Test Number	Maximum Horizontal Permanent Displacement of Face [mm]	Maximum Settlement of Crest [mm]
1	31	27
2	47	40
3	150	150
4	95	85

Figure 2. Test 2: Post earthquake (159% of Kobe's PGA) Frontal View – see excavated section of same wall in Figure 3a.

Generally, the displacements reflect a well-developed active wedge where the shear strength of the soil is mobilized. Sufficient strength and stiffness of a geocell will enable acceptable structural long-term performance with even smaller displacements. Post-test exhumation of the retention systems while measuring dislocations of the white sand seams helped in establishing the location of the active wedge surface (e.g., see Figure 3, a–c).

This enables complete limit equilibrium (LE) stability analysis where the soil strength is fully mobilized rendering an active wedge, meaning the factor of safety on soil strength, Fs, equals unity. To find an equivalent seismic coefficient for design, it is convenient to define seismic reduction factor, RFs=a/PGA, where a is the equivalent pseudostatic seismic coefficient. RFs for each test was determined using the recorded PGA that caused an active wedge to develop without rendering excessively large displacement combined with an adequate LE analysis.

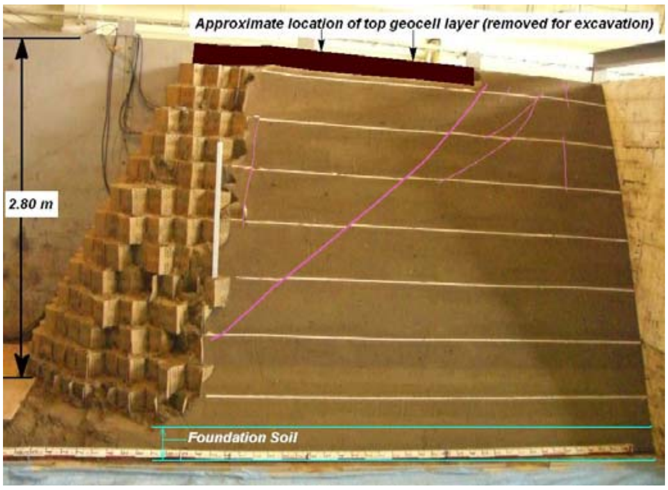

Figure 3a. Test 2 (Applied Excitation was 159% of Kobe's)

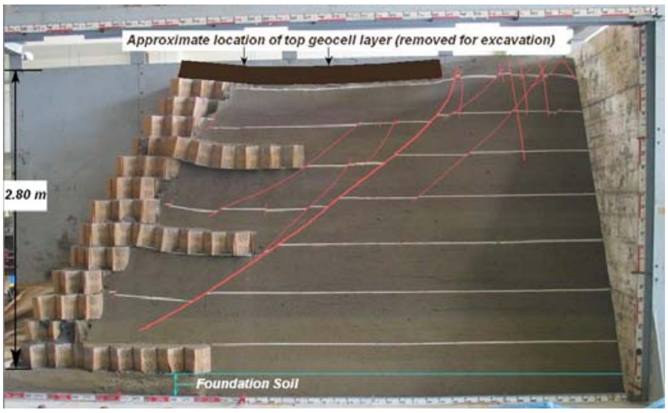

Figure 3b. Test 3 (Applied excitation was 205% of Kobe's).

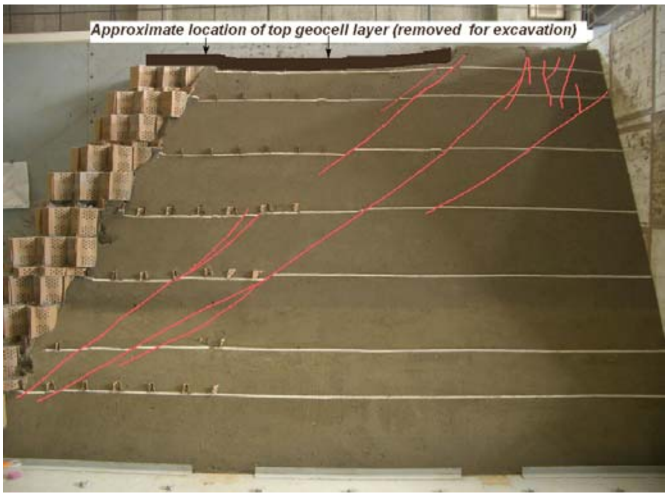

Figure 3c. Test 4 (Applied Excitation was 205% of Kobe's; note the sections through the 5 cm high geocell reinforcement).

Figure 3. Post-Shaking Exhumed Sections through Backfill and Geocell (Note: Dislocations of white sand seams indicate locations where slip surface developed and soil strength was fully mobilized)

The pseudostatic acceleration in the LE analysis was adjusted to render Fs of unity; i.e., to reflect the existence of an active wedge. The locations of the predicted and observed active wedges were compared and used to assess the predictive value of the analysis. It is noted that in LE design, one would input a(=RFs PGA) to obtain adequate seismic stability where the factor of safety, Fs, under pseudostatic conditions is typically about 1.1. In fact, if one had to design the tested retention systems, use of RF and Fs>1.1 would have produced smaller displacements than those reported in Table 2.

LE stability analysis was performed using program ReSSA (Leshchinsky and Han, 2004). Rotational (Bishop) and translational (Spencer) analyses were conducted to determine the RFs. The safety map feature (Baker and Leshchinsky, 2001) facilitated the process. (For example, see Figure 4).

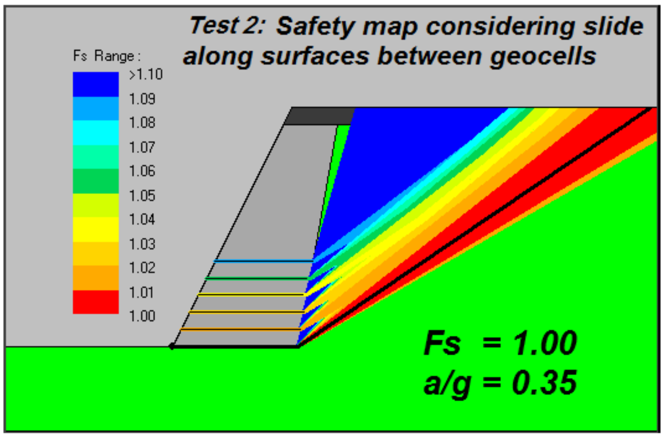

Figure 4. Test 2: Safety Map Rendered by Program ReSSA(3.0) Using Spencer Method and Pseudostatic Analysis

While the observed slip surface emerged between the second and third geocell facing layer, the numerically predicted surface (at *a*/PGA=0.35) emerged along the interface between the geocell and the foundation soil. However, the safety map shows that practically this is an insignificant difference, as the safety factors for any predicted slip surface emerging at the lower geocell layers is within about 1–2%. Such an observation affords confidence in the predictions, especially when comparing Figures 3a and 4; i.e., the observed and predicted traces of slip surfaces, respectively. Figures 5a and 5b show the predicted active wedges and their respective RFs values; they can be compared with the observed wedges shown in Figures 3b and 3c, respectively.

Apropos Figures 3c and 5b: As can be seen, contrary to a common legend, these figures demonstrate that slip surfaces can develop through the reinforcement. Such "internal" global instability can occur when the reinforcement is too soft or weak. Clearly, while the HDPE geocell used was adequate to test a design-oriented analysis, it lacks long-term strength to serve as reinforcement. However, it enables one to establish the desired properties in geocells so it can serve as soil reinforcement.

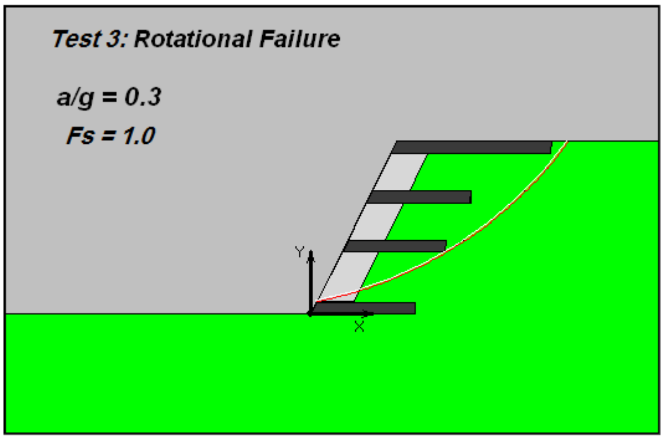

Figure 5a. Test 3: Predicted Active Wedge using Bishop Analysis

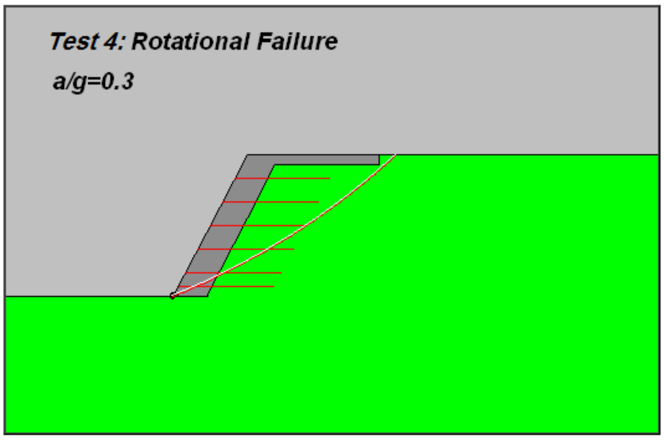

Figure 5b. Test 4: Predicted Active Wedge using Bishop Analysis

Figure 5. Predicted Critical Slip Surfaces in Geocell Reinforced Retention Systems

Table 3. Seismic Reduction Factors

Test Number	Seismic Reduction Factor RFs=*a*/PGA
1	0.38
2	0.37
3	0.25
4	0.25

As can be seen from Table 3, for geocell gravity systems, RFs of about 0.4 are adequate. For geocell-reinforced soil systems, RFs of 0.3 are adequate.

3.4 Commentary

Current practice of designing reinforced or unreinforced slopes and walls is to identify the local PGA and use a fraction, RFs, of it in a pseudostatic analysis. This fraction is the reduction factor for pseudostatic analysis. In reinforced walls AASHTO is recommending RFs>1.0 using a rather convoluted and fragmented analysis. There is no need for distinction between reinforced 'walls' and reinforced 'slopes'. This works confirms that current slope stability practice is also applicable to geosynthetic reinforced soil structures.

The Kobe earthquake was used as a reference for an excitation to identify this coefficient. It is likely that if another excitation was used, the reduction factor would be different. However, the Kobe earthquake was significant in terms of damage to slopes and walls, thus qualifying it to serve as a good reference for calibrating this reduction factor and the associated seismic coefficient.

Tests results are compared with a pseudostatic limit equilibrium analysis. The predicted failure mechanisms are similar to those observed in the tested geocell retention systems. The seismic coefficients required to produce failure in the analysis were much smaller than the actual peak value obtained in the tests. For the geocell gravity wall, the seismic reduction factor, RFs, needed to render failure is about 0.4. For geocell reinforced retention systems RFs is about 0.3. The FHWA (2001) guidelines for reinforced steep slopes allow for RFs of 0.5. Hence, compared with this work, the FHWA recommendation for reinforced slopes is slightly conservative. The IITK (2005) recommendation for unreinforced slopes of one-third of the PGA is amazingly close to the measured results.

Tests 1 and 2 show that gravity walls made of geocell can perform well under seismic loading. Such gravity systems may be economical for walls up to 3-4m high. Tests 3 and 4 show that a reinforced system, made entirely of geocell and soil, can be effective and likely economical. The tests reported herein are relevant to short-term performance when considering the utilized HDPE geocell. Without improvement, the HDPE geocell used is not suitable for long-term applications as it tends to creep even under low loads.

Finally, the exhumed structures indicated that soil matrix suction was present thus enabling one to trace slip surfaces within the soil mass. While such suction creates significant apparent cohesion thus possibly rendering the reinforcement dormant, once the active wedge is formed the suction vanishes in its vicinity. Hence, assuming in the back-analysis, as well as in design, zero cohesions is reasonable and indeed prudent. Leshchinsky et al (2009) further discuss this issue.

4. IMPORTANCE OF STATICS IN INTERPRETING FIELD DATA

4.1 Background

Generally, field measurements of force in geosynthetic reinforcement are smaller than expected. "Smaller than expected" means smaller than the minimum required to maintain global equilibrium at a limit state. To an engineer this may appear as magic; an observed phenomenon that contradict the basis of mechanics: statics. In fact, it has resulted in a new formulation, called the K-stiffness method (Allen et al. 2003), which is a statistical analysis based on a compilation of field data of reinforcement loads in retaining walls from various, unrelated researchers. The formulation is *not* based in physics or statics, but in statistics, ignoring long-term design stability essential to the safe function of a structure. The approach empirically links stiffness, spacing of the reinforcement layers, facing properties, batter, and shear strength of the soil using little more than statistical correlations. However, the method ignores the vital inclusion of statics in design for the sake of rendering less conservative reinforcement tensile forces. Seasoned engineers would, and should, be skeptical about the feasibility of such statistical shortcuts. The following discussion implies that interpretation of field data should be done carefully. Mechanics should not be forfeited in lieu of magic (e.g., Leshchinsky 2009).

4.2 Impact of Apparent Cohesion

Refer to Figure 6. Shown is an excavator on top of an unreinforced steep sandy slope during the deconstruction of the Indian River Inlet Bridge (IRIB) approach embankments in Sussex County, Delaware (see also Leshchinsky et al. 2010b). This photo was taken near the location where strains in geogrid panels were measured. The height of the unreinforced sandy slope is about 6 m and its inclination is roughly 75°. The slope is comprised of medium sand with less than 5% passing sieve 200. Following mechanics of slope stability, this cohesionless slope cannot remain stable even without the heavy, constantly vibrating excavator on its top. This large-scale structure exhibits the same phenomenon as in sandcastles – very steep, apparently stable unreinforced slope.

The observed phenomenon in Figure 6 can be attributed solely to soil matrix suction due to moisture in the sand effectively produces apparent cohesion. This cohesion keeps sandcastles and even larger structures stable. In fact, this phenomenon has been studied using centrifugal modeling (Ling et al. 2009b). This study shows that increase in the sand's moisture content (for example, due to rainfall) diminishes the cohesion resulting in collapse of the sandy steep slope. One can imagine now that geosynthetic layers had been installed in the unreinforced slope in Figure 6. Considering that the unreinforced slope seems stable, the expected mobilized strains in the installed layers would be zero as it is not needed for stability. In reality, perhaps small values of strains may exist at random locations along reinforcement layers, likely induced by compaction and differential movements of backfill during construction. However, substantial strains, in the order of 3% to 5%, were measured in the geogrids embedded in the adjacent reinforced sand wall (Leshchinsky et al. 2010b). Unlike the slope over which the excavator operated for a few hours and where no precipitation occurred, the reinforced wall was subject to many rainfall events in its life. These events caused the moisture content in the sand to increase and the apparent cohesion to vanish. Hence, the dormant reinforcement was activated resulting in substantial mobilization of its strength. It is interesting to note that the geogrids 'remembered' the maximum induced strains; that is, after the rain event there was likely apparent cohesion thus making the reinforcement dormant again, however, the induced strains were locked in.

Figure 6. Excavator on top of unreinforced steep sand slope

The observation related to IRIB is commonly noticed in construction. It is presented not to 'warn' designers to ignore 'cohesion', as this should be an obvious practice in design of geogrid-reinforced walls. It is presented to warn researchers who monitor gages in walls to realize that 'smaller than expected' measured forces are not necessarily because the reinforcement is excessively strong but rather because an apparent cohesion renders a stable system where the reinforcement is dormant. Any significant increase in moisture may diminish the apparent cohesion thus making the small force observation inherently unreliable in the context of design. It is suggested that what appears as magic is actually due to apparent cohesion which is dependent on the moisture content of the backfill.

The reality observed in Figure 6 was attributed to an apparent cohesion of sand. Using an acceptable slope stability method, log spiral analysis, one can relate the apparent cohesion required to render a 'stable' slope, albeit without the surcharge induced by the excavator. Table 4 shows the minimum required cohesion considering different frictional strengths values for 90° and 75° slopes, all 6 m high having unit weight of 20 kN/m^3. The sand at IRIB was dense and likely had frictional strength of about 45°. Hence, for a 75° slope the required minimum apparent cohesion is 7 kPa. Such value of cohesion due to suction in sand is feasible but should be considered completely unreliable and ignored in design.

No wonder that some geotechnical engineers consider cohesion as the "invention of the devil"; i.e., a little cohesion can make even a sandy, steep slope stable. Its unreliability, however, can lead to a disaster if one depends on it. Fortunately, the alternative to apparent cohesion is geosynthetic reinforcement. It has an equivalent impact to cohesion; however, this manmade material is predictable, reliable, durable, and easy to integrate in existing geotechnical analysis. Unlike apparent cohesion, there is *no* magic with geosynthetics, just sound geotechnical engineering.

Table 4. Required cohesion to render stable slope

Slope: Inclination	Slope: Height [m]	Unit Weight, γ [kN/m^3]	Internal Angle of Friction, ϕ	Cohesion, c [kPa]
75°	6.0	20	30°	>12.1
75°	6.0	20	35°	>10.3
75°	6.0	20	40°	> 8.6
75°	6.0	20	45°	> 7.0
90°	6.0	20	30°	>18.0
90°	6.0	20	35°	>16.2
90°	6.0	20	40°	>14.5
90°	6.0	20	45°	>12.9

Apparent cohesion in sand may sound as an oxymoron. When using the term cohesionless soil, one will typically refer to sand as an example. Cohesion existence in 'cohesionless' soils is a result of soil matrix suction which is often associated with capillary suction. Soil matrix suction is a subset of soil physics and soil mechanics. Its effects on soil behavior (e.g., compaction, strength) can be very significant. In fact, behavior of unsaturated soils is an important and promising emerging research area. In general, due to its surface tension, water molecules in the interparticle voids bond the soil grains at their interface with the air that is present in the voids and where menisci develops – see Figure 7. The smaller the grain size the greater the bonding or apparent cohesion. For example, suction effects on uniformly graded gravel would be negligible while the effects on well graded gravel could be significant. Saturation or complete dryness causes loss of this bond. Increase in moisture content causes rapid loss of cohesion. Even a small amount of fines in sand can result in measurable cohesion. In the context of reinforced walls and slopes, the research on the behavior of unsaturated soils may lead to better interpretation of field data. However, one doubts if it will lead to a change in design methodologies as this apparent cohesion is an unreliable long-term parameter. It is extremely important, however, in interpreting field data.

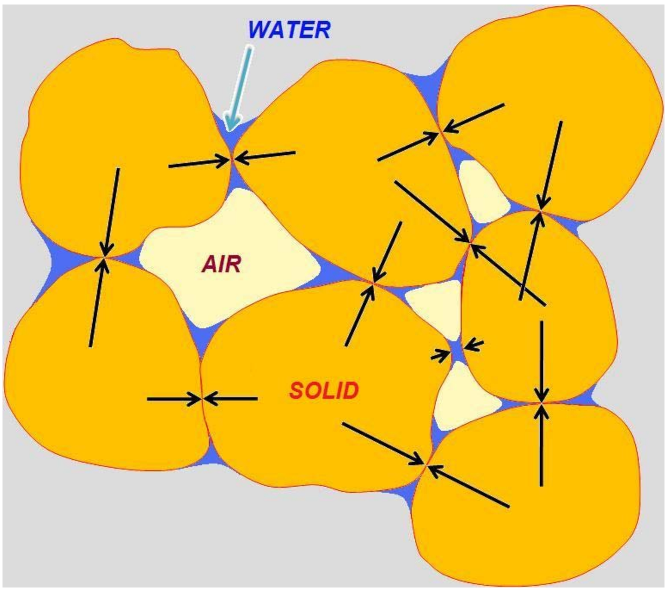

Figure 7. Soil Matrix: Solid particles and voids filled with water and air (interparticle forces generated by suction are illustrated by vectors)

4.3 Commentary

The indications that suggest current design is conservative do not transitively imply that the remedy offered by the K-stiffness method is correct. In fact, without a mechanistic benchmark, its use may lead to overly reduced conservatism, an unsafe conclusion that could result in failure.

Design should produce structures that are safe and economical for a set life span. Often field measurements indicate that the load in geosynthetic reinforcement used in constructed walls and slopes is significantly smaller than predicted in design. One well-known element in design that contributes to overestimation of load is a significant underestimate of the backfill's frictional strength. That is, *tan*(ϕ) used in design is typically as low as half when compared with the actual value. Such a discrepancy produces the impression that the mechanics used in design is overly conservative contributing to the mystery of low measured loads. Apparent cohesion, however, has much greater impact than friction. While apparent cohesion stabilizes in a similar process as geosynthetic, it is unreliable and should not be used in design. Presence of cohesion may lead to smaller loads measured in reinforcement. Such apparent cohesion can be formed by soil matrix suction. Ignoring suction in interpreting measured field data may lead to unsafe conclusions. It replaces mechanics with magic as it ignores cohesion but attributes its impact to the presence of geosynthetics. Unfortunately it is a daunting mission to consider suction in interpreting field measured data. Furthermore, suction will vary with moisture content; hence, it is not a reliable design parameter considering the life span of a structure. Underestimating frictional strength and disregard of existing apparent cohesion leads to a paradoxical 'conclusion' where magic is real and basic rules of mechanics are unreal.

Reports on measured force that is smaller than predicted are often mentioned to reflect 'at working' condition. This condition is explained by the absence of a slip surface in the backfill soil. Hence, it is claimed that design which considers a limit state in determining the strength (and length) of the geosynthetic is overly conservative as the premise of failure is not realized. This explanation also serves as a reason for uncritical acceptance of measured data in lieu of mechanics. However, existence of apparent cohesion and higher than assumed frictional strength can prevent the formation of continuous slip surface (e.g., Figure 6) thus providing an equally compelling and physically sound explanation for the 'at working' conditions. Such conditions underestimate the required strength of the geosynthetic should the apparent cohesion diminish or should the designer use the actual frictional strength of the backfill. Paradoxically, to prevent the formation of slip surfaces by stiff geosynthetic layers alone, it has to be stronger than the load that causes the slip surface to fully develop. That is, they have to be able to resist backfill movements, therefore preventing the soil from mobilizing its frictional strength. To ensure stability, the reinforcement has to compensate for the smaller contribution of resistance from the 'restrained' soil. Hence, the 'at working' condition does not explain the magic of low measured force; the unaccounted soil strength does. Proper use of soil strength leads to design that is sound and compatible with statics.

Finally, design of geotechnical structures nearly always considers the safety against collapse. Apparent cohesion is ignored in design, as it should be. Determining the required reinforcement strength based solely on measured field data while ignoring the apparent cohesion may result in a structure that is inherently unsafe. That is, *globally* there could be a substantial deficit in the sum of resistance of all layers of reinforcement relative to what is *statically* needed to stabilize the *cohesionless* reinforced structure. Static global equilibrium must be considered as a benchmark when assessing experimental data (Leshchinsky 2009). Indeed, the current reduction factor for creep could be excessive and thus may make up for a magic-based unconservative approach. However, counting on two wrongs to make one right promotes magic associated with the use of geosynthetics in reinforced soil.

Moreover, since engineering is not science fiction, magic in design is a step in the wrong direction. Soil reinforcing is a subarea of slope engineering for which well-established, sound designs already exist.

5. CONCLUDING REMARKS

While current design of geosynthetic reinforced soil is well-established, rendering safe and economical structures, there are still some important issues. Research and development can resolve these issues and improve the economics and avoid pitfalls. This paper presents three such issues. The first issue deals with the artificial definition of reinforced walls and reinforced slopes. The distinction is based on an arbitrary slope angle. Such division results in two incompatible design methodologies. The second issue deals with the current seismic design of geosynthetic reinforced walls. This design actually inhibits the use of such walls in seismic areas. However, field experience indicates that such walls actually behave very well under seismic loads. Results of large scale shake table tests demonstrate the performance of geosynthetic reinforced retention structures. An alternative pseudostatic design approach, including reduced seismic coefficients, is proposed as a conclusion. The third issue deals with observations of 'smaller than expected' field measured load in geosynthetic reinforcement. These measurements have resulted in 'calibration' of a new design methodology that completely ignores statics and entirely relies on statistics. A plausible explanation for the apparent conservatism is due to apparent cohesion which is generated by soil matrix suction. Without this cohesion, which is likely to disappear during the life span of the reinforced structure, the statistically-based approach yields a structure that is globally unsafe. Hence, in the context of design, the statistical approach without a benchmark based on statics, is unsafe. The references provided enable the interested reader to further explore the raised issues.

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