Earth Reinforcement, Ochiai, Yasufuku & Omine (eds) © 1997 Balkema, Rotterdam, ISBN 90 5410 833 9

Geosynthetic reinforced soil: Evolution of design methods in the USA

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ABSTRACT: An overview of past and present design methods for geosynthetic reinforced soil structures is discussed. The focus is not on details but rather on difference in assumptions and methodologies. It is concluded that current designs are not more complicated than those used in the past. They are by far less conservative with regard to reinforcement strength. Current designs also use partial safety factors in a rational way. Unlike past designs, current ones are based on a standardized testing techniques to characterize materials properties. Current designs outcome produce safe, and in many cases economical, reinforced soil structures. Simplicity of design methods promotes the application of soil reinforcing and ensures safe structures.

1 INTRODUCTION

It should be stated up-front that the scope of this paper is limited to a few, but typical, design methods currently used in the USA. The details of these methods can be found in the cited references. The writer has chosen to provide just an overview of these methods, including an historical perspective, so that the reader may focus on the current state of the state-of-practice. Hopefully, a judgement about the current state of design methods could be rendered independently by the reader. It should also be stated that the writer tries to be an active researcher; i.e., attempts to come up with solutions to problems and thus possesses a critical view of existing knowledge. However, in writing this paper, the critical view towards existing knowledge was somewhat suppressed. Instead, the perspective of the "real world," where economics plays a major role, was adopted (i.e., the quest for the absolute, but perhaps elusive and sometimes inconsequential, "truth" is intentionally being abandoned).

Reinforced soil structures constructed in the 70's caused excitement among many geotechnical practitioners. These structures were designed based on simple methods, directly extrapolating sound designs of conventional geotechnical structures. Happily, these methods produced conservative, yet economical, structures (e.g., the Glenwood Canyon walls as reported by Bell, Barrett and Ruckman, 1983). One can just wonder what would have happened if these design methods were unconservative (i.e., rendering an occasional failure) or complicated (i.e., difficult to comprehend by practicing engineers). That is, would we have today proliferating reinforced soil structures?

2 OVERVIEW OF PAST METHODS

2.1 Design of Walls

Formal design guidelines were issued by the US Forest (Steward et al., 1977), following recommendations by Bell et al. (1975). The basic approach was a direct adaptation from design guidelines developed for steel reinforced walls as presented by Lee et al. (1973, 1975). Analysis was divided into internal and external stability. The internal stability was used to determine the required tensile strength of the reinforcement, as well as its initial length. At-rest lateral earth pressures were assumed to exist and thus K_0 was used to calculate the reactive tensile force in the reinforcement needed to resist these pressures. Essentially, tieback analysis was carried out by assuming a wedge, inclined at $(45+\phi/2)$, to act as the active (i.e., unstable) soil mass. The soil behind this wedge was assumed to be stable. Therefore, the reinforcement was anchored (tied-back) into this stable soil. This anchorage ensured the capacity of the reinforcement to develop its required tensile force. The overall length of each layer was evaluated by combining the wedge width and the calculated anchorage length at each layer's elevation. The maximum combined length value, considering the lengths of all layers, was used to determine the required *uniform* length for the wall.

External stability was conducted similar to that of conventional gravity walls. The reinforced soil mass (its initial dimensions determined by internal stability analysis) had to resist direct sliding and overturning. It also had to have a sufficient margin of safety against bearing capacity failure.

The length of the final uniform layout was based on the maximum length obtained from internal and all external stability analyses. Typically, a minimum length value of L/H = 0.8 and a maximum spacing of S_v = 0.45 *m* were used in case the analyses indicated less stringent values.

2.2 Design of Slopes and Embankments

In the context of this paper, the term slopes refers to internally reinforced steep slopes. The term embankments refers to embankments over soft soil that are reinforced only along their interface with the foundation.

Limit equilibrium slope stability analyses were modified to include reinforcement (e.g., Christie and El-Hadi, 1977; Fowler, 1982; Christopher and Holtz, 1984). The modified analyses were, to start with, conservative in terms of unreinforced slope stability evaluation (e.g., Fellenius method, Two-Part Wedge method with a horizontal interwedge force resultant). These simplified analyses are easier to modify and apply; however, they do not fully satisfy the basic premise of the analysis, that is, equilibrium. Reinforcement force inclination was assumed to be either horizontal, tangential or some arbitrary value in between. Since limit equilibrium deals with global stability, the total reinforcement force, needed to ensure a prescribed margin of safety, was determined. It was assumed that this force is evenly distributed among all reinforcing layers.

Numerical complexity delayed the use of rigorous stability analyses. Lack of computer programs made the design more of an art. Consequently, this situation motivated the development of simplified design charts (e.g., Leshchinsky and Reinschmidt, 1985; Schmertmann et al. 1987).

2.3 Safety Factors and Tests

An integral part of any design is the prescribed factors of safety. Furthermore, material properties determined based on standard tests are also essential.

A bulk factor of safety applied to the geosynthetics was used. It multiplied the calculated required tensile resistance so that a geosynthetic possessing adequate tensile strength could be selected. Another factor of safety was applied in the calculations for the required anchorage length.

Standard laboratory tests were adopted from the garment industry (e.g., grab tensile strength, seam strength). These tests were far from reflecting the actual application of geosynthetic as a reinforcing material. However, they usually produced conservative results. Additional data needed for design (e.g., friction along soil-geosynthetic interface) were developed on an *ad-hoc* basis and in many cases, a conservative value was selected in lieu of conducting a test.

3 OVERVIEW OF CURRENT METHODS

In the past 15 years, extensive advanced numerical analysis, mainly finite element (FE), has been conducted on reinforced soil structures. Simultaneously, experimental research, using physical models and full scale structures, has also been conducted. Especially useful were the FE parametric investigations using a code that has been validated against experimental tests.

Numerous studies concentrated on walls, a few on slopes, and very few on embankments. These studies resulted in:

1. Refined design methods for walls;
2. Evolution of walls with new types of facings that satisfy both aesthetics and economics (e.g., segmental blocks); and
3. Little change in design-oriented analysis for slopes and embankments though generic software packages are available, as tools, for conducting rigorous and comprehensive limit equilibrium designs.

3.1 Design of Walls

Currently, two popular design manuals are used for reinforced walls in the USA. One manual is titled *DEMO 82: Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines* (Elias and Christopher, 1996). It was developed under the sponsorship of the Federal Highway Administration (FHWA). This manual is compatible with, but more detailed than, AASHTO guidelines to appear in 1997. The second manual is titled *National Concrete Masonry Association Segmental Retaining Wall Design Manual* (Simac et al. 1993). It was developed under the sponsorship of the National Concrete and Masonry Association (NCMA). Both manuals present comprehensive designs. The manuals address the selection of material properties, consider various aspects of construction, and assess every aspect of structural stability. Furthermore, design details related to connection strength of the geosynthetic to the facia, as well as earthquake loading and reinforcement spacing and length, are also given.

NCMA manual is limited to segmental walls whereas DEMO 82 addresses all types of reinforced walls. NCMA account for some effects of soil-facing block friction while DEMO 82 ignores these effects. Consequently, DEMO 82 uses Rankine's wedge whereas NCMA uses Coulomb's wedge. In both cases the lateral earth pressures used are much smaller than K_0 (either Rankine's or Coulomb's). For typical granular soil this implies the tensile force at-work conditions is nearly half the value used in the 70's. Both NCMA and DEMO 82 allow for a minimum reinforcement length (i.e., minimum L/H is restricted to either 0.6 or 0.7) that is shorter than the minimum values used in the 70's.

It is interesting to note that DEMO 82 is actually simpler, though more comprehensive, than the previous FHWA manual (Holtz and Christopher, 1984). For example, the previous manual required to determine the vertical stress at the facing using Meyerhof's vertical stress distribution at each reinforcement level and then calculate the horizontal stress using Rankine's K_a. DEMO 82 does not require the use of Meyerhof stress for calculating the tensile reaction but rather uses a simple overburden stress calculation. The end result is somewhat lower tensile force in the reinforcement. Furthermore, the previous FHWA manual required overturning stability evaluation; DEMO 82 requires only that eccentricity be within a certain range. Most importantly, the current FHWA manual allows the use of geosynthetic reinforcement in properly designed permanent structures (e.g., bridge abutments) and complex geometries (e.g., cascading walls).

3.2 Designs of Slopes and Embankments

Design of reinforced slopes is addressed in DEMO 82 (Elias and Christopher, 1996) and the Corps of Engineers (WES; Leshchinsky, 1995, 1996). Both designs use comprehensive stability analyses approach, examining a variety of potential failure mechanisms so as to ensure sufficient length and strength of reinforcement. Both are based on limit equilibrium analyses.

Limit equilibrium analysis can be used to assess the global stability of a reinforced slope. It does not deal with local equilibrium. If this equilibrium is being ignored, some layers might be overstressed, while others are under-stressed, potentially resulting in rupture or pullout of reinforcement layers. Such a situation may lead to a progressive failure. Both modified designs address this aspect. DEMO 82 assumes that the total required force to restore stability is uniformly distributed among all reinforcing layers if $H < 6.0\ m$. However, for $H > 6.0\ m$ an empirical distribution is used: layers embedded in bottom H/3 carry $T_{total}/2$, in middle H/3 carry $T_{total}/3$, and in top H/3 carry $T_{total}/6$. The layers within each H/3 are uniformly stressed. WES ensures analytically both local and global equilibria at a limit state stability. Local equilibrium is assessed in a similar fashion to walls; however, log spiral slip surfaces are used. This analytical approach leads to a probable range of tensile force in each layer (Leshchinsky et al., 1995). Consequently, the reinforcement strength could be selected.

There has been very little change in the design of reinforced embankments over the past 15 years. This problem is complicated; it possesses high uncertainty in defining the soft soil properties; the potential for deep-seated failures that may lead to extremely high-strength and expensive reinforcement; significant dependence of design on consolidation rate (difficult to predict) and on complex construction (which may result, for example, in mud waves). Indeed, this design is to a large extent an *art*. Many large-scale projects using geosynthetic reinforcement were successfully completed in recent years. The writer is aware of some projects where the economics of the reinforced embankment by far outweighed alternative engineering solutions. Since some of these projects are well instrumented and monitored, it is likely that the experience gained will lead to some refinements in design (e.g., what should be the inclination of reinforcement force in analysis? in embankment over soft soil using tangential instead of horizontal inclination may lead to half the required strength). It is also likely that economics will dictate the use of strip drains and staged construction.

3.3 Safety Factors and Tests

Partial factors of safety (or reduction factors) for geosynthetics are currently being used. Their value depends mainly on the application, environment of the installation site, construction, and polymer type. These reduction factors address creep, installation damage, and chemical and biological degradation. They are selected rationally so as to ensure that the required strength will be available at the end of the design life of the structure. To account for uncertainties associated with material parameters and with design assumptions, a global factor of safety is also used (typically its value is between 1.3 and 1.5). Unlike the bulk safety factor used in the 70's, the break down to partial factors allows the designer to conduct a rational design in which each "weakness" of the material is explicitly accounted for.

Whether designing a wall, a slope or an embankment, standard laboratory tests are available to determine the reinforcement properties in a realistic manner. These standards were developed by ASTM (American Society for Testing and Materials) and GRI (Geosynthetic Research Institute). They are constantly being revised. These standard tests ensure credible input for design and allow for uniform specifications of design output.

4 CONCLUSION AND DISCUSSION

The following main points can be stated when comparing past and present designs:

1. Past design of walls was based on at-rest working stress for calculating the reinforcement tensile reaction. Stability of walls was evaluated using various limit equilibrium analyses. Current design is based on active state working stress for calculating the reinforcement reactive force. There has been very little change in methods assessing the stability of walls.
2. Past design of slopes was based on limit equilibrium analysis, disregarding local equilibrium. Current designs are also based on limit equilibrium, however, local equilibrium (or stability) is considered either implicitly (DEMO 82) or explicitly (WES).
3. Past and present designs of embankments over soft soil are based on limit equilibrium and contain a considerable amount of engineering judgement.
4. In the past, a bulk factor of safety was applied on the required tensile resistance. Currently, partial factors of safety are being used to account for conditions such as the environment at the site, construction and the actual reinforcing product.
5. In the past, materials properties were loosely defined. Currently, there are standard laboratory tests to define necessary materials properties in design and specifications.
6. Past structures reinforced with geosynthetics were built in secondary and temporary applications. Confidence in current designs and material characterization allows for use of geosynthetics in important and permanent structures.

Engineers and owners desire safe and economical designs. Evolution over the past 15 years resulted in refinement and rationalization of design procedures. The refinement of designs was made possible by field tests, model tests, and advanced numerical analyses. The end result is indeed safe and economical structures.

The analytical portion of current designs is *not* more complicated than the one used in past designs. This agrees well with the philosophy that design requiring simple analysis is likely to be used. Furthermore, it is likely to be used successfully. Considering current specified backfill soil and type of structures, the writer think that further refinement of designs for walls and slopes may yield marginal results. In fact, it may result in adversely affecting constructability (if construction is ignored in design) or simply be impractical (i.e., require a "paper thin" geosynthetic that will fail during installation or a design rendering a layout where each layer possesses a different length). One has to bear in mind that the typical cost of geosynthetics is only 5 to 10% of the overall cost of a project. However, to alleviate the concerns some designers have with creep, it is desirable to address this issue in design by allowing stress relaxation to be coupled with creep. Under such coupled conditions, one may realize in a rational way that creep does not necessarily means intolerable deformations in the future.

The writer foresees a major application of geosynthetic reinforcement in conjunction with poor backfill soils. The economics of such an application seems to be very appealing. Designs for such backfills will have to be completely revamped.

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