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A new concept of seismic design of geosynthetic-reinforced soil structures: Permanent-displacement limit

Hoe I. Ling & Dov Leshchinsky
University of Delaware, Newark, Del., USA

Edward B. Perry
Waterways Experiment Station, US Army Corps of Engineers, Vicksburg, Miss., USA

ABSTRUCT: Seismic design of geosynthetic-reinforced soil structures is conducted using a pseudo-static analysis. Based on this analysis, design charts are developed for a vertical slope/wall and the effects of seismic inertia force are investigated. However, at large seismic accelerations, stability requires an unreasonably long geosynthetic. Consequently, procedures are proposed to determine the yield acceleration of geosynthetic-reinforced soil structures leading to a permanent displacement analysis. An example is included to illustrate usage of permanent-displacement limit in design.

1 INTRODUCTION

Geosynthetic-Reinforced soil structures are becoming popular as part of infrastructure systems used in, for example, highway and railway construction. Increasing number of these structures are indeed for critical and permanent applications (e.g., Tatsuoka and Leshchinsky, 1994). It lacks, however, a simple yet rationale procedure for seismic design. The performance of geosynthetic-reinforced soil structures under seismic loadings are reported to be satisfactory, but some suffered from minor cracking and sliding after the recent major earthquakes, such as 1989 Loma Prieta Earthquake (Collin et al., 1992), 1994 Northridge Earthquake (White and Holtz, 1995), and 1995 Kobe Earthquake (Tatsuoka et al., 1995; Matsui et al., 1996).

Severe market and design competition have enabled a more economic design of reinforced soil structures. It is possible that traditional design procedures (e.g., Christopher et al., 1990) which do not consider *explicitly* seismic effects may not be conservative in the events of earthquake with large intensity.

This paper is presented in two parts: a pseudo-static design procedure followed by a permanent-displacement design procedure. The effects of earthquake acceleration on design are investigated for geosynthetic-reinforced retaining walls. Finally, a design example is illustrated.

2 PSEUDO-STATIC ANALYSIS

Seismic design of conventional gravity retaining walls is typically done based on the Mononobe-Okabe approach (Mononobe, 1926; Okabe, 1926), which is an extension of Coulomb analysis. The earthquake inertia force is considered to act permanently as a percentage of the deadweight of the assumed failure soil mass. The calculated seismic active earth pressure allows for a proper dimensioning of the wall to ensure stability against direct sliding and overturning.

Extension of Mononobe-Okabe analysis to geosynthetic-reinforced soil retaining walls has been reported by Richardson and Lee (1975) and Bathurst and Cai (1996). This approach assumed a planar failure surface in the backfill. Analysis based on a two-part wedge approach has also been proposed by Bonarparte et al. (1986), Koga and Washida (1992), Murata et al. (1994), Yamanouchi and Fukuda (1993), among others. These procedures are regarded as "analysis" procedure (instead of "design") in the sense that factors of safety against different failure modes are evaluated based on an assumed wall configuration and geosynthetic force.

In the proposed procedure, seismic force is considered as pseudo-static through a seismic coefficient C_s. It is an extension of limit equilibrium analysis proposed earlier by Leshchinsky and Boedeker (1989), Leshchinsky (1993, 1995), and Leshchinsky et al. (1996). This procedure is considered as "design" which determines the required geosynthetic force and lengths to satisfy a prescribed factor of safety. This approach is valid for slopes of different inclinations although only that of a wall (i.e., 90-degree slope) is presented herein.

Different modes of failure are considered: rotational failure within and beyond the reinforced soil zone and direct sliding. The required geosynthetic force is determined through tieback analysis while the required geosynthetic lengths are determined through tieback/compound analysis and direct sliding analysis.

In the rotational sliding analysis, a log-spiral failure surface is considered. This mechanism degenerates to extended Coulomb analysis in a vertical slope. Note also that the inclination of reinforcement force is considered as horizontal in the proposed procedure. Direct sliding is considered using a two-part wedge mechanism. Details of proposed pseudo-static design procedure are given in Leshchinsky et al. (1996) and Ling, Leshchinsky and Perry (1995).

There are no proper guidelines available for the selection of the seismic coefficient, C_s, used in design of reinforced soil structures. However, this may be selected in a manner similar to design of conventional retaining walls based on the seismic risk map. The coefficient is recommended to be as large as 0.3 in seismically active areas, such as Japan and California.

The required strength and lengths of geosynthetic for a design are conveniently expressed using normalized coefficients:

$$K = \frac{\Sigma t_j}{\frac{1}{2}\gamma H^2} \approx \frac{t_j}{\gamma h_j D_j} \tag{1}$$

$$L_c = \frac{l_c}{H} \tag{2}$$

$$L_{ds} = \frac{l_{ds}}{H} \tag{3}$$

where γ and H are the unit weight of soil and the wall height, respectively; h_j the depth of the j-th geosynthetic layer measured from the wall crest, t_j and D_j the required geosynthetic tieback strength and tributary area of layer j; l_c and l_{ds} the required length to resist tieback/compound failure and direct sliding. It should be pointed out that t_j is the required strength of layer j to assure local stability. It is analogous to conventional analysis of reinforced walls where K_a, overburden pressure and tributary area are used to calculate the required reinforcement strength.

In a design, it is practical to select the required length at the top layer based on L_c and at the bottom based on the greater length of L_c and L_{ds}. Length of other layers is obtained by interpolation.

To ensure global stability, where failure extends from the wall face through the reinforced soil and into the retained soil, geosynthetic having allowable strength greater than or equal to that calculated from tieback analysis is specified for each layer. Typically, at layer j, the specified geosynthetic has an allowable strength, $t_{j\text{-allowable}}$, larger than the required strength, t_j. It is, thus, practically required that only the bottom *m* layers be designed against compound failure so that the following relationship is satisfied:

$$\sum_{j=1}^{m} t_{j-allowable} \geq \sum_{j=1}^{n} t_j \tag{4}$$

The required anchorage length of each layer, $l_{e,j}$, is determined as

$$l_{e,j} = \frac{t_j}{2C_i \cdot \sigma_{v,j} \tan\phi} \tag{5}$$

where ϕ, C_i, $\sigma_{v,j}$ are the internal friction angle, soil-geosynthetic interaction coefficient, and average overburden pressure acting on the geosynthetic layer, respectively.

Figures 1 to 3 show the required geosynthetic strength and lengths for a vertical slope with ϕ

ranging from 20 to 45 degrees under static and seismic loadings. The analysis was conducted using ReSlope program (Leshchinsky, 1995) on a 5 m high wall and 20 layers of geosynthetics. The normalized results are valid for design of walls of any height and soil properties. Note that the direct sliding coefficient used is C_{ds}= 0.8.

An increase in the lengths and strength of geosynthetic is required following a decrease in ϕ, or when comparing seismic and static designs. At a typical value of ϕ, say 30°, two times the static tieback length and strength may be needed when comparing C_s = 0.3 and static designs. The difference is several times larger considering direct sliding stability.

3 PERMANENT DISPLACEMENT ANALYSIS

Direct sliding stability can be a concern when soil with a small friction angle is used in a reinforced soil retaining wall or when the seismic intensity is large. Figure 3 shows that for walls with ϕ slightly less than 30°, the reinforcement length becomes impractically long. Therefore, a permanent displacement limit could be instead used in design so that the resulting direct sliding from an earthquake would be tolerable, similar in concept to that of gravity retaining walls (Richards and Elms, 1979).

The presented permanent displacement analysis is an extension of rigid sliding block theory (Newmark, 1965; Whitman, 1953). In the Newmark's sliding block, the yield acceleration is given as C_{sy}= $\tan\phi_b$, where ϕ_b is the friction angle between the block and the planar surface it rests on. The yield acceleration of the reinforced soil block in a two-part wedge mechanism (Figure 4) is obtained when the factor of safety against direct sliding, including the seismic inertia force, equals to unity (Ling, Leshchinsky and Perry, 1995). That is,

$$C_{sy} = \frac{\tan(\phi-\theta)\{W_A - C_{ds}(W_A+W_B)\tan\delta\tan\phi\} + C_{ds}W_B\tan\phi}{W_A+W_B-\tan\delta\{W_B\tan(\phi-\theta)+C_{ds}W_A\tan\phi\}} \tag{6}$$

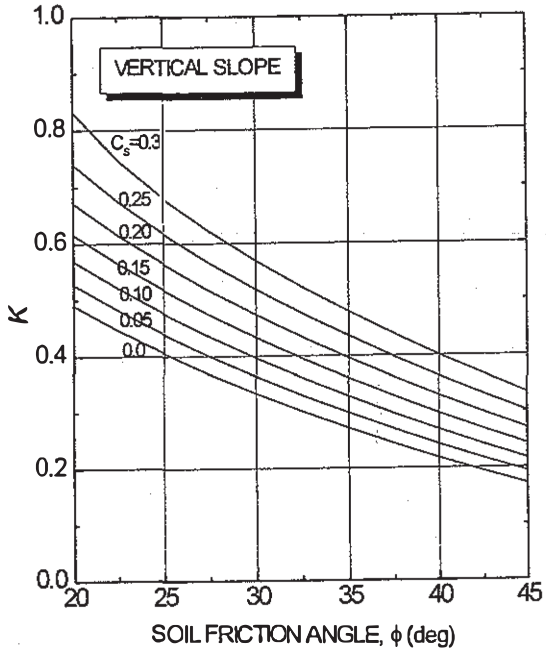

Figure 1. Total geosynthetic normalized strength required to resist tieback failure

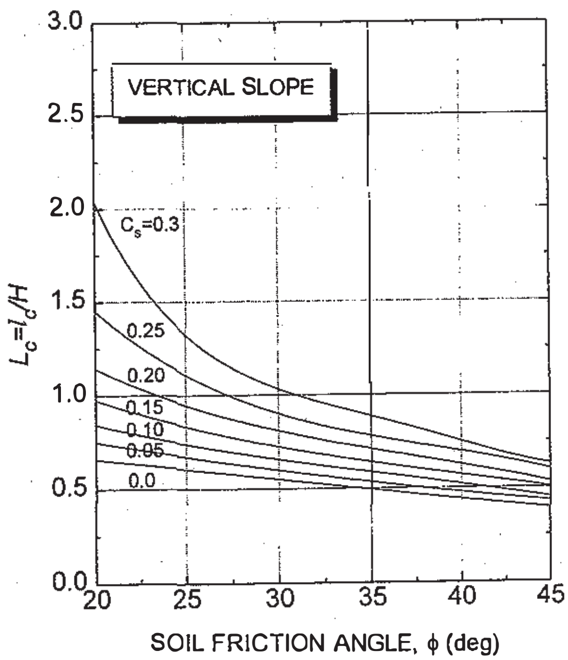

Figure 2. Geosynthetic length required to resist tieback/compound failure

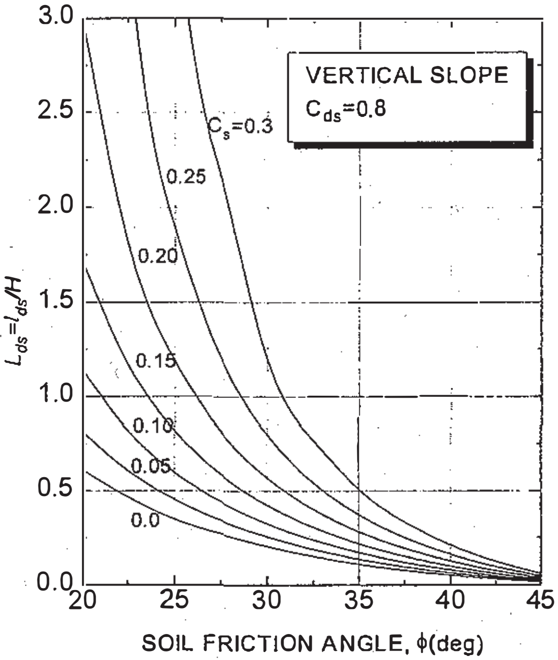

Figure 3. Geosynthetic length required to resist direct sliding

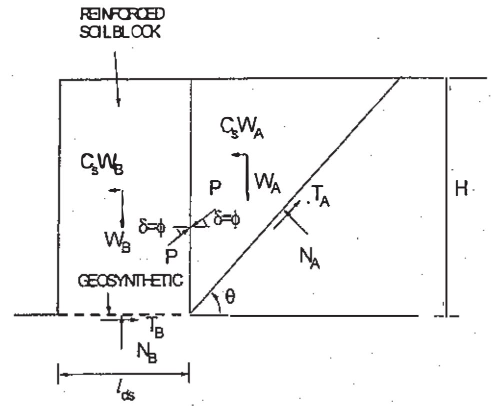

Figure 4. Direct sliding stability analysis

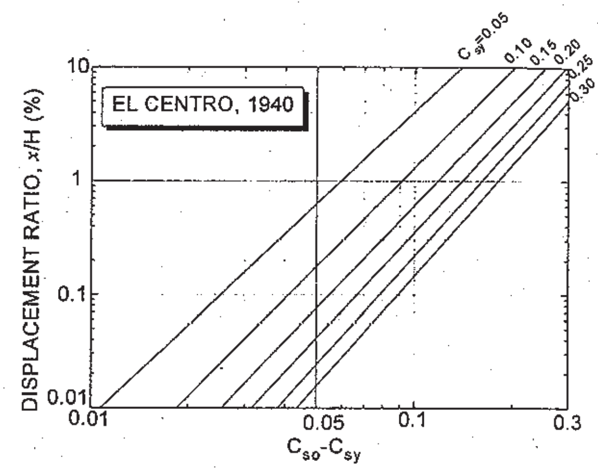

Figure 5. Permanent displacement ratio - El Centro Earthquake records

where θ is the critical angle of inclination of the retained soil wedge which varies with C_s, δ is the interwedge friction angle, taken as φ in this study.

The reinforced soil block starts moving as C_s exceeds C_{sy}. The acceleration of reinforced soil block, $\ddot{x}$, is obtained by establishing the equation of motion:

$$\ddot{x} = (C_s - C_{sy}) \cdot g \tag{7}$$

The permanent displacement is calculated by double-integrating Eq. (7). For a random earthquake, this has to be done numerically following the scheme presented in Ling and Leshchinsky (1995). Figure 5 shows the relationships between the permanent displacement for several peak values of acceleration and yield accelerations using the scaled El Centro Earthquake records.

4 DESIGN EXAMPLE

A wall 5 m high is constructed with soil of design properties φ= 25°, γ= 18 kN/m^3, subjected to an earthquake with peak acceleration 0.3 g, i.e, C_{so}= 0.3. By reducing C_s from 0.3 to 0.2 (i.e, C_{so}-C_{sy}= 0.1 in Figure 5), a permanent displacement of about 1.75 cm is expected, which is considered acceptable for most applications. From Figure 3, the required geosynthetic length is L_{ds}= 1.2 or l_{ds}= 6.0 m (φ= 25°, C_s= 0.2). Comparing Figures 2 and 3, the length of the bottom geosynthetic layer is actually governed by tieback/compound length instead of that of direct sliding.

From Figures 1 and 2, the required geosynthetic total strength and tieback/compound length are determined as Σt_j= 153 kN/m (K= 0.68) l_c= 6.6 m (L_c= 1.32), respectively. Table 1 summaries the required length and strength of geosynthetic at each elevation, using

25 layers of geosynthetic spaced at 20 cm. The required allowable geosynthetic strength is given as 20 kN/m and pullout interaction coefficient, C_i, is 0.8.

Note that the bottom 8 layers are designed with full allowable strength to resist compound failure (see Equation 4), extending from the reinforced soil into the retained soil. Therefore, the anchorage length is slightly longer. A uniform layout can be specified for this wall with the largest anchorage length added to the required length, i.e., 6.6 + 0.5 = 7.1 m.

Table 1. Design example: geosynthetic strength and lengths

j	z (m)	σ_{vj} (kN/m^2)	t_j (kN/m)	$t_{j\text{-}allow}$ (kN/m)	l_{ej} (m)
1	0.0	90.0	15.30	20.0	0.3
2	0.25	85.5	14.53	20.0	0.3
3	0.50	81.0	13.77	20.0	0.3
4	0.75	76.5	13.01	20.0	0.4
5	1.0	72.0	12.24	20.0	0.4
6	1.25	67.5	11.48	20.0	0.4
7	1.50	63.0	10.71	20.0	0.4
8	1.75	58.5	9.94	20.0	0.5
9	2.0	54.0	9.18	9.18	0.2
10	2.25	49.5	8.41	8.41	0.2
11	2.5	45.0	7.65	7.65	0.2
12	2.75	40.5	6.89	6.89	0.2
13	3.0	36.0	6.12	6.12	0.2
14	3.25	31.5	5.36	5.36	0.2
15	3.5	27.0	4.59	4.59	0.2
16	3.75	22.5	3.83	3.83	0.2
17	4.0	18.0	3.06	3.06	0.2
18	4.25	13.5	2.30	2.30	0.2
19	4.5	9.0	1.53	1.53	0.2
20	4.75	4.5	0.76	0.76	0.2

j : geosynthetic layer number
z : depth of layer
σ_{vj} : overburden pressure of j-th layer
t_j : required geosynthetic strength at j-th layer (tieback, local stability)
$t_{j\text{-allow}}$: allowable geosynthetic strength at j-th layer (compound, external stability)
l_{ej} : anchorage length of j-th layer

5 CONCLUSIONS

A well-established design procedure that is valid for slope of different inclinations was extended to include seismic effects based on a pseudo-static approach. Effects of seismic inertia force is significant in design of reinforced soil structures, particularly in direct sliding stability. A procedure to evaluate the permanent displacement resulting from direct sliding instability was included. Usage of this procedure was illustrated by a design example.

Discussions on permanent displacement analysis, including effects of different slope inclinations, vertical seismic inertia force and verification against a case history of Kobe Earthquake, are detailed by Ling, Leshchinsky and Perry (1995) and Ling and Leshchinsky (1996).

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