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SEISMIC DESIGN OF SLOPES REINFORCED WITH GEOGRIDS AND GEOTEXTILES

CONCEPTION SISMIQUE DES REMBLAIS RENFORCES AVEC DES GEOGRIDS ET DES GEOTEXTILES

ENTWURF VON DURCH GEOGRIDS UND GEOTEXTILIEN BEWEHRTEN BÖSCHUNGEN HINSICHTLICH SEISMISCHER BELASTUNGEN

The behavior of slopes and embankments reinforced with horizontal layers of polymer geogrid or geotextile and subjected to earthquake loading stresses is investigated. A simple psuedo-static, rigid-body analytical model is used to compare the amount, length and distribution of reinforcement required to maintain slope equilibrium during various seismic events. The results of the investigation are presented in a series of charts that compare the required reinforcing force and reinforcement length for seismic and static gravity loading conditions. The results show that for many practical applications, the number of layers of polymer reinforcement required for the static loading condition provides sufficient reinforcing force to maintain equilibrium during the seismic event. However, the length of reinforcement will often need to be increased somewhat to maintain equilibrium during seismic loading. Justification for these conclusions is presented and exceptions are highlighted.

Das Verhalten von Boeschungen und Daemmen, verstaerkt mit horizontalen Lagen aus Geogrid oder Geotextilien unter Erdbeben-Belastungen werden untersucht. Einfache pseudo-statische, und festkoerper untersuchende Modele werden gebraucht um die Menge, Laenge und Belastung der Verstaerkung zu bestimmen, die gebraucht wird um Gleichgewicht waehrend verschiedenen seismischen Belastungen zu gewaehren. Die Ergebnise werden in einer Serie von Tabellen vorgestellt, die die seismischen Belastungszustaende, mit den statischen lang zeitbelastungen vergleicht. Die Ergebnise zeigen das fuer viele praktische, Zustaende, die Menge der Verstaerkung die fuer die statischen Belastungszustaende ausreicht, auch gleichzeitig fuer die seismischen Belastungszustaende ausreicht. Die Laenge der Verstaerkung sollte fuer seismische Belastung ein bisschen vergroessert werden. Gruende fuer diese Schlussfolgerung und deren Ausnahmen werden vorgestellt.

INTRODUCTION

Polymer reinforcement materials such as geogrids and geotextiles are increasingly utilized in the construction of slopes and embankments with side-slopes steeper than the angle of internal friction (angle of repose) of the slope fill. These polymer materials, which are strong in tension, are placed in horizontal layers throughout the slope height during construction. The number, length, and spacing of the layers of reinforcement are selected to maintain stability of the slope with an adequate factor of safety. Design of slopes with polymer reinforcing elements has been discussed previously (1, 4, 9). Little attention has been given, however, to the design of reinforced slopes subjected to seismic loading.

This paper summarizes an investigation of the seismic stability of slopes reinforced with polymeric materials such as geogrids and geotextiles. The paper is limited to dry to moist slopes constructed of purely frictional soils resting on firm foundations. A simple, rigid body analytical model is used to compare the total reinforcement force and length required to maintain equilibrium in slopes subjected to a range of pseudo-static seismic coefficients. Conclusions are then drawn regarding the increase in the number of layers and length of reinforcement required to resist the induced horizontal inertial forces. Practical recommendations for design and construction are also provided.

FACTORS INFLUENCING SEISMIC SLOPE STABILITY

Factors that should be considered in comparing seismic and static slope stability include: (1) ground motions at the site; (2) slope geometry; (3) strength of the slope fill; (4) strength of the reinforcement; and (5) strength of the soil/reinforcement interface. These factors are discussed below.

Ground Motions - Ground motion parameters relevant to any site are a function of many factors including the earthquake Richter magnitude, the distance from the causative fault, the geologic conditions along the path between the causative fault and the site, and the foundation conditions at the site. For psuedo-static analyses, the ground motion parameter of greatest interest is peak horizontal ground acceleration, a, defined as the ratio of the horizontal ground acceleration to the acceleration of the earth's gravitational field. For the types of applications considered herein, the peak horizontal ground acceleration will be either: (1) prescribed by relevant building codes, local practice or other regulations; or (2) determined from attenuation relationships such as those presented by Idriss (3), Fig. 1.

Slope Geometry - Soil slopes are deformable bodies and the peak horizontal accelerations within the slope vary from a maximum (which is usually larger than the corresponding peak ground acceleration) at the slope crest to a minimum at the slope base. For any potential failure surface within a slope, the variation in horizontal acceleration along the surface can be averaged to obtain a single acceleration value for design (6, 11). For small to medium slopes (5 to 30 m) constructed with well compacted frictional soils resting on firm foundations, this average peak slope acceleration will be about equal to, or somewhat less than, the peak horizontal ground acceleration. Therefore, it has been assumed that the reinforced slopes considered herein respond to earthquakes as rigid bodies.

Correlations between a psuedo-static seismic coefficient for design, k, and the peak horizontal ground acceleration, a, vary considerably. Recommendations made by Seed and Whitman (12) for gravity retaining walls suggests that it is reasonable to assume that $k = 0.85a$ for reinforced slopes.

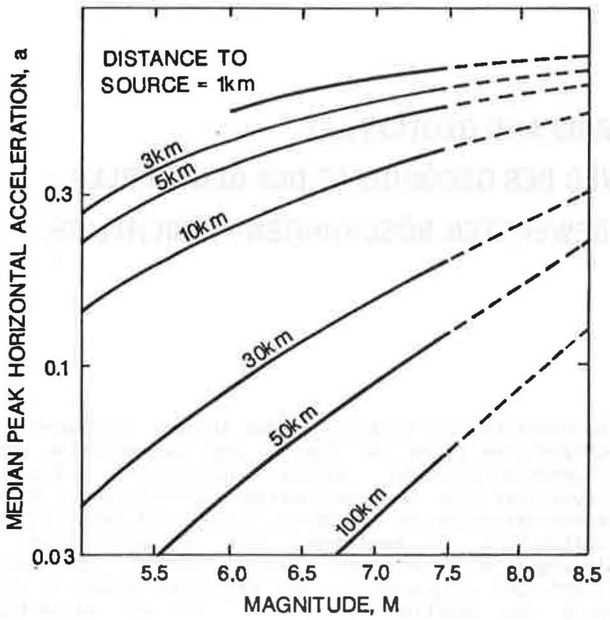


Fig. 1 Variation of median peak horizontal ground acceleration with earthquake magnitude and distance to the causative fault for rock and stiff soil sites (from 3).

Soil Strength - The embankments and slopes considered herein are assumed to be comprised of well-compacted dry to moist frictional materials which are relatively free-draining and not structurally sensitive. Therefore, excess pore-pressure development and cyclic mobility due to seismic loading has not been considered. For these soils, there is very little effect of earthquake shaking on the soil's large-strain, constant volume effective stress friction angle, ϕ_{cv}. The use of ϕ_{cv} rather than ϕ_{peak} is consistent with the use of polymer reinforcement since the strains required to develop the full tensile strengths of these materials will usually be greater than the soil's strain at peak strength. Jewell et al (4) recommended the use of ϕ_{cv} for the static design of cohesionless slopes reinforced with polymer geogrids.

Reinforcement Strength - The three main polymer types used to manufacture soil reinforcement materials are high density polyethylene (HDPE), polypropylene (PP) and polyester (PET). All three exhibit visco-elastic behavior and rate-dependent load-strain response. This rate-dependence means that for in-service conditions, the allowable reinforcement tensile force, α_ε (kN/m), for a limiting strain, ε, is different than that measured using standard laboratory tensile tests such as the Proposed Standard Test Method for Tensile Properties of Geotextiles by the Wide Strip Tensile Method (ASTM Designation 61-201). These standard test methods, or the results from these tests, will therefore require modification for design purposes to account for the effects of time and temperature, as well as for other factors such as site damage and soil/reinforcement interaction (1).

Design of reinforced slopes for long-term gravity loads should incorporate values of α_ε that are less than those values measured in short-duration standard tests. The percentage reduction in α_ε from the standard value will depend on a number of factors. In-isolation constant-loading creep tests by McGown et al (7) on different polymer reinforcement materials show that this reduction may be 50%, or more.

Seismic loading conditions will induce reinforcement strain rates higher than those used in standard laboratory strength tests. Thus, under seismic loading conditions α_ε may be higher than the value obtained from standard laboratory tests. However, as the loading strain rate increases, the reinforcement strain to rupture decreases and the polymer exhibits increasingly brittle behavior. If this strain to rupture decrease is large enough, and if slope deformations exceed some limiting value, brittle rupture of the reinforcement could occur during the seismic event. Therefore, α_ε for seismic design should consider both the increase in reinforcement strength due to rapid loading, as well as the decrease in ductility associated with high strain rates.

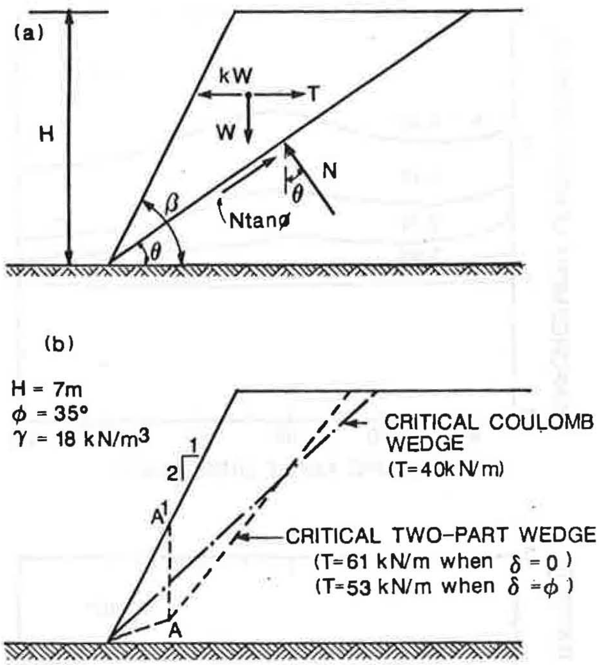
Test results presented by McGown et al (8) indicate that for both a PP and HDPE geogrid product, the tensile reinforcement force at peak load, α_p, at strain rates on the order of 1% per second are about 20% - 40% higher than the values of α_p at a standard test rate of 2% per minute. At a 1% per second strain rate, however, and a ground temperature of 10°C, the reinforcement ruptures prior to reaching a defineable yield point. Based on McGown's data, if the reinforcement tensile force is limited to approximately 90% of α_p obtained under standard test conditions, the possibility of brittle rupture of the reinforcement is precluded. This 90% limit would therefore seem appropriate for these products for seismic design. In general, a design parameter, R_α, can be evaluated for polymer materials as the ratio of α_ε for the seismic loading rates and limiting strains, to α_ε for the static long-term loading condition.

Strength of Soil/Reinforcement Interface - Only very limited information is available on the influence of cyclic loading and deformation rates on the behavior of the soil/reinforcement interface. For the soil types considered herein it appears reasonable to assume that the friction at the soil/reinforcement interface can be measured using suitably dimensioned, one-directional direct shear tests. Because slope deformations due to earthquake loading may be locally large, however, the large-strain interface friction rather than peak interface friction would appear to be relevant to design. The large-strain interface frictional strength is characterized by a coefficient of interaction defined as the ratio of the interface strength to the soil strength. The coefficient of interaction influences the reinforcement length required to prevent base sliding and the anchorage length required to prevent pullout.

ANALYSIS OF SEISMIC SLOPE STABILITY

The three main questions to be answered with respect to the stability of reinforced slopes subjected to horizontal accelerations are: (1) how much does the total required reinforcing force need to be increased, compared to the static case, to maintain equilibrium; (2) how much does the reinforcement length need to be increased, compared to the static case, to prevent reinforcement pullout or sliding of the reinforced soil mass; and (3) how should the vertical reinforcement distribution within the slope change for the seismic case compared to the static case? Simple analyses were used to gain insight into these questions.

Total Reinforcement Force - The increase in the total required reinforcing force was determined through a limit equilibrium analysis assuming a Coulomb failure wedge, Fig. 2a. The use of this simple wedge analysis usually results in an underestimation of the total required reinforcement tensile force, T, compared to the force obtained from analyses using more complex failure mechanisms, such as a circle, logarithmic spiral, or multi-part wedge, as shown in Fig. 2b. However, for



(c)

METHOD OF ANALYSIS	T_s (a=o) (kN/m)	T_e(a=0.2) (kN/m)	$R_T = T_e/T_s$
COULOMB WEDGE	40	86	2.15
TWO-PART WEDGE	53	105	1.98

Fig. 2 Seismic analysis of a reinforced slope using Coulomb wedge: (a) free body diagram; (b) comparison with two-part wedge failure mechanism; and (c) influence of method of anlaysis on ratios of required tensile forces. (Note: δ is the angle the interslice force makes with the normal to AA' for the two-part wedge analyses.)

direct comparison of the seismic and static tensile force requirements, the error introduced through use of the Coulomb wedge analysis has been found to be small, Fig. 2c.

The results of the calculations to determine the increase in horizontal tensile force needed to maintain the critical wedge in a state of limiting equilibrium are shown in Fig. 4. These results are for soils with angles of internal friction equal to 25°, 30° and 35°. The results are presented as a reinforcement force ratio, R_T:

$$R_T = T_e/T_s \tag{1}$$

where: T_e = the total reinforcement tensile force per unit width (kN/m) to maintain equilibrium for the seismic loading case; and T_s = the total reinforcement tensile force per unit width (kN/m) to maintain equilibrium for the static loading case. For static conditions, R_T equals one. For seismic conditions R_T is greater than one. Results are presented for four different values of psuedo-static seismic coefficient (k = 0.05, 0.10, 0.15 and 0.20). It can be seen that the value of R_T is proportional to the magnitude of this coefficient. The large values of R_T at low values of β are due to the fact that T_s approaches zero as β approaches ϕ.

Length of Reinforcement - The increase in the required length of reinforcement due to the horizontal inertia force was evaluated based on two criteria: (1) reinforcement pullout behind the critical wedge; and (2) sliding of the reinforced soil mass over a layer of reinforcement at the elevation of the toe of the slope, Fig. 3.

The large-strain value for the coefficient of interaction was taken as 0.8. The reinforcement length, L, was evaluated using both criteria in Fig. 3 and the longer of the two was taken as the length required to maintain stability. With both criteria it was assumed that all reinforcement layers had the same length, giving the reinforced soil mass a parallelogram shape.

The results of the calculations to determine the increase in reinforcement length needed to maintain stability are shown in Fig. 5. Results are presented for the same range of seismic accelerations and soil strengths as for the tensile force calculations. Presentation of the calculated results is in the form of a reinforcement length ratio, R_L:

$$R_L = L_e/L_s \tag{2}$$

where: L_e = the reinforcement length (m) to maintain equilibrium for the seismic loading case; and L_s = the reinforcement length (m) to maintain equilibrium for the static loading case. Detailed comparisons have indicated that neither R_L or L_1 (Fig. 3a) is significantly affected by the choice of either the Coulomb wedge or the two-part wedge method shown in Fig. 2b. For static conditions, R_L equals one. For seismic conditions R_L is greater than one. It can be seen that the value of R_L is proportional to the value of k.

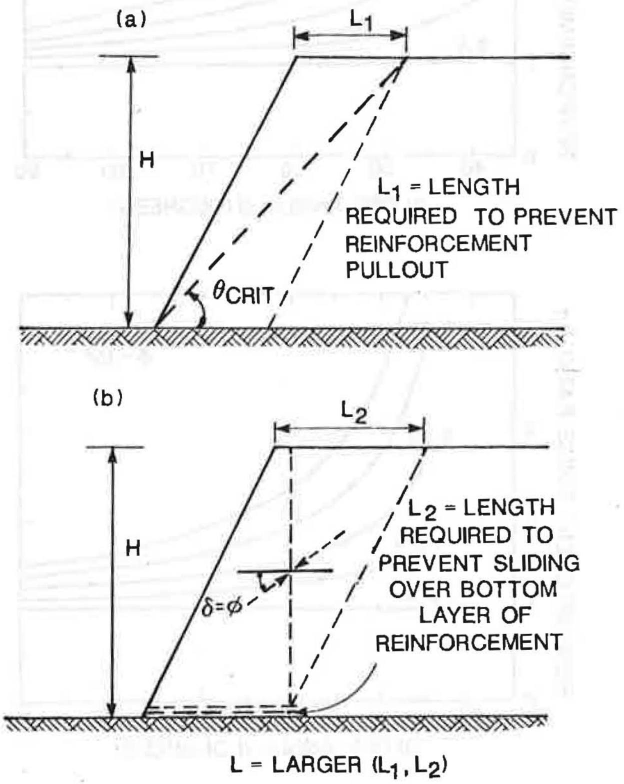


Fig. 3 Method for determining the required length of reinforcement: (a) length required to prevent reinforcement pullout; and (b) length required to prevent sliding of the reinforced soil mass.

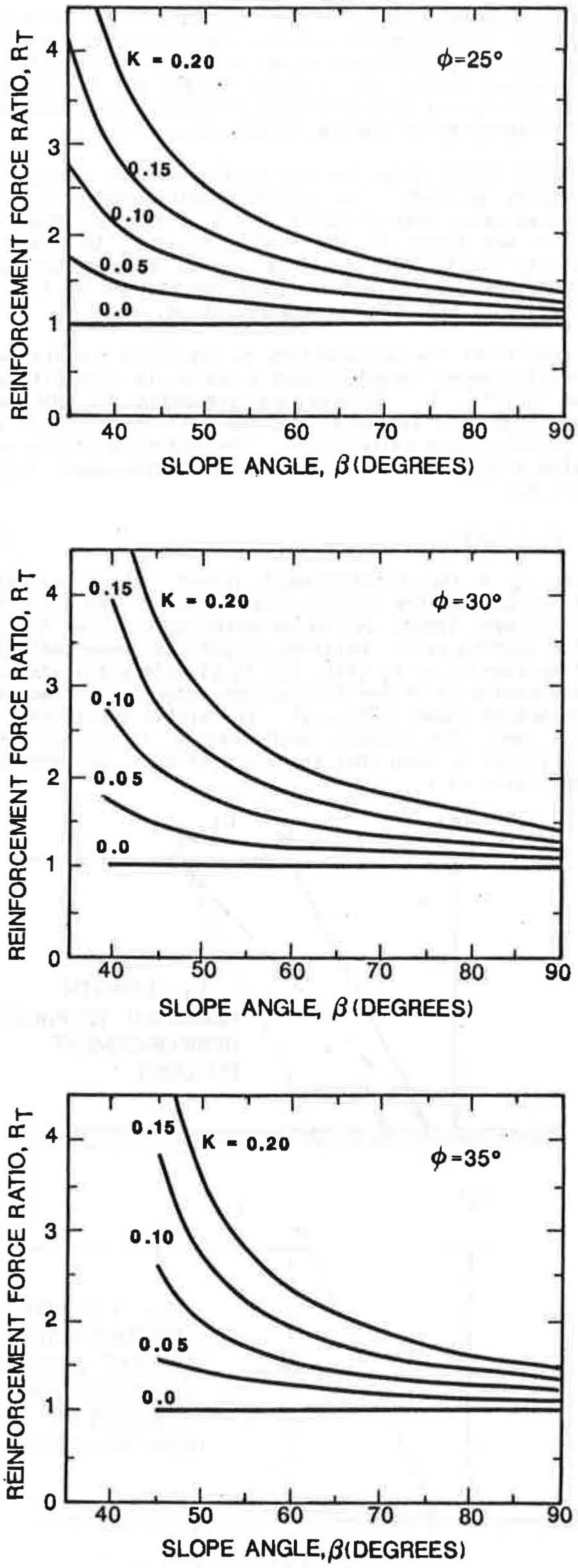


Fig. 4 Reinforcement force ratios, R_T, versus slope angle, β, soil strength, ϕ, and pseudo-static seismic coefficient, k.

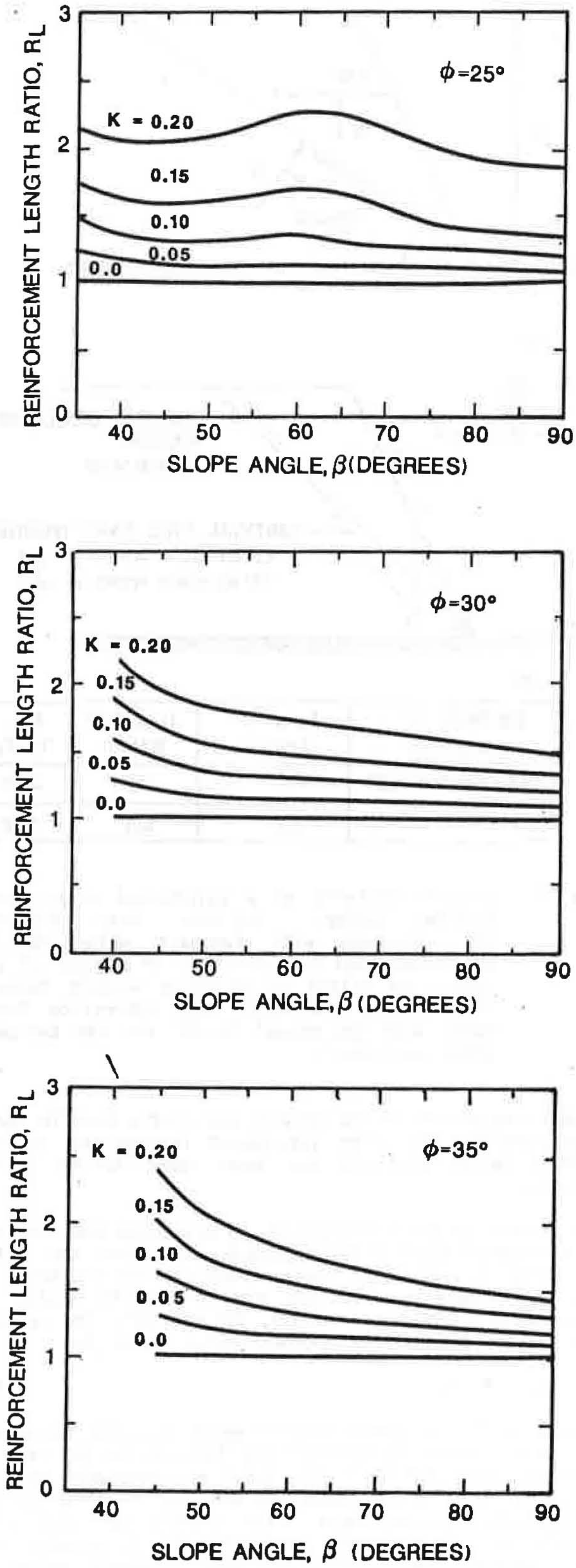


Fig. 5 Reinforcement length ratios, R_L, versus slope angle, β, soil strength, ϕ, and pseudo-static seismic coefficient k.

Distribution of Reinforcement - Reinforcement is placed in a slope at vertical intervals to ensure equilibrium at every level within the slope. For the static case, the reinforcement spacing should ideally decrease in proportion to the depth below the slope crest, and the resultant of the available reinforcement tensile force should be about 0.33H above the toe of the slope. Jewell et al (4) provide a practical method of determining reinforcement spacing based on this criterion.

Seed and Whitman (12) summarized results from a number of sources with respect to the dynamic increment in lateral earth pressure on concrete gravity walls due to seismic excitation. These results indicate that the dynamic lateral pressure increment acts at a height varying from about 0.5H to 0.67H. Richardson's (10) investigations of Reinforced Earth walls indicate that the distribution of dynamic lateral earth pressures is dependent on the stiffness of the soil/reinforcement system. The greater the wall stiffness, the higher the point of application of the dynamic resultant force, with a maximum of 0.67H. Given these results, and the fact that polymer reinforcement provides a system with lower stiffness than metallic reinforcement, it is reasonable to assume that the dynamic increment in required tensile force (T_e-T_s) acts at the midpoint of the slope. This force should be distributed uniformly through the height of the slope.

PRACTICAL CONSIDERATIONS FOR DESIGN

Factors of Safety - Selection of 85% of the peak horizontal ground acceleration as the psuedo-static seismic coefficient is conservative. The primary reason for this is that the horizontal inertial force associated with the peak acceleration is applied for only a very short period of time. In a slope with a calculated seismic factor of safety of one, yielding and deformation of the slope can occur. Catastrophic slope failure should not occur if the slope soils do not exhibit significant strength loss during the seismic event, which is the case for the dry to moist frictional soils considered herein. Slope deformations will be limited by the brevity with which the horizontal inertial forces act on the slope. For slopes not supporting highly strain sensitive structures, these deformations should be within acceptable limits.

The above factors, coupled with the knowledge that large seismic events are rare, suggest that it is reasonable to accept a lower factor of safety for a seismic analysis than the value appropriate for a static design. Applying an overall factor of safety to the soil shear strength equal to 1.1 to 1.15 is reasonable for seismic design. This recommendation is similar to that made by Seed (11) for the psuedo-static design of slopes in earth and rockfill dams constructed with materials that do not lose significant strength during earthquakes.

Number of Reinforcement Layers - The increase in the number of reinforcement layers required to maintain stability during a seismic event, with respect to the static loading case, will be based on three factors: (1) the increase in required tensile force, T_e, due to the horizontal inertial forces (Fig. 4); (2) the increase in allowable reinforcement tensile force, α_ε, associated with high strain rate loading conditions; and (3) the low permissable factor of safety for seismic design. When these three factors are combined in practical applications, it will often be found that the number of layers of reinforcement required for seismic design does not exceed the number required for the long-term gravity loading condition. Exceptions to this general conclusion include: (1) slopes subjected to very strong ground shaking; and (2) slopes in which very little reinforcement is needed for the static stability condition.

Length of Reinforcement - The increase in the length of reinforcement required to maintain stability during the seismic event, with respect to the static loading case, will be based n: (1) the increase in reinforcement length required to resist the horizontal inertial forces (Fig. 5); and (2) the low permissable factor of safety for the seismic event. When these factors are combined, it will often be found that a small relative increase in reinforcement length will be required. These increases will be largest for: (1) slopes subjected to very strong ground shaking; and (2) slopes in which very little reinforcement is needed for the static stability condition.

Illustrative Example - The practical conclusions reached in the preceding sections can be demonstrated through evaluation of a reinforced slope with typical soil and reinforcement properties. Parameters and results for this evaluation are given in Table 1. The following calculations were made for the long-term static condition (F.S. = 1.5, k = 0). The required tensile force was determined with the critical two-part wedge failure mechanism, and the required reinforcement length was determined as shown in Fig. 3. This resulted in T_s = 112 kN/m and L_s = 4.4 m. Thus, for long-term design using reinforcement with α_ε = 10 kN/m, 12 layers of reinforcement, 4.4-m long, would be required.

For the seismic case the above calculations were first repeated for k = 0 and a seismic factor of safety equal to 1.1. This resulted in T_s = 65 kN/m and L_s = 3.2 m. From Figs. 4 and 5, R_T and R_L are approximately equal to 1.75 and 1.5, respectively, for k = 0.15. Therefore, T_e = 114 kN/m and L_e = 4.8 m. For the reinforcement, it was assumed that R_α = 1.5. Thus, for the seismic loading condition, 8 layers of reinforcement, 4.8-m long would be required.

Table 1
INFLUENCE OF SEISMIC FORCE ON A TYPICAL REINFORCED SLOPE

Design Data

Slope Geometry:	H = 7m, 2V:1H
Soil Properties:	ϕ_{cv} = 35°, γ=18 kN/m^3, c =0
Safety Factors:	Static = 1.5, Seismic = 1.1
Seismic Data:	a = 0.18, k = 0.15
Reinforcement:	α_ε static = 10 kN/m, R_α =1.5

Static Design (F.S. = 1.5)
T = 112 kN/m : N = 12
L = 4.4 m

Seismic Design (F.S. = 1.1)

k = 0:	T = 65 kN/m	
	L = 3.2 m	
k = 0.15:	R_T = 1.75	T = 114 kN/m
	R_L = 1.5	L = 4.8 m
	R_α = 1.5	N = 8

Summary: Required number of reinforcement layers, N, did not increase for k = 0.15

Required reinforcement length, L, increased by about 10% for k = 0.15

Construction Considerations - A secondary effect of earthquakes on reinforced slopes constructed of frictional fill materials is ravelling and sloughing of the unconfined fill near the edge of the slope. This unconfined fill will be susceptible to movement under seismic excitation. A simple construction step to reduce this ravelling and sloughing potential is to place narrow strips of reinforcement at close vertical spacings at the edge of the slope to provide improved compaction and increased confinement. Use of this type of "intermediate" reinforcement with polymer grid and mesh structures has been described by Iwasaki and Watanabe (5) and Devata (2).

CONCLUSIONS

This investigation represents an initial attempt to understand the factors governing the behavior of reinforced slopes during earthquakes. Simple assumptions and analytical procedures have been used to evaluate the stability of dry slopes under pseudo-static, rigid body loading conditions. Clearly, additional experimental and analytical research is required to fully understand both the static and seismic behavior of slopes reinforced with extensible polymeric materials.

Preliminary conclusions which can be drawn from this investigation are:

- The number of additional layers of polymer reinforcement required to resist earthquake induced tensile forces in slopes, compared to the reinforcement required for static conditions, is often small. In many cases no additional layers are required.
- The primary reasons that additional reinforcement layers are often not required, evcn though seismically induced tensile forces are significant, are: (1) the visco-elastic properties of polymer reinforcement permit the use of higher reinforcement tensile forces under high strain-rate loading conditions; and (2) low factors of safety are typically used for seismic design.
- The length of reinforcement required to resist earthquake induced tensile forces in slopes is typically slightly greater than the length required for static loading conditions.
- Based on research by others, the dynamic increment in required tensile force should be distributed uniformly over the height of the slope.
- The allowable reinforcement tensile force for seismic design should: (1) consider the high loading strain rate; (2) ensure that brittle rupture of the reinforcement is precluded; and (3) result in working strains compatible with mobilization of the large-strain soil shear strength.

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