Use of geosynthetic basal reinforcement to limit lateral spreading of embankments subjected to earthquake loadings

Chris Lawson
TenCate Geosynthetics Asia Sdn Bhd, Malaysia

ABSTRACT: Historically, little attention was given to the seismic performance of embankments because they were not considered critical structures. Of recent times, their seismic stability has gained in importance because of the need to ensure serviceable infrastructure immediately following earthquake events. The major form of embankment distress during earthquakes is lateral spreading which occurs during liquefaction of the foundation. The paper provides a method where geosynthetic basal reinforcement is used to limit the lateral spreading of embankments during and following earthquake loadings thereby maintaining serviceability. The method uses a pseudo-static limit equilibrium approach to determine the tensile load generated in the basal reinforcement. By limiting the horizontal displacement of the embankment toe to meet serviceability requirements the maximum allowable basal reinforcement strain is established. The combination of reinforcement tensile load and maximum allowable strain enables a suitable geosynthetic reinforcement to be defined that accounts for tensile strength, strain, design life, as well as installation and durability effects.

Keywords: geosynthetics, basal reinforcement, embankments, earthquake loadings, liquefaction, instability, lateral spreading

1 INTRODUCTION

Historically, the seismic performance of infrastructure embankments was not considered important as damage was normally limited to a loss of serviceability with no resulting loss of life. Also, it was considered that corrective measures could be undertaken in a short period with services operating reasonably quickly. However, of recent times, more importance has been placed on the seismic performance of infrastructure embankments, especially near bridge structures, to ensure ready access for critical relief efforts in the immediate aftermath of earthquake events.

In assessing the effects of earthquake loadings on embankments one must take into consideration the earthquake magnitude and duration, likely changes in foundation shear strength and deformation during and immediately after the earthquake, the effect of aftershocks and the resulting instability and loss of serviceability of the embankment. Figure 1 shows various types of embankment instability resulting from earthquake loadings, e.g. Sasaki and Tamura (2007). These range from surface sliding of embankment fill (figure 1a), to development of slip surfaces within the embankment fill (figure 1b), to development of slip surfaces through an unstable foundation (figure 1c), to development of slumping and lateral spreading of the embankment fill on top of an unstable foundation (figure 1d). Figures 1a and 1b are where instability occurs within the embankment fill alone and where the foundation remains in a stable condition. Figures 1c and 1d are where the foundation becomes unstable, e.g. due to liquefaction, resulting in distress of the embankment. The use of basal reinforcement to enhance stability and prevent lateral spreading during and following an earthquake is only appropriate for the conditions depicted in figures 1c and 1d, as it is here that these forms of embankment distress interact directly with the basal reinforcement.

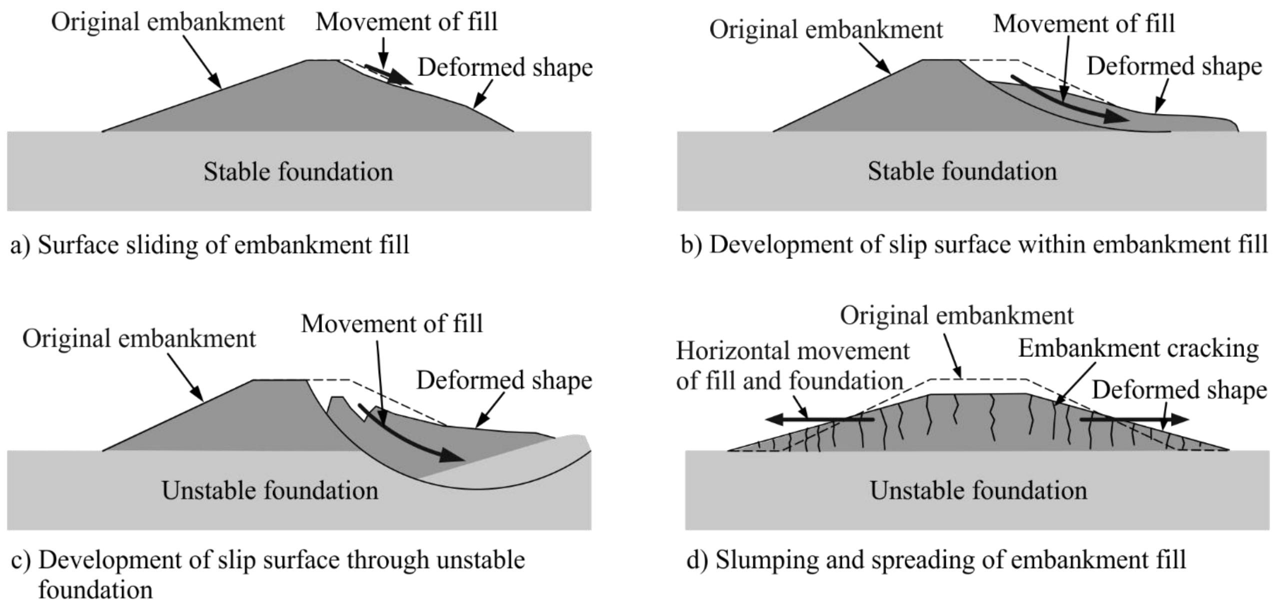

Figure 1. Types of embankment instability that can result from earthquake loadings.

The concept of using basal reinforcement to enhance stability and limit lateral spreading of embankments subject to earthquake loadings is shown in figure 2. Here, an earthquake loading causes a loss in strength, i.e. liquefaction, in the foundation stratum leading to lateral displacements and consequent embankment distress. The liquefied foundation layer undergoes changes in shear strength, beginning with ϕ'_p conditions prior to the earthquake loading, to residual shear resistance S_r conditions at liquefaction, to critical state shear strength ϕ'_{cv} conditions once equilibrium conditions have been re-established. The presence of the basal reinforcement prevents embankment instability occurring during the changes in these foundation shear resistance conditions and prevents consequent lateral spreading of the embankment. The method described in this paper assumes that liquefaction occurs in the foundation only, and that there is no raised water table within the embankment fill itself which may lead to internal embankment liquefaction.

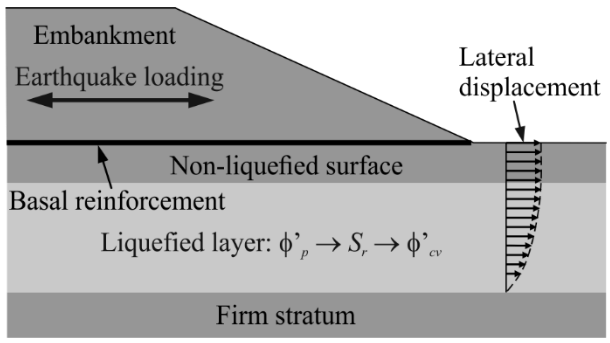

Figure 2. Concept of using basal reinforcement to prevent instability and lateral spreading of embankments.

In many cases, it is common for the liquefied foundation layer to be overlain by a non-liquefied surface layer (see figure 2). This surface layer consists of either a more-recently deposited fine soil layer, or may be the same soil as the liquefied layer but with the water table remaining at some depth below ground surface. Depending on the thickness of the surface layer horizontal lateral displacements are translated from the liquefied layer through to the ground surface. The role of the basal reinforcement is to ensure that these lateral displacements are not transferred into the embankment while maintaining embankment stability.

2 LIQUEFACTION OF EMBANKMENT FOUNDATIONS

Earthquake induced liquefaction is associated with a loss of strength and stiffness of the liquefied soil layer, with consequent large ground deformations due to the development of large excess pore pressures within this liquefied layer. The severity of liquefaction, and consequent ground deformations, depend on the magnitude of the embankment loading, the geometry (slope) of the liquefied layer, and the type of liquefied layer and its insitu density as well as the level of the water table. Saturated, recent sandy deposits of low to medium insitu density are the most prone to liquefaction, however, silty sands, sandy silts and gravelly sands are also prone.

Earthquakes vary in magnitude and in duration, ranging from as short as 5 seconds to over 2 minutes. Generally, the more powerful earthquakes also have longer durations. Where the initial earthquake shock may cause embankment instability, earthquake aftershocks can also cause instability and may make the initial condition worse. Liquefaction has been observed to occur following both short and long duration earthquakes, and this has resulted in the worst damage to embankments (Sasaki and Tamura 2007).

An important aspect of the design of embankments on a liquefiable foundation layer is to establish its susceptibility to liquefaction, and if so, its likely residual shear strength. Various procedures have been developed for assessing the potential for foundation liquefaction and the resulting residual shear strength. Boulanger and Idriss (2014) provide a review of how SPT and CPT insitu testing can be used to evaluate the susceptibility of foundation layers to liquefaction, while Idriss and Boulanger (2015) provide correlations between the same SPT and CPT insitu testing to estimate the residual shear strength S_r of liquefied soils. Design counter measures for embankments located on top of liquefaction susceptible soils have ranged from changing the location of the embankment to providing suitable ground improvement measures. These measures can prove to be very expensive.

The thickness of the non-liquefied surface layer (H_1 in figure 3a) has a major effect on the shape and extent of embankment instability. Where the surface layer thickness is extensive, i.e. $H_1 > 3$ m, the embankment would normally remain stable, and be unaffected by foundation liquefaction, as the surface layer is thick enough to mask the effects of liquefaction (e.g. Bowen and Jacka 2013). Where there is a substantial surface layer, i.e. 2 m < H_1 < 3 m, the form of embankment instability is lateral movement due to the lateral deformations generated in the liquefied foundation layer that are translated to ground surface, figure 3a. The resulting lateral spreading of the embankment is essentially horizontal in nature. Where there is an intermediate thickness surface layer, i.e. 1 m < H_1 < 2 m, the form of embankment instability is a combination of both lateral movement and rotational failure depending on the integrity of the surface layer, figure 3b. Here, loss of integrity of the surface layer may be observed by the occurrence of sand boils at ground surface. Where there is only a thin or no surface layer, i.e. $H_1 < 1$ m, the form of embankment instability is rotational failure with no integrity contributed by the surface layer, figure 3c. Thus, the nature of the lateral spreading of the embankment is related to the thickness of any non-liquefied surface layer and whether it maintains its integrity during liquefaction.

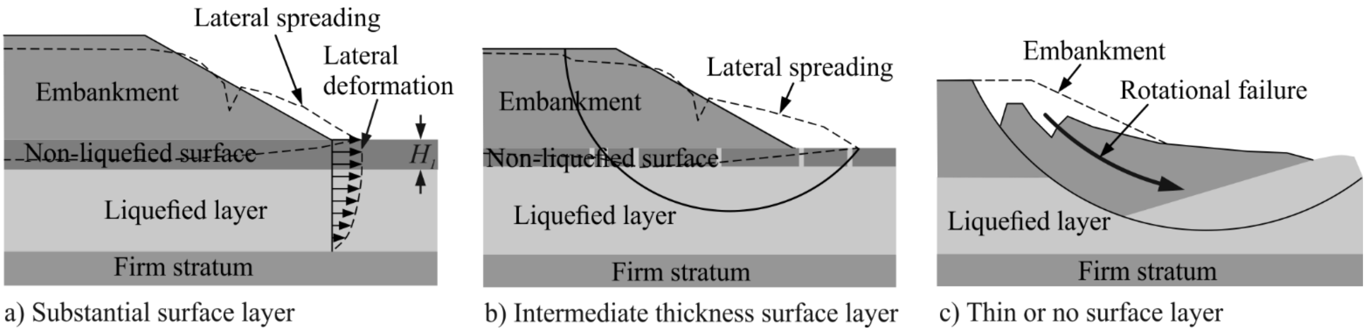

Figure 3. Influence of thickness of non-liquefied surface layer on shape and extent of embankment instability.

It should be noted that all three embankment instability modes shown in figure 3 can result in extensive lateral spreading of embankments. The cases shown in figures 3a and 3b can be analyzed using a simple approach dealing with horizontal loads and deformations only. The case in figure 3c where a rotational failure occurs is more complex as both vertical and horizontal loads and deformations should be analyzed.

3 USE OF BASAL REINFORCEMENT TO LIMIT LATERAL SPREADING OF EMBANKMENTS

Geosynthetic basal reinforcement has been used for many years to provide stability to embankments constructed on soft foundation soils. For the case where an earthquake loading results in liquefaction of the embankment foundation the role of the basal reinforcement is the same since it must maintain embankment stability and limit horizontal displacement until the foundation has stabilized and can support the embankment itself.

In analyzing the effect of earthquake loadings on geotechnical structures it is common to use a pseudo-static limit equilibrium approach which, historically, has given reasonable results. This approach accounts for earthquake effects by applying a horizontal seismic coefficient, k_h, (see figure 4) to the weight of the embankment sideslope, W. Generally, the magnitude of k_h is equal to a percentage of the Peak Ground Acceleration (*PGA*) at the site with different guidance documents recommending k_h = 100% to 20% of *PGA*. This large range of recommended values has led to confusion and over-conservatism in the values of k_h used for pseudo-static analyses. Recently, more rational procedures have been developed for the determination of the appropriate values of k_h (e.g. Bray and Travasarou 2011).

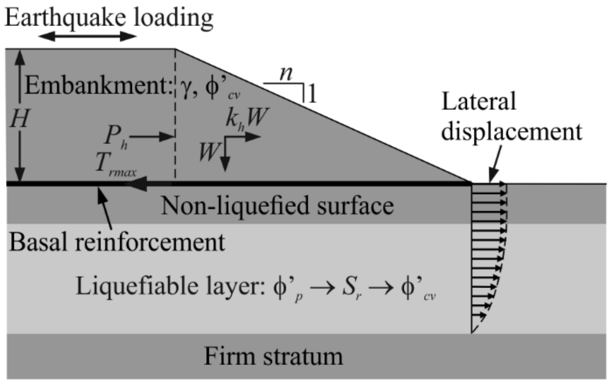

Figure 4. Pseudo-static limit equilibrium approach to determining the basal reinforcement load during and after earthquake loadings.

Details of the use of geosynthetic basal reinforcement to maintain stability and limit lateral spreading of an embankment during and immediately following an earthquake loading are shown in figure 4. The embankment geometry is normally defined based on static equilibrium using peak shear strength parameters (e.g. ϕ'_p) for the liquefiable foundation layer. However, at some point in time it is recognised that the embankment may be subject to an earthquake loading resulting in the liquefiable layer undergoing a loss in shear strength to a residual value, S_r. Under the embankment loading, this loss in foundation shear resistance can result in large lateral displacements until equilibrium conditions are reached where the liquefiable layer reverts to its critical state shear strength defined by ϕ'_{cv}. The role of the basal reinforcement during this time is to maintain embankment stability and prevent the lateral displacement of the toe of the embankment progressing beyond a safe, serviceable level.

Where the primary directional instability of the embankment is essentially horizontal (i.e. outwards), such as occurs where an appreciable non-liquefied surface layer exists (figures 3a and 3b), the maximum tensile load generated in the basal reinforcement during an earthquake loading T_{rmax} may be calculated as follows:

$$T_{r\max} = \frac{1}{2}\gamma H^2 \left\{ \tan^2\left(\frac{\pi}{4} - \frac{\phi'_{cv}}{2}\right) + k_h n \right\} \tag{1}$$

where the various parameters are shown in figure 4.

To ensure the basal reinforcement does not pull-out of the embankment during the earthquake loading, the required reinforcement bond length beyond the embankment failure plane L_E should be:

$$L_E \geq \frac{f_p T_{r\max}}{2\gamma H \alpha' \tan\phi'_{cv}} \tag{2}$$

where the various parameters are shown in figure 4, α' = bond coefficient between the basal reinforcement and surrounding soil, f_p = factor of safety against reinforcement pull-out (normally equal to 1.3 or 1.5). For situations where there is negligible or no non-liquefied surface layer (figure 3c) a more sophisticated method should be used to determine T_{rmax}, such as slip-circle stability analysis incorporating pseudo-static components.

The sideslopes of the embankment should be flat enough to ensure the base of the embankment cannot slide laterally over the surface of the basal reinforcement during the earthquake loading. The minimum embankment sideslope length ratio n to prevent sliding over the basal reinforcement is:

$$n \geq \frac{f_s \tan^2\left(\frac{\pi}{4} - \frac{\phi'_{cv}}{2}\right)}{\alpha' \tan\phi'_{cv} - f_s k_h} \text{ or } n \geq \frac{f_s \tan^2\left(\frac{\pi}{4} - \frac{\phi'_{cv}}{2}\right)}{(1-k_v)\alpha' \tan\phi'_{cv} - f_s k_h} \tag{3}$$

where the various parameters are shown in figure 5, f_s = factor of safety against sliding (normally equal to 1.3 or 1.5). The left-hand relationship in Equation (3) is where only the horizontal seismic coefficient is considered when assessing sliding resistance over the basal reinforcement. However, it may be prudent to account for both horizontal and vertical seismic coefficients in a sliding resistance assessment as this provides a more conservative (i.e. safe) determination, and this is shown in the right-hand relationship in Equation (3).

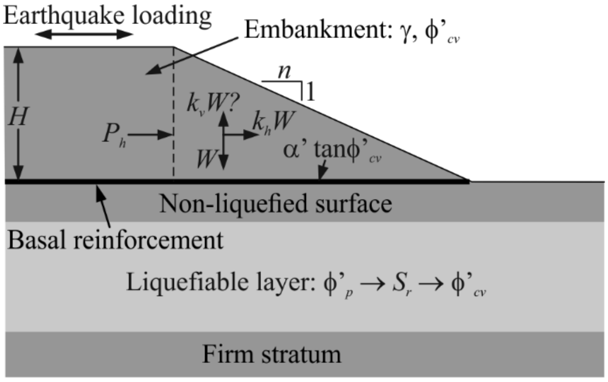

Figure 5. Description of parameters used in Equation (3).

To maintain embankments in a serviceable condition their maximum horizontal toe displacements should be limited to 0.2 m (generally for embankment heights up to 4 m) or 0.3 m (generally for embankment heights over 4 m). Where pile foundations exist near embankments, e.g. for bridge abutment embankments, greater restrictions may be required on the allowable maximum toe displacement than stated above. Figure 6 shows the maximum allowable reinforcement strain ε that is required to fulfil this toe displacement criterion for various embankment heights *H* and sideslope length ratios *n*. It is observed that for most practical cases the maximum allowable reinforcement strain ranges between 2% and 5%.

The pseudo-static reinforcement load T_{rmax} calculated using Equation (1) must be compatible with the maximum allowable reinforcement strain ε obtained from figure 6. This is shown in Figure 7. From this, the basal reinforcement design strength T_D is determined as follows:

$$T_D = \left(\frac{T_m}{T_{r\max}}\right) T_{r\max} \tag{4}$$

where, T_m = reinforcement strength compatible with reinforcement load T_{rmax} and allowable reinforcement strain as shown in figure 7.

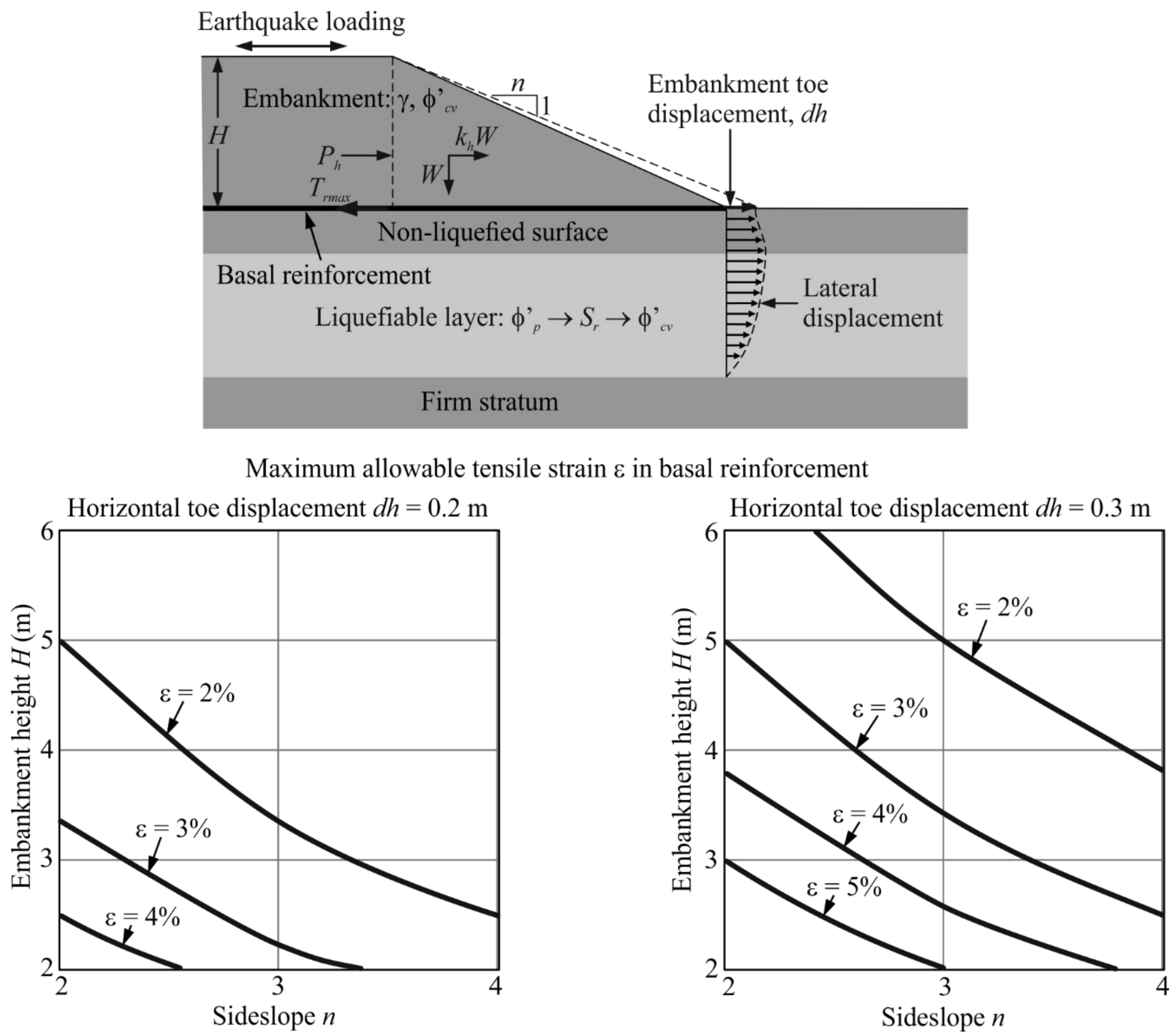

Figure 6. Maximum allowable strain in basal reinforcement for limiting horizontal toe displacement.

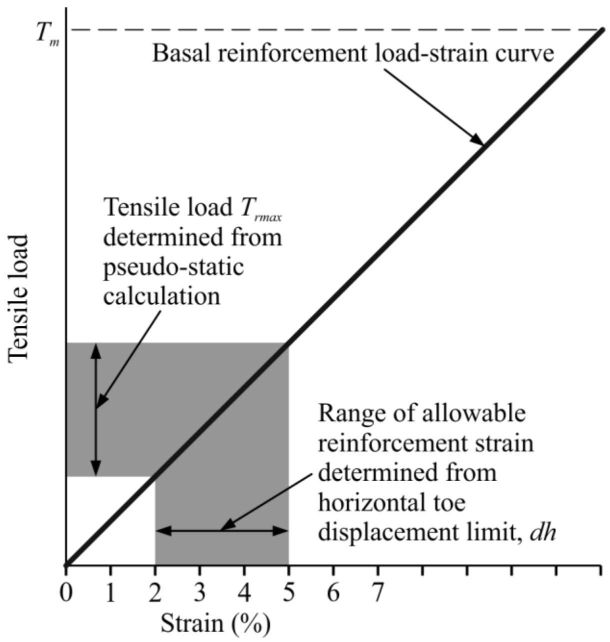

Figure 7. Compatibility between basal reinforcement load and maximum allowable strain.

4 BASAL REINFORCEMENT LOAD AND DESIGN STRENGTH OVER TIME

For this application, the basal reinforcement is used as a form of insurance where it is required to perform when (or if) a design earthquake loading occurs at some point during the design life of the embankment. As shown in figure 8, an earthquake loading may occur at any time during the design life of the embankment. From a design perspective, the worst case is where the design earthquake loading occurs at, or near, the end of the design life of the embankment (at t_d in figure 8). Here, the reinforcement design strength T_D calculated using Equation (4) equals the insitu strength of the reinforcement at the design life t_d of the embankment.

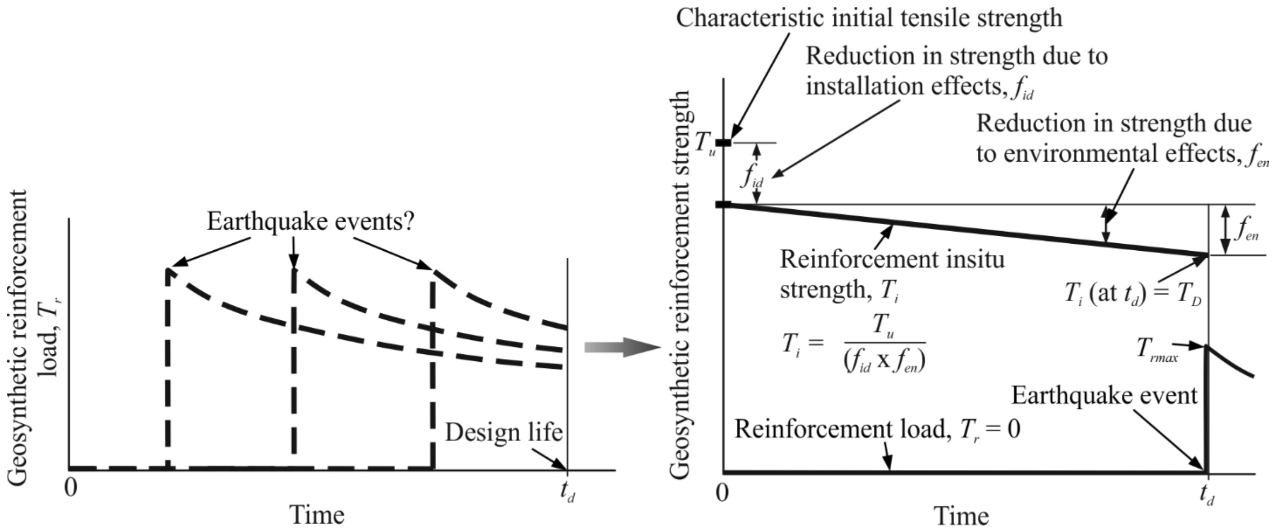

Figure 8. Principle of basal reinforcement load and design strength over time.

The reinforcement insitu strength T_i at any time during the design life of the embankment is equal to its characteristic initial tensile strength T_u divided by a reduction factor due to installation damage f_{id}, divided by a reduction factor due to environmental effects f_{en}, figure 8. At the design life of the embankment, $T_i = T_D$, and thus,

$$T_u = T_D \left(f_{id} \times f_{en} \right) \tag{5}$$

The three important reinforcement performance properties are:

1. The required design life of the basal reinforcement (120 years?).
2. The maximum reinforcement load T_{rmax} at the maximum allowable reinforcement strain ε:
3. The characteristic initial tensile strength T_u of the reinforcement.

5 EXAMPLE

Suppose we have the embankment geometry shown in figure 9a which will have a 120-year design life. From a pseudo-static stability analysis the maximum basal reinforcement tensile load has been determined as T_{rmax} = 150 kN/m. The horizontal displacement of the embankment toe *dh* is limited to 0.3 m to maintain embankment serviceability. Determine the initial tensile strength of the appropriate geosynthetic basal reinforcement if its load-strain characteristic curve is as shown in figure 9b. The reduction factor for installation effects in sand fill is f_{id} = 1.10 and the reduction factor for environmental effects over 120 years is f_{en} = 1.06.

From figure 6, the maximum allowable basal reinforcement strain ε = 3% to restrict the horizontal toe displacement $dh \leq 0.3$ m. From the reinforcement curve in figure 9b, this maximum allowable strain ε = 3% generates a reinforcement load of 28% of T_m. Using Equation (4), the reinforcement design strength at 120 years T_D = (100/28)x150 = 536 kN/m. Now, using Equation (5), the characteristic initial tensile strength of the geosynthetic reinforcement T_u = 536 x 1.10 x 1.06 = 625 kN/m → 700 kN/m.

The two important specification requirements for the basal reinforcement is that it has to generate 150 kN/m load at 3% strain and it has a minimum characteristic initial tensile strength of 700 kN/m using appropriate material reduction factors for installation damage and environmental effects over 120 year embankment design life.

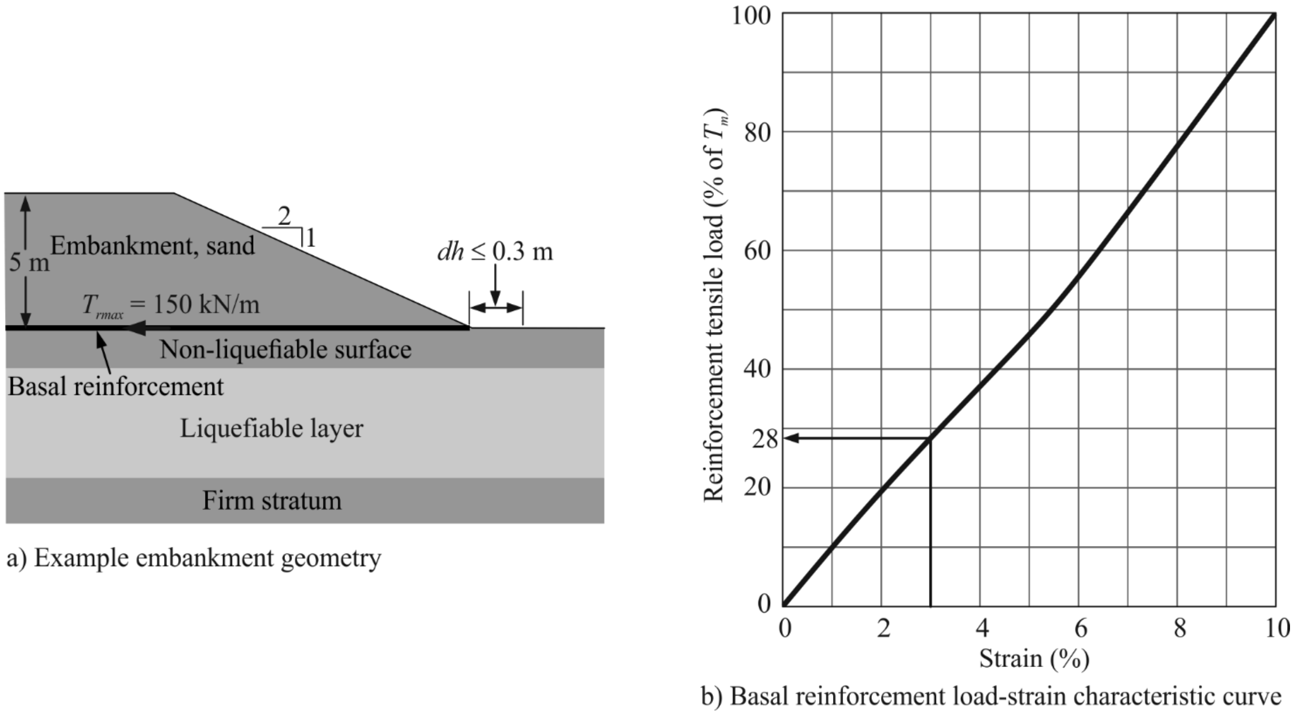

Figure 9. Basal reinforcement design example.

6 CONCLUSIONS

Geosynthetic basal reinforcement would appear to provide a cost-effective solution to the problem of embankment instability and lateral spreading during and after earthquake loadings. Here, the basal reinforcement is used as a form of insurance, and when the earthquake loading occurs, maintains the embankment in a serviceable condition throughout any liquefaction phase of the foundation layer.

A method is given to determine the reinforcement load using a pseudo-static analysis. When this is combined with a maximum allowable reinforcement strain to ensure embankment serviceability is maintained, the required reinforcement ultimate tensile strength properties can be determined. The critical timing of the earthquake loading on basal reinforcement performance is when it occurs on reaching the design life of the embankment.

An example is presented where the required reinforcement ultimate strength is determined based on a reinforcement load and reinforcement strain to satisfy the earthquake loading condition and embankment design life.

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