BEHAVIOR OF REINFORCED EMBANKMENT OVER SOFT SUBSOIL

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ABSTRACT: The functions of reinforcement on the behavior of reinforced embankment over soft subsoil are discussed first. The mobilized tensile force in the reinforcement contributes to the stability of the embankment, and the confining effect of the reinforcement to embankment fill material and soft subsoil can increase the bearing capacity and reduce the lateral deformation of soft subsoil. However, the effect of the reinforcement on soft subsoil deformation can only become significant when soft subsoil approaches to failure. Then some techniques for modeling the reinforced embankment on soft subsoil are presented, namely, modeling the soil/reinforcement interaction behavior, embankment construction process and hydraulic conductivity variation during consolidation process of soft subsoil. It is demonstrated that these factors are important on simulating the behavior of the reinforced embankment on soft subsoil. Finally, two case histories of the reinforced embankment on soft subsoil are presented. One of the embankments is a geogrid reinforced embankment on Muar clay deposit in Malaysian and another one is a built-to-failure geotextile reinforced embankment in Lian-Yun-Gang, China.

INTRODUCTION

In case of constructing an embankment over soft subsoil of low strength and high compressibility, the engineering tasks are to prevent the failure of the embankment and to control the subsoil deformation within an allowable limit. Several methods have been developed for economically and safely constructing embankment on soft subsoil. Using basal reinforcements is one of the methods.

The mechanisms of reinforcing an embankment over soft subsoil are discussed elsewhere (Bonaparte and Christopher 1987; Jewell 1988). The functions of basal reinforcement are: (1) increasing the stability of embankment by mobilized tensile force in the reinforcement, and (2) providing a confinement to embankment fill and foundation soil adjacent to the reinforcement. This confining effect can reduce the lateral distortion to subsoil due to embankment load, and therefore, the shear stress in soft subsoil. However, the mobilization of these functions depends on the soil/reinforcement interaction behavior, which relates to the strength of reinforcement, the relative stiffness of reinforcement and subsoil, embankment geometry, and the factor of safety (FS) of the embankment at working state.

This paper first describes the functions of basal reinforcement for an embankment over soft subsoil. Then some techniques of modeling the reinforced embankment on soft subsoil are discussed. Finally, two case histories of reinforced embankment on soft subsoil are presented to illustrate the magnitude of the reinforcement effects on FS of the embankment and subsoil deformation.
FUNCTIONS OF REINFORCEMENT

Effect of reinforcement on FS of embankment

Practically, the limiting equilibrium method is used in analyzing FS of a reinforced embankment on soft subsoil. In most proposed methods (e.g. Milligan and Rochelle 1984; Myleville and Rowe 1988), it is assumed that the reinforcement does not change the failure surface much and only considers the restoring moment due to the mobilized reinforcement tensile force. The FS of a reinforced embankment can be expressed as the follows.

\[
\text{FS} = \frac{\text{Restoring} \cdot \text{Moments}}{\text{Overturing} \cdot \text{Moments}} = \frac{\text{MRR} + \text{MRSOIL}}{\text{MOSOIL}}
\]

(1)

where MRR is restoring moment due to tensile force developed in the reinforcement, MRSOIL is restoring moment due to the mobilized shear strength of soil along the failure surface, and MOSOIL is overturning moment due to embankment fill. The effect of MRR on FS depends on the magnitude of MRR and the ratio of MRR/MRSOIL. For a given value of MRR, the larger the embankment (larger the MRSOIL), the smaller the MRR/MRSOIL, and the less effect of MRR on FS.

The magnitude of MRR depends on the mobilized tensile stress in reinforcement (T). T depends on both mobilized tensile strain in reinforcement and the reinforcement stiffness. Rowe and Soderman (1985) discussed the strain compatibility between reinforcement and soil and pointed out that the allowable compatible strain, which is the tensile strain for design reinforcement, is a function of both subsoil properties and the geometry of embankment. From their finite element analysis results, the allowable compatible strain is in a range of 1% to 9%. By reviewing some of the published data, Bonaparte and Christopher (1987) suggested that the allowable strain is mainly controlled by subsoil properties. The recommended values are 2~3% for sensitive subsoil, 4~6% for medium to low sensitive subsoil, and 6~10% for non-sensitive plastic subsoil.

For geogrids and woven geotextiles, the confined in-soil stiffness is not different much from the in-air value. However, for non-woven geotextiles, it is generally agreed that the soil confinement increases the friction resistance between fabrics of geotextile and therefore, the stiffness of a geotextile. Miura and Chai (1999) proposed a method for determining the in-soil stiffness of a geotextile by combining a small-scale soil/geotextile interface shear test and a large-scale pullout test results. For the geotextile tested, for tensile strain less than 5%, the in-soil stiffness was 2~3 times of in-air one. Beyond this limiting strain, the in-soil tangent stiffness was the same as the in-air value.

There has been debate on the orientation of the mobilized tensile force in the reinforcement, i.e. horizontal or tangent to the failure surface. Rowe and Li (2002) stated that there is strong evidence that the reinforcement tensile force should be taken to act in its original horizontal orientation.

In calculating the FS of a reinforced embankment on soft subsoil, evaluating the mobilized undrained shear strength ($S_u$) of soft subsoil is also an important issue. Ladd (1991) proposed an empirical equation for estimating $S_u$ of soft clay as follows:

\[
S_u = S(OCR)^m \sigma_v'
\]

(2)

where $S$ and $m$ are constants, $OCR$ is overconsolidation ratio, and $\sigma_v'$ is vertical effective stress. Normally, the value of $S$ is in a range of 0.2~0.4, and $m$ is from 0.75 to 1.0. If $S$ value can be determined by local experience or back calculated from some case histories, Eq. 2 provides a reasonable estimation of $S_u$ of soft subsoil.

For embankment construction, the critical time is the end of construction. During construction, the partial drainage effect can increase the effective stress in subsoil and therefore the value of $S_u$, especially for prefabricated vertical drain (PVD) improved subsoil.

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Chai and Miura (2000) developed a one-dimensional finite element program to predict the value of \( S_u \) of PVD improved subsoil at the end of embankment construction.

**Effect of reinforcement on subsoil response**

As shown in Fig. 1(a) that in case of without reinforcement, the lateral spreading of fill material will induce the outward shear stress (\( \tau \)) on the surface of the soft subsoil, which will cause the reduction of the bearing capacity and increase the lateral displacement of the soft subsoil. In case of reinforced embankment, the reinforcement provides a confinement to the embankment fill and the soft subsoil (Fig. 1(b)) and reduces the lateral displacement of the soft subsoil.

**Bearing capacity**

With a high stiffness and strength reinforcement, the slip surface is difficult to pass through the embankment itself and the failure mechanism of the embankment will be the bearing capacity failure of soft subsoil. The ideal (or ultimate) condition is that the reinforced embankment behaves like a rigid footing. From general bearing capacity theory, the ultimate bearing capacity (\( q_u \)) of a rigid footing on soft subsoil can be expressed as:

\[
q_u = N_c S_{u0} + q_s \quad (3)
\]

where \( N_c \) is bearing capacity factor, \( S_{u0} \) is initial undrained shear strength of soft subsoil, and \( q_s \) is distributed surcharge load on the surface of subsoil. The strength of natural clay deposit is not uniform and normally increases with depth. Davis and Booker (1975) published a solution considering the effect of strength increase with depth. For rough footing case, Rowe and Soderman (1987) provided a graphic form (Fig. 2) for obtaining the bearing capacity factor, \( N_c \), based on Davis and Booker’s solution.

Under embankment loading, the loading intensity varies under the shoulder. Rowe and Soderman (1987) proposed an approximate method to convert the embankment load to an equivalent rigid footing as illustrated in Fig. 3. From plasticity solution, the pressure at the edge of a rigid footing is \( (2 + \pi)S_{u0} \). Under embankment shoulder, there is a point with a fill thickness of \( h^* \), at which the following condition holds:

\[
\gamma h^* = (2 + \pi)S_{u0} \quad (4)
\]

where \( \gamma \) is unit weight of embankment fill. Then the effective width \( (b) \) of the equivalent
footing will be between these kind points on either side of the embankment and can be expressed as the following:

\[ b = B + n(H - h^*) \]  

(5)

where \( B \) is top width and \( n \) is cotangent of the embankment slope angle. The equivalent loading will be the average embankment fill loading within \( b \). The embankment loads outside the effective width (\( b \)) are considered as surcharge loads and averaged within a distance \( x \) from the edge of the footing. The value of \( x \) approximately equals to the depth (\( d \)) to which the failure mechanism is expected to extend and can be estimated from Fig. 4 (after Rowe and Li 2002). At the either side of the embankment toe, the embankment width will be considered as surcharge loading is \( nh^* \). In case that \( nh^* > x \), only the loads within \( x \) will be considered.

Subsoil deformation.

The confinement from the reinforcement to soft subsoil can reduce the lateral spreading of the subsoil and which can contribute to reduce the settlement. Rowe and Li (2002) stated that for a particular geometry of embankment and soil profile, there is a threshold reinforcement stiffness below which the reinforcement has no effect on settlement. However, Chai et al. (2002) demonstrated that stress level in subsoil is a most crucial factor controlling the effect of the reinforcement on subsoil deformation. It is concluded that only when the embankment load reaches the level at which the unreinforced case very close to failure (FS=1.0), the reinforcement will have obvious effect on subsoil deformation. At working condition of FS=1.2 to 1.3, the effect of reinforcement on subsoil deformation is small and insignificant. Figure 5 is used to illustrate why the effect of reinforcement on subsoil deformation increases when embankment approaches failure. Normally, the stress-strain curve of soft clay soils is non-linear. Assuming that the reinforcement can reduce certain amount of shear stress (\( \Delta \tau \)) on a soil element. At \( P-1 \), with a higher FS (about 1.2), the reduction of shear stress \( \Delta \tau \) can result in a reduction of shear strain of \( \Delta \varepsilon_1 \). However, at \( P-2 \) with a value of FS close to unity, \( \Delta \tau \) can result in a larger shear strain reduction \( \Delta \varepsilon_2 \) (\( \Delta \varepsilon_2 >> \Delta \varepsilon_1 \)).

MODELING REINFORCED EMBANKMENT ON SOFT SUBSOIL

The behavior of the reinforced embankment on soft subsoil depends on soil/reinforcement interaction as well as fill/soft subsoil interaction. Finite element method (FEM) is suitable for analyzing this kind of problem. However, the accuracy of the finite element analysis not only depends on the constitutive models and the parameters used but also on the numerical techniques adopted, such as selecting the proper soil/reinforcement interface properties, the methods of applying the embankment load and simulating the hydraulic conductivity variation during consolidation process of soft subsoil.
Modeling soil/reinforcement interaction

Soil/reinforcement interface property is one of the important parameters that influence the behavior of the embankment. Soil/reinforcement interaction mode can either be direct shear or pullout (Fig. 6, after Hird and Kwok 1989). For grid or strip reinforcements, these two different interaction modes will yield different interface strength and deformation parameters. Usually, the direct-shear mode will yield higher interface strength than the pullout mode (Myklebust and Rowe 1988; Chai and Bergado 1993b). The finite element technique should be able to automatically select proper interface properties according to the interaction modes. The technique proposed by Chai and Bergado (1993b) considers the interface elements above and below the reinforcement as pair elements, and the signs of the shear stresses of the pair elements are compared to determine whether the direct shear (same sign) or the pullout (different sign) is the acting mode.

Modeling the construction process

The actual embankment construction is carried out by placing and compacting fill material layer by layer. In finite element analysis, the mesh for embankment is pre-defined and the embankment load is applied by turning on the gravity force of the embankment elements layer by layer. If the construction settlement is not significant or if the embankment height (not fill thickness) at the end of construction is specified, this process is satisfactory. However, for embankments on soft subsoil, if the fill thickness is specified during the analysis, such as predicting the embankment behavior under a given fill thickness, this process will yield a wrong result. The analysis of embankment on soft subsoil is a large deformation problem because amount of settlement at the end of construction can vary from 20 to 100% of the total settlements (Asaoka et al. 1992). The large deformation phenomenon can be approximately considered by updating the nodal coordinates during the incremental analysis. Usually, this operation does not include the elements above the current construction level. As a result, the finally applied embankment fill thickness will be larger than pre-defined value. There are two methods to treat this problem.

1. Include all the embankment elements into all steps of incremental analysis and apply the gravity force by percentage in each step (Britto and Gunn 1987).

2. Apply the embankment elements layer by layer and correct the coordinate of the elements above the current construction level according to the settlement of the current construction top surface as illustrated in Fig. 7 (after Chai and Bergado 1993a).

In case of an elastic ground, the above two methods may not yield much different results, but soft subsoil behaves elasto-plastically and these two methods give a quite different results. For a reinforced embankment on soft Muar clay deposit, Malaysia, the settlement profiles obtained by
the above two approaches are compared in Fig. 8 (after Chai and Bergado 1993b). It can be seen that the method of applying the embankment elements layer by layer and correct the coordinate of un-constructed elements yielded a much better simulation of the field behavior. The details of the embankment will be described in next section.

Modeling hydraulic conductivity variation

Hydraulic conductivity \( (k) \) is one of the main factors controlling the consolidation process of soft subsoil. The value of \( k \) depends on the size, shape and distribution of voids of soil. Taylor (1948) proposed a relation between \( k \) and the void ratio \( (e) \) of clay as follows.

\[
k = k_0 \cdot 10^{-(e_0 - e)/C_k}
\]  

(6)

where \( k_0 \) is initial hydraulic conductivity, \( e_0 \) is initial void ratio, \( k \) is current hydraulic conductivity, \( e \) is current void ratio, and \( C_k \) is a constant \((C_k=0.5e_0, \text{Tavenas et al. } 1983)\) .

The coefficient of consolidation \( (C) \) is a function of \( k \) and the coefficient of volume change \( (m_v) \). Most elasto-plastic soil models for clay use a linear \( e-\ln(p') \) relation \((p' \text{ is effective mean stress})\). For structured natural clay, Chai (2001) proposed a linear \( \ln(e+e_c)-\ln(p') \) relation \((e_c \text{ is a constant})\). From these relationships, the value of \( m_v \) can be evaluated. Fig. 9 shows an example comparison of the

<table>
<thead>
<tr>
<th>Deposit</th>
<th>( C_f ) value</th>
<th>Method for evaluation of hydraulic conductivity</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>Bangkok clay at Asian Institute of Technology campus (about 100 km from sea)</td>
<td>25</td>
<td>Back-analysis</td>
<td>Chai et al. (1995)</td>
</tr>
<tr>
<td>Bangkok clay at Nong Hao (close to sea)</td>
<td>4</td>
<td>Back-analysis</td>
<td>Chai et al. (1996)</td>
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<tr>
<td>Malaysia Muar clay deposit</td>
<td>2</td>
<td>Back-analysis</td>
<td>Chai and Bergado (1993a)</td>
</tr>
<tr>
<td>Ariake clay (close to sea area)</td>
<td>4</td>
<td>Back-analysis</td>
<td>Chai and Miura (1999)</td>
</tr>
<tr>
<td>Louiseville (Canada)</td>
<td>About 1(^a)</td>
<td>Self-boring permeameter</td>
<td>Tavenas et al. (1986)</td>
</tr>
<tr>
<td>St-Alban (Canada)</td>
<td>About 3(^a)</td>
<td>Self-boring permeameter</td>
<td>Tavenas et al. (1986)</td>
</tr>
<tr>
<td>Soft mucky clay (eastern China)</td>
<td>6</td>
<td>Back-analysis</td>
<td>Shen et al. (2000)</td>
</tr>
</tbody>
</table>

\(^a\) Laboratory value was determined by direct measurement. For other cases, laboratory values were deduced from \( C_f \) value \((C_f \text{ is the vertical coefficient of consolidation})\).
variation of $C$ with consolidation pressure under several different assumptions. In the figure, “$k$ varies” means that $k$ varies with void ratio according to Eq. 6. It is very clear that if not considering the $k$ variation with void ratio, $C$ will be over estimated.

Another factor is that laboratory test normally under-estimates the field $k$ value because of sample disturbance and sample size effect. If define the ratio between the field value and the laboratory value as $C_f = \frac{k_{\text{field}}}{k_{\text{laboratory}}}$, some available data of $C_f$ are summarized in Table 1. It is recommended that the field value of $k$ should be obtained by back-analysis of case histories or the field hydraulic conductivity measurement.

**TWO CASE HISTORIES**

A geogrid reinforced embankment on Muar clay deposit, Malaysia, and built to failure embankments (with and without reinforcement) at Lian-Yun-Gang, China, are presented. Both the field measured data and the results of finite element analysis are presented. For the details of the FEM analysis, readers can refer Chai and Bergado (1993a) and Chai et al. (2002).

**A geogrid reinforced embankment on Muar clay deposit, Malaysia**

A geogrid reinforced embankment was constructed on soft Muar clay, Malaysia. The soil profile at the site consisted of a weathered crust at the top 2.0 m which is underlain by about 5 m of every soft silty clay. Below this layer lies a 10 m thick layer of soft clay which in turn is underlain by about 0.6 m of peat. Then, a thick deposit of medium dense to dense clayey-silty sand is found below the peat layer. In order to ensure that the estimated settlement can be obtained within the allowed construction period of 15 months, vertical band drain (without filter) were installed in foundation with a square pattern of 2.0 m spacing to 20 m depth. After vertical drain installation, geogrid were placed at the base of the embankment to ensure FS about 1.3 during construction (Ting et al. 1989). Two layers of geogrid were laid in a 0.5 thick sand blanket with 0.15 m vertical spacing. The strength of the geogrid was 110 kN/m with a failure strain of 11.2%. A stiffness of 650 kN/m was adopted.

The embankment was constructed with a base width of 88 m and length of 50 m, initially to a fill thickness of 3.9 m. Then a 15 m wide berm was left on both sides and the embankment was constructed to a final fill thickness of 8.5 m. The geometry and the field instrumentation points are shown in Fig. 10.

**Excess pore pressure**

A typical variation of the excess pore pressure with elapsed time for a piezometer point 4.5 m below ground surface and on the embankment centerline is shown in Fig. 11. In the figure, “Higher permeability” means that the vertical hydraulic conductivity is two time of that “Low permeability”, and “Varied
permeability” means that the initial values are the same as “Higher permeability” but varied with void ratio following Eq. 6. It can be seen that the “Varied permeability” analysis, simulated both excess pore pressure build up and dissipation for stage constructed embankment well. The settlement and lateral displacement presented in the next paragraph are the results of “Varied permeability” analysis.

Settlement and lateral displacement

The measured and simulated settlements and lateral displacement profiles are depicted in Figs. 12 and 13. For this case, the finite element analysis simulated the field behavior very well. Although this is a class “C” prediction, it is encourage that FEM has the ability to predict the behavior of the reinforced embankment on soft subsoil.

Effect of the reinforcement on stability

The calculated reinforcement tension force distributions for different fill thicknesses are shown in Fig. 14. It shows that at the early stage, higher tension force developed near the embankment toe at the location of higher shear stress level zone. Later on, since the berms are placed on both sides of the embankment and also due to consolidation effect, the tension force increased at the embankment center position and decreased under the berm. At the end of the construction, the calculated maximum tension force in each layer of the geogrids is 13 kN/m (totally 26 kN/m), which is equivalent to 2% of axial strain. Slip circle analysis result indicates that this tensile force can only increase the FS of the embankment about 0.015. If the tensile strength can be fully mobilized, it can increase the FS about 0.12.

Soil/reinforcement interaction mode

Figure 15 shows the calculated shear stress distributions at soil/reinforcement interfaces of bottom layer of the geogrid at different fill thickness. The sign convention is also shown in the figure. It can be seen that the signs of shear stress at upper and lower interfaces are the same for most interface areas, i.e. the direct shear interaction mode is applicable for this case. In the zone near the toe of the embankment and the intersection point between the berm and the main embankment, the lateral displacement of the fill is large because of the free face of the
embankment fill, and the interface shear stress has a negative sign. In other zones, the lateral squeezing of the foundation soil causes the interface shear stresses to have a positive sign. If assuming the reinforcement as steel grid with a stiffness of 156 MN/m, the pullout interaction mode is observed.

**Built to failure embankments at Lian-Yun-Gang, China.**

The test site is located in an alluvial plain, Lian-Yun-Gang area, Jiangsu province, China. The region belongs to lowland with an altitude of 2.6 m. The ground water level fluctuated between 0.5~1.0 m below ground surface. The soil profile consists of 2.0 m thick clay crust underlain by 8.5 m thick soft clay layer (called mucky clay in China). Below the soft layer there are medium to stiff sandy clay and silt sand layers.

Two built to failure embankments each had a length of 45 m. The embankments had a base width of 42 m. After placing a 0.5 m thick sand mat, a berm with a width of about 8.0 m was left on both sides and the embankments were built to failure. The embankments had a 1V:1.75H slope. Fill material was sandy clay and the average filling rate was about 0.1 m/day. The embankment geometry, the location of reinforcement, and the main instrumentation points for geotextile reinforced embankment are illustrated in Fig. 16.

Two types of geotextiles were used. One was a woven polypropylene geotextile with a unit weight of 303 g/m². In-air tensile strength from wide-width strip test (ASTM 1994) was 40 kN/m and failure strain was about 18% at a strain rate of 2%/min. Another was a heat-bounded non-woven geotextile with a unit weight of 260.8 g/m². In-air tensile strength from wide-width strip test was 38.5 kN/m and failure strain was about 20%.

**Effect of reinforcement on failure fill thickness**

The field measured failure fill thickness was 4.35 m for reinforced case and 4.04 m for unreinforced case. In analysis, the modeled strength of subsoil was adjusted (slightly change the yield locus) in such a way that the analyzed failure fill thickness for unreinforced case is the same as field value of 4.04 m. Then, the reinforced case was analyzed, which yielded a failure fill thickness of 4.25 m and 0.10 m less than the field value.

In order to illustrate the effect of the reinforcement on embankment failure height, an analysis with other conditions the same as reinforced case but no reinforcement (refer to as assumed case) was conducted. The numerical results revealed that, for the assumed case, the
failure fill thickness is the same as reinforced case. The analysis indicates that the difference of failure fill thickness of test embankments may not only be due to the effect of reinforcement alone. The slight difference in construction schedule may also have contributed to the difference of the embankment failure fill thickness, as more consolidation takes place with longer construction period.

Since it is not convenient to obtain a value of FS from FEM analysis, the limit equilibrium analysis was conducted to quantify the effect of reinforcement on FS of the embankment. The result indicates that the reinforcement may only increase FS of the embankment by less than 0.055 (assume a mobilized tensile strength of 40 kN/m). Symmetric failure pattern was observed in the field, i.e. the embankments failed at both sides. The observed failure surfaces are not much different for with and without geotextile reinforcements. It supports the hypothesis in conventional stability analysis that the existence of geotextile reinforcements does not alter the failure surface much.

**Surface settlement**

The measured and simulated surface settlement-time curves under embankment centerline together with embankment construction histories are shown in Fig. 17. For reinforced case, up to just before failure, the FEM analysis fairly simulated the field value. However, for unreinforced case, the analysis over predicted the field value. Also, the following observations can be made from the figure. (a) Before fill thickness exceeded 2.0 m, the settlement was small. The top crust is stiffer and when the surcharge load was less than the yield stress of the crust, the crust limited the subsoil deformation. (b) When the embankment approached failure-state, the settlement was rapidly increased due to the lateral distortion of the subsoil. (c) Just before failure, measured data shows that the geotextile reinforcement slightly reduced the settlement rate. It is reasoned that the combination of geotextile (strong in tension) and top crust (strong in compression) improved the contribution of the top crust to the embankment stability.

**Lateral displacement**

The lateral displacement profiles of reinforced case are given in Fig. 18. For unreinforced case, the measured data are not available. It indicates that the lateral displacement was small before the fill thickness reached 4.35 m (failure occurred). Before the failure, the simulated values over-predict the measured values and when the failure state is reached, the simulation under-predict the field values at near ground surface. Generally, FEM analysis yielded a poor simulation on lateral deformation. This is partially due to the adopted soil model may not fully represent the behavior.
of subsoil and partially due to the limitation of FEM analysis in simulating large deformation at close to failure state. Comparing the assumed case (unreinforced) and reinforced case at the same fill thickness, it can be seen that at lower fill thickness (2.6 m), there are very small differences in the lateral displacement. However, when the fill thickness was increased to approach the failure state, the lateral displacement of the assumed case is gradually becoming larger than the reinforced case. Thus the confining effect of the reinforcement is apparent.

**Excess pore pressures**

Variations of excess pore pressure at 2.0 m, 4.5 m (4.4 m for unreinforced case), and 7.5 m depths under embankment centerline are given in Figs. 19 (a) and (b) for unreinforced and reinforced cases, respectively. Generally, the FEM analyses simulated the field data well. When fill thickness was less than about 2.5 m, the measured data showed that during rest (consolidation) period between two load increments, there was a tendency of dissipation of excess pore pressure due to consolidation effect. While when fill thickness was more than about 2.5 m, even during rest period, field data showed that there was no reduction or an increase in excess pore pressure at 4.5 m and 7.5 m depths. When fill thickness approached 4.0 m, there was an obvious increase of excess pore pressure during rest period. However, the FEM analysis failed to simulate this phenomenon because it did not consider the creep behavior of the subsoil. It can be explained that due to the progressive development of shear strain in the subsoil, shear induced excess pore pressure increment was larger than partial dissipation effect. Also, when the subsoil approaches the failure state, the coefficient of consolidation is reduced due to reduction of soil stiffness, which will reduce the dissipation rate of excess pore pressure.

**Mobilized tensile force in the reinforcement**

Figure 20 depicts the calculated tensile forces in the geotextile. Just before embankment failure, for a stiffness $J=1600$ kN/m case, the maximum the tensile force is 36 kN/m, which is about 90% of tensile strength. The corresponding mobilized tensile strain is 2.3%. For a stiffness $J=800$ kN/m case, the mobilized maximum tensile force is 23 kN/m and about 60% of the tensile strength. The corresponding mobilized tensile strain is 3%. We consider that FEM analysis can only simulate pre-failure state, but not the failure process accompanying a large deformation. In the field, the rupture of the reinforcement was observed.
However, we believe that the rupture might be a post failure phenomenon, i.e. it was ruptured due to larger deformation of the embankment after the failure.

CONCLUSIONS

The mechanisms of reinforced embankment on soft subsoil as well as some modeling techniques are discussed and two case histories are presented.

(1) Effect of the reinforcement on embankment behavior. The mobilized tensile force in the basal reinforcement contributes to the stability of the embankment, and the confining effect of the reinforcement to embankment fill material and soft subsoil can increase the bearing capacity and reduce the lateral deformation of soft subsoil. However, the effect of the reinforcement on subsoil deformation can only become significant when soft subsoil approaches to failure.

(2) Some techniques for modeling the reinforced embankment on soft subsoil. (a) Soil/reinforcement interaction modes are pullout and direct shear. In case of grid and strip reinforcements, soil/reinforcement interface behavior is different for these two modes and it should be properly modeled in numerical analysis. (b) During construction process, the geometry of the embankment changes due to the deformation of soft subsoil. This is a kind of large deformation problem and the numerical analysis should simulate it closely. (c) Hydraulic conductivity of soft subsoil changes with the void ratio of soil and which has to be considered in simulating the consolidation process.

(3) Two case histories of reinforced embankment on soft subsoil. One of the embankments is a geogrid reinforced embankment on Muar clay deposit in Malaysian and another one is a built-to-failure geotextile reinforced embankment in China. For both cases, the limit equilibrium analysis results show that the mobilized reinforcement tensile force may only had a marginal effect on FS of the embankment. For the built-to-failure embankment in China, the analysis shows that only when the embankment approaches to failure, the reinforcement has noticeable effect on lateral displacement of the soft subsoil. The numerical analysis results also show that soil/reinforcement interaction mode is a function of the relative stiffness of reinforcement and surrounding soil. In most cases, the direct shear mode is a dominate one, and only when the reinforcement is very stiff, like steel grid, the pullout mode becomes a dominate mode.

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