Some techniques for finite element analysis of embankments on soft ground

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The accuracy of finite element results depends on the numerical models and the parameters used as well as the numerical techniques adopted. Three aspects of modelling the behavior of embankment on soft ground are discussed in this technical note: (*i*) simulating the actual construction process, (*ii*) modelling the soft ground permeability variation during the loading and consolidation process, and (*iii*) selecting proper soil–reinforcement interface properties according to the relative displacement pattern of the upper and lower interface elements placed between the soil and reinforcement in the case of a reinforced embankment. The significance of these factors on the performance of the embankment on soft ground is demonstrated by case studies.

Key words: finite element method, loading, permeability, reinforced soil.

La précision des résultats d'éléments finis dépend des modèles numériques et des paramètres utilisés de même que des techniques adoptées. Trois aspects de la modélisation du comportement de remblai sur argile molle sont discutés dans cette note technique : (*i*) la simulation de la procédure réelle de construction; (*ii*) la modélisation de la variation de la perméabilité du sol mou durant le chargement et le processus de consolidation; et (*iii*) la sélection des bonnes propriétés de l'interface sol–armature d'après le schéma de déplacement relatif des éléments supérieurs et inférieurs de l'interface placés entre le sol et l'armature dans le cas d'un remblai armé. La signification de ces facteurs sur la performance du remblai sur sol mou a été démontrée par ces études de cas.

Mots clés : méthode d'éléments finis, chargement, perméabilité, sol armé.

[Traduit par la rédaction]

1. Introduction

Finite element technique is suitable for analyzing the problem of an embankment on soft ground because the construction procedure, elastoplastic and time-dependent properties of the soft ground, and compacted fill materials can be explicitly modelled. However, the accuracy of a finite element analysis not only depends on the constitutive models and the parameters used but also on the numerical techniques adopted, such as the methods of applying the embankment load, simulating the permeability variation of the soft ground, and selecting the proper interface properties between the soil and reinforcement in the case of a reinforced embankment.

In finite element analysis, the incremental embankment load is applied by one of the following methods:

(*i*) applying a surface loading

(*ii*) increasing the gravity of all or part of the embankment elements, or

(*iii*) placing a new layer of embankment elements.

If the embankment load is treated as a surface load, the stiffness of the embankment and lateral spreading force from the embankment fill are completely ignored. Applying the incremental load by increasing the gravity of the whole embankment is much better than applying the surface loading, but still the sequence at which the load is applied to the soft ground is not closely simulated. Since soft ground is not an elastic material, the response depends on the sequence of loading. Furthermore, the stiffness of the embankment may not be modelled properly. Applying the incremental load by placing a new layer of elements is more realistic. However, in the case where the fill is of specified thickness, the node coordinates of the embankment elements above the current top surface of the embankment must be updated to account for the deformation during the construction process. Otherwise, the applied total fill thickness

will be more than the actual value because of the settlement during the construction process. Most computer programs used for analyzing the behavior of embankments on soft ground do not model construction well, such as the CRISP computer program (Britto and Gunn 1987).

For predicting the behavior of embankments on soft ground, another key point is to simulate the consolidation process. The consolidation rate is mainly influenced by the foundation soil permeability. The permeability of soft ground varies during the loading and consolidation process, and significant changes occur before and after the soil yields (Tavenas and Leroueil 1980; Tavenas et al. 1983). However, most finite element models do not consider the significant change in the soft-ground permeability before and after the soil yields (Tavenas and Leroueil 1980) and, therefore, cannot simulate the whole consolidation process well. To simulate the whole consolidation process, it is important to consider the foundation soil permeability variation during the construction and consolidation process.

The most important parameters controlling the performance of a reinforced earth structure, among others, are the soil–reinforcement interface properties. The interface properties are usually determined by direct-shear or pullout tests. However, for grid reinforcements, the different soil–reinforcement interaction mode (direct shear or pullout) yields different interface properties (Rowe and Mylleville 1988). In the case of a reinforced embankment, in the numerical modelling, different soil–reinforcement interaction properties should be used for the corresponding interaction mode.

2. Simulating the actual construction process

The actual embankment construction is carried out by placing and compacting the fill material layer by layer. In finite element analysis, if the construction settlement is not
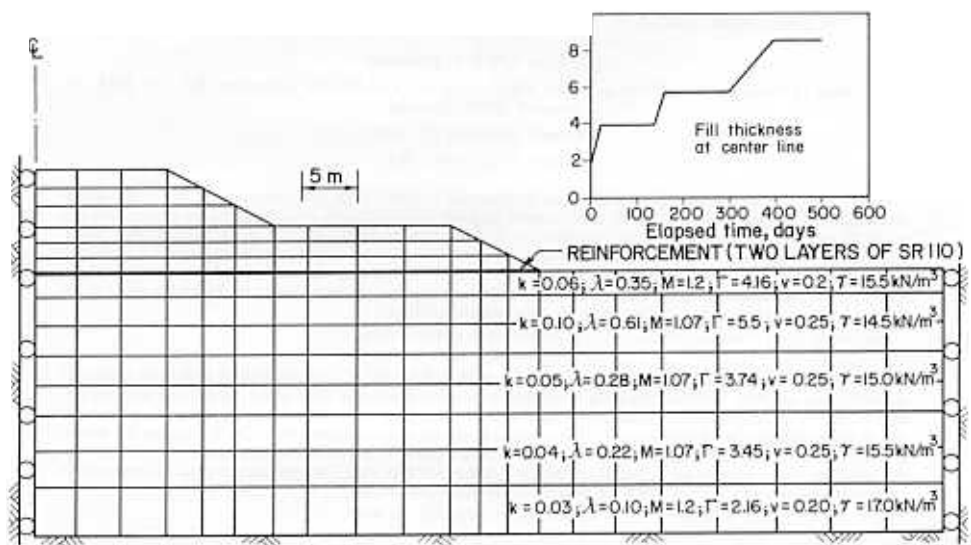
FIG. 1. Finite element mesh for Malaysian trial embankment. Γ value corresponds to $p' = 1$ kPa. K, slope of void versus ln (p') plot during rebound; λ, slope of void ratio versus ln (p') plot during loading; M, slope of failure line in (q, p') plane; Γ, specific volume in critical state line corresponding to unit mean pressure; V, Poisson's ratio; γ, unit weight; P'', effective mean stress; Γ, $P'' = 1$ kPa.

significant or if the embankment level at the end of construction is specified, applying the incremental load by assigning the gravity load of the embankment elements layer by layer is satisfactory. However, for embankments on soft ground, if the fill thickness is specified during the analysis, such as predicting the embankment settlement under a given fill thickness, the correction of the node coordinates of the elements above the current topmost surface of the embankments should be considered. The analysis of embankment on soft ground is a large deformation problem because the amount of settlement until the end of construction can vary from 20 to 100% of the total settlements, depending on the foundation permeability and loading intensity (Asaoka et al. 1992). Using the finite element method for analyzing this geotechnical problem, 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. A new approach is proposed in this technical note that allows the embankment elements above the current construction level to follow the foundation settlement, but they are not included in the finite element analysis procedure until their gravity force is applied. The method of correcting the coordinates of the embankment elements above the current top surface is based on the following assumptions:

(i) the original vertical lines are kept vertical, and the horizontal lines remain straight, and

(ii) the incremental displacements of the nodes above the current uppermost surface of the fill are linearly interpolated from the displacements of the two end nodes of the current top surface of the fill according to their horizontal coordinates.

The significance of the method for applying the embankment loading can be demonstrated by an example. One of the Malaysian trial embankments (Scheme 6/8) (Malaysian Highway Authority 1989) was analyzed using different methods of applying the embankment load. 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 berm was placed on both sides and the embankment was constructed to a final fill thickness of 8.5 m. Two layers of Tensar SR110 geogrids were laid at the base of the embankment with 0.15-m spacing in between, and vertical band drains were installed in underlying soft clay to 20-m depth in a square pattern with 2.0-m spacing (Malaysian Highway Authority 1989). The soil profile at the test site consists of a topmost 2.0-m weathered crust that is underlain by about 5 m of very soft silty clay. Below this layer lies a 10-m-thick layer of soft clay, which in turn is underlain by a 0.6-m layer of peat with high water content. Then, a thick deposit of medium dense to dense clayey silty sand is found below the peat layer. The finite element meshes and the boundary conditions used in the analysis are shown in Fig. 1. The construction history is shown by the inset in Fig. 1.

The analyses were carried out by using a computer program CRISP-AIT (Chai 1992), which was developed based on the CRISP computer program (Britto and Gunn 1987). The soil behavior models involved are the modified Cam clay model for soft foundation soil (Roscoe and Burland 1968) and a hyperbolic, nonlinear constitutive model (Duncan et al. 1980) for the embankment fill. Bar and interface elements are used to represent the reinforcement and the soil–reinforcement interface, respectively. The modified Cam clay model parameters for foundation soil are also indicated in Fig. 1. All the parameters were directly obtained from test results (Asian

TABLE 1. Hyperbolic soil model parameters for fill materials

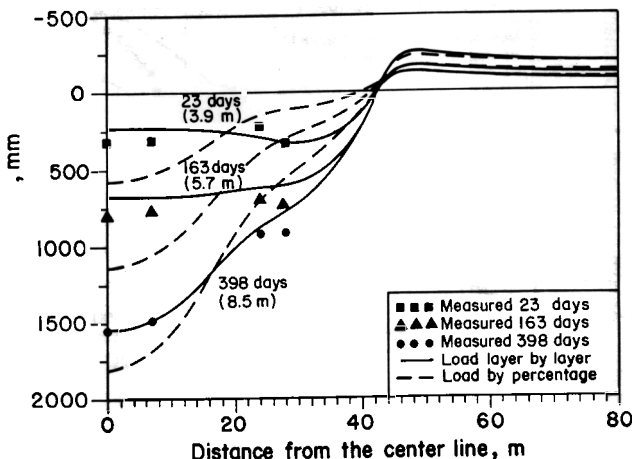|	Cohesion c (kPa)	Friction angle, ϕ (deg.)	Modulus number, k	Modulus exponent, n	Failure ratio, R_f	Bulk modulus number, k_b	Bulk modulus exponent, m	Unit weight. γ (kN/m^3)
Fill for Malaysian embankment	19	26	320	0.29	0.85	270	0.29	20.5
Fill for Asian Institute of Technology embankment	60	32.5	1078	0.24	0.96	1050	0.24	20.0

FIG. 2. Comparison of the settlement profiles for different methods of applying the embankment load.

Institute of Technology 1989). The values of permeability will be explained in the next section. The fill materials consisted of decomposed granite with the consistency of sandy clay. The hyperbolic soil model parameters adopted for the analysis are indicated in Table 1.

Two options were used to apply the embankment load:

(i) increasing the self weight of all of the embankment elements; and

(ii) the method proposed in this study.

For both methods, the total loading rate was the same and closely simulated the actual total loading rate. Figure 2 shows a comparison of settlement profiles together with field data. It can be seen that applying a percentage of the self-weight of the whole embankment yielded a larger settlement under the central point of the embankment and a lower settlement under the toe at the early stages of construction. Figure 3 shows a comparison of maximum lateral displacements at the inclinometer location, which is indicated in the figure. It can be seen that applying a percentage of the embankment self-weight as an incremental load results in much larger lateral displacements, especially at the beginning of construction. This phenomenon coincides with the settlement pattern. The soft foundation soil behaves elasto-plastically and displacement is time dependent. The deformation pattern of the soft ground not only depends on the

magnitude of the load, but also on the sequence of applying the load. Although the total loading rate is the same for both methods, when applying a percentage of the self-weight of the whole embankment, for the soil under the embankment center position, at the beginning, the loading rate is higher than the actual rate and later on becomes lower than the actual rate. For the soil under the embankment toe, the tendency is the reverse. Furthermore, as emphasized previously, applying a percentage of the self-weight of the embankment implies that the stiffness of the whole embankment elements existed at the beginning of the analysis.

Figure 2 also indicates that for the stage-constructed embankment, the end of construction settlement under the embankment center line is about 1.5 m. For this case, the finite element mesh was drawn up according to the actual fill thickness. In applying the embankment load, by placing a new layer of the embankment elements, if the coordinates of the embankment elements above the current top fill surface were not corrected, at the end of construction, the applied fill thickness would be 1.5 m more than actual at the embankment center line.

When using the finite element method to simulate the construction process of a wall or embankment with a vertical face, if the coordinate changes of the elements above the current construction level are not considered, the post-

TABLE 2. Selected permeability values for Muar clay

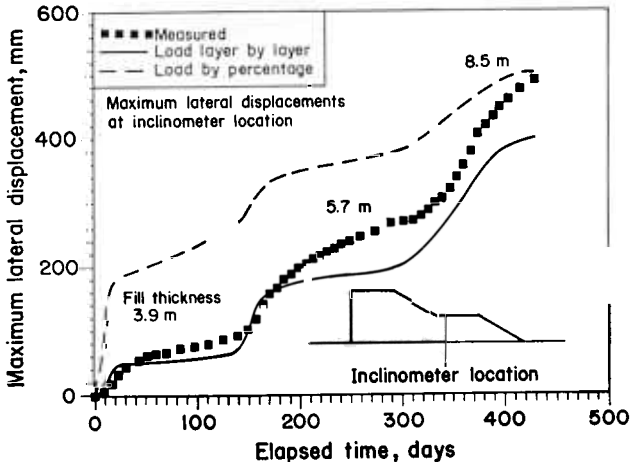| Permeability		Depth (m)			
		2–7	7–12	12–18	18–22
Horizontal, k_h (×10⁻⁸ m/s)	2.78	1.40	1.04	0.70	14 000
Vertical, k_v (×10⁻⁸ m/s)					
With drain	2.78	1.40	1.04	0.70	14 000
Without drain	1.39	0.70	0.52	0.35	7 000

FIG. 3. Comparison of the lateral displacements for different methods of applying the embankment load.

analysis treatment is necessary for obtaining the correct lateral wall face displacements (Schmertmann et al. 1989). The technique proposed in this study guarantees that the lateral displacements obtained directly from the finite element analysis are the correct values and there is no need for further correction.

3. Permeability variation of the soft ground

Permeability is one of the fundamental properties of soils, and it is one of the main factors controlling the consolidation process of the soft ground. The permeability of porous media is related to the size, shape, and distribution of their voids. Taylor (1948) proposed a relationship between permeability and the void ratio of clay as follows:

$$[1] \quad k = k_0 \cdot 10^{[1-(e_0-e)/C_k]}$$

where e_0 is the initial void ratio, e is the void ratio at the condition considered, k is the permeability, k_0 is the initial permeability, and C_k is a constant. Several natural clays were used to study the variation of permeability during consolidation through laboratory tests (Tavenas et al. 1983). It was suggested that Taylor's formula (eq. [1]) is valid, and the constant C_k can be estimated as half of the initial void ratio of the soil ($0.5e_0$).

The laboratory permeability test can be subject to error resulting from the size of sample, the temperature, and the large difference between the hydraulic gradient in the field and in the laboratory. Investigating the behavior of embank-

ments on soft ground, it was proposed that during the early stages of construction, the soft-ground foundation deforms under almost drained conditions because of the in situ overconsolidated state of the soil. When the embankment load reaches the preconsolidation pressure of the clay, the foundation yields and behaves close to undrained condition (Tavenas and Leroueil 1980). This means that the permeability of the soft ground changes significantly after the soil yields.

In this study, two methods have been used in finite element analysis to simulate the permeability-variation process of soft ground during the consolidation:

(I) The foundation permeability varied with the void ratio of soil according to Taylor's equation (eq. [1]).

(II) The soft-ground permeability is drastically changed before and after soil yielding, which is controlled by the modified Cam clay soil model. In this case, the permeability also varied with the void ratio by using [1] before and after yielding.

The Malaysian trial embankment has been used as an example to demonstrate the effects of the permeability variations on the performance of the embankment. The vertical band drains were considered as vertical seams that increased the mass permeability of the foundation soil in the vertical direction, and it is assumed that in the zone with vertical drains, the vertical permeability is twice as large as the value in the zone without vertical drains. The basic foundation permeability values are selected based on
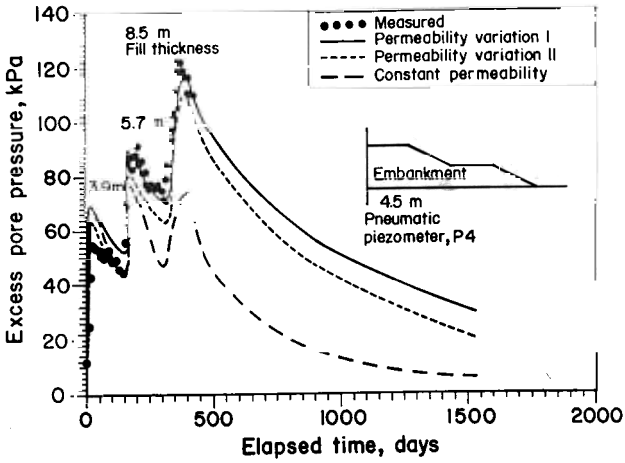
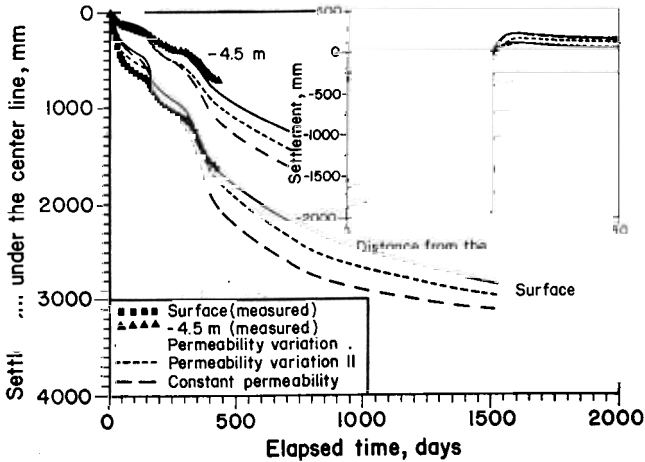FIG. 4. Comparison of excess pore pressures with different foundation permeability options.

FIG. 5. Comparison of settlement with different foundation permeability options.

the existing information (Poulos et al. 1989; Magnan 1989) as tabulated in Table 1. For the untreated zone, the vertical permeability is twice the laboratory test value (Asian Institute of Technology 1989), and the horizontal permeability is twice the corresponding vertical value. The initial values for permeability variation I are the same as the values in Table 2. For permeability variation II, the initial values of after soil yield are also the same as the values in Table 2, but the initial values of before soil yield are five times the values in Table 2. The response of the soft foundation soils has been compared with different permeability options in terms of excess pore pressures, settlements, and lateral displacements. The field data are also included.

3.1. Excess pore pressures

The typical variation of the excess pore pressure with elapsed time for a piezometer at a point 4.5 m below the ground surface and on the embankment center line is shown in Fig. 4. It can be seen that using constant permeability values cannot simulate the excess pore-pressure development and dissipation process well. At the early construction stages, the calculated values are higher than the field data, and later on they are lower than the field data. Varied permeability analyses yield much better predictions for excess pore pressure. However, permeability variation II yielded a better prediction at an early stage of construction. This confirms the necessity of modelling the permeability
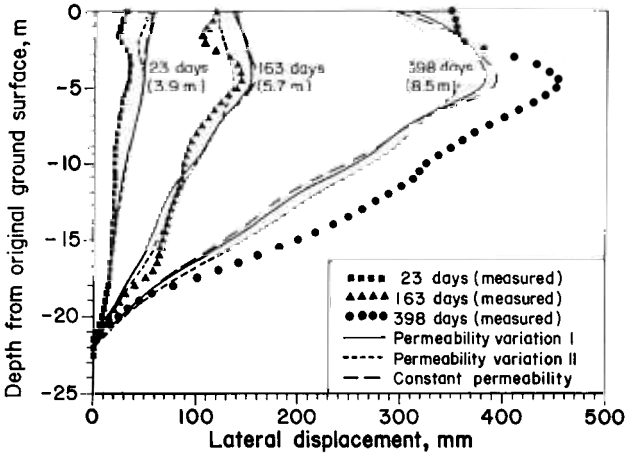
FIG. 6. Comparison of lateral-displacement profiles with different foundation permeability options.
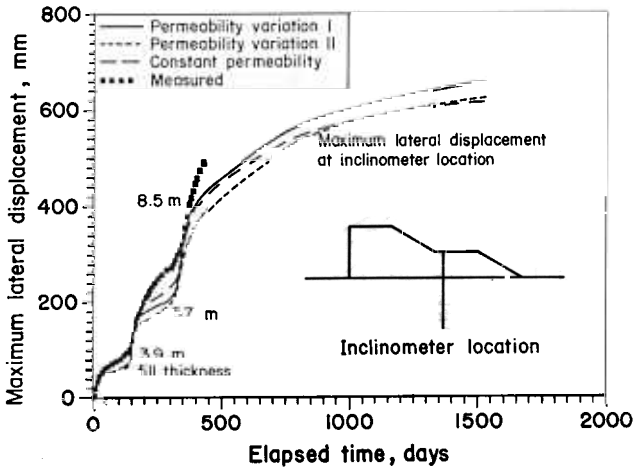

FIG. 7. Comparison of lateral displacements with different foundation permeability options.

change before and after the soil yield. It should be noted that when using the permeability-variation options, in the zones directly under the embankment, the soil permeability will reduce. However, in the zones away from the loading area, the soil permeability is unchanged. Consequently, the foundation permeability value will vary in both the vertical and horizontal directions. For permeability variation II, a strong permeability variation in both directions can be expected.

3.2. Settlements

Figure 5 shows the comparison of settlement–time curves for the points on the embankment center line. The comparison of surface settlement profiles is indicated by the inset

in Fig. 5. The settlements behave according to the excess pore pressure distribution pattern, the constant-permeability analysis underpredicted the settlements at the early stage of construction but overpredicted the settlements at the end of construction, and the varied-permeability analyses yield better results. It can be also observed that using the option of drastically changing the permeability before and after the soil yield (permeability variation II) results in larger settlement at an early stage of construction, but lower heave near the embankment toe because of the higher permeability of the soil outside of the embankment base. Although there are no measured data for heave, the information from an embankment rapidly built to failure on the same site shows that using a constant permeability the values of heave
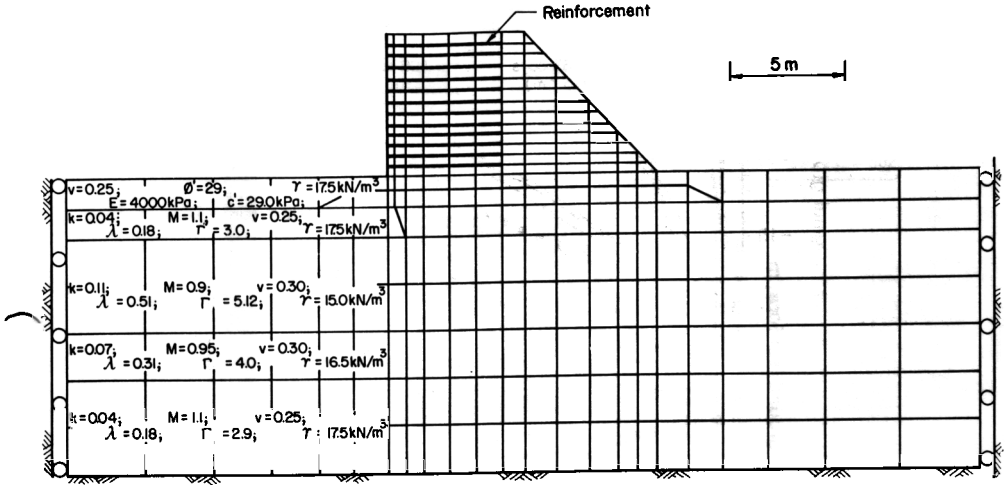
FIG. 8. Finite element mesh for Asian Institute of Technology test reinforced wall–embankment system. Γ corresponds to $p' = 1$ kPa. E, Young's modulus of reinforcement; ϕ, effective friction angle; c', effective cohesion; Γ, $P' = 1$ kPa.

were overpredicted (Brand and Premchitt 1989). Therefore, the lower predicted foundation heave resulting from varying the permeability before and after the soil yield might be closer to the actual behavior.

3.3. Lateral displacements

Figures 6 and 7 show a comparison of the lateral-displacement profiles and maximum lateral displacements at inclinometer position. It shows that at the early stage of the construction, the calculated values overestimated the lateral displacements, and at the end of construction, the calculated values slightly underestimated the lateral displacements. Using permeability variation II, slightly better predictions were obtained at the early stages of construction. From Fig. 7, it also can be seen that most of the discrepancies between the finite element results and the measured data mainly occurred during the consolidation period between the different construction stages. Many factors have been cited for poor lateral-displacement prediction, such as Poisson's ratio, anisotropy, principal stress rotation under the toe of embankment, and creep effect (e.g., Poulos 1972). In this study, most of these factors were not considered. The analysis results presented indicate that using constant foundation soil permeability values cannot simulate the soft-ground consolidation process well. In finite element analysis, it is necessary to consider the permeability variation of the soft foundation during the loading and consolidation process. The permeability of the soft natural ground changes drastically before and after the soil yields.

4. Modelling different soil–reinforcement interaction modes

In the case of a reinforced embankment (or wall), the soil–reinforcement interface property is one of the important parameters that influences the behavior of the structure. The soil–reinforcement interaction mode can either be direct shear or pullout. 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 (Rowe and Mylleville 1988; Chai 1992).

The finite element technique should be able to automatically select the proper interface properties according to the interaction modes. The technique proposed in this study 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.

Soil–reinforcement interface shear resistance can be determined by the direct-shear test and simulated by a hyperbolic shear stress – shear displacement model or other direct-shear constitutive models. However, the pullout of reinforcement, especially the grid reinforcement from the soil, is a truly three-dimensional problem and it can only be approximately modelled in a two-dimensional analysis. Pullout resistance of grid reinforcement from soil consists of skin friction resistance from grid longitudinal members and passive resistance from grid transverse members. It is assumed that the pullout resistance is uniformly distributed over the entire interface areas. Pullout interface shear stiffness consists of stiffness from skin friction resistance, k_{sf}, and stiffness from passive bearing resistance, k_{sp}, respectively. The total equivalent tangential shear stiffness, k_s, is the sum of k_{sf} and k_{sp} (Chai 1992):

$$[2] \quad k_s = k_{sf} + k_{sp}$$

The stiffness from passive bearing resistance, k_{sp}, can be determined by a hyperbolic pull-out resistance – pullout-displacement model (Chai 1992).

The example used to show the effect of the different soil/reinforcement interaction modes is a test steel grid reinforced wall–embankment system constructed on the campus of the Asian Institute of Technology (Bergado et al.

TABLE 3. Hyperbolic direct-shear interface parameters (steel grid – lateritic backfill)

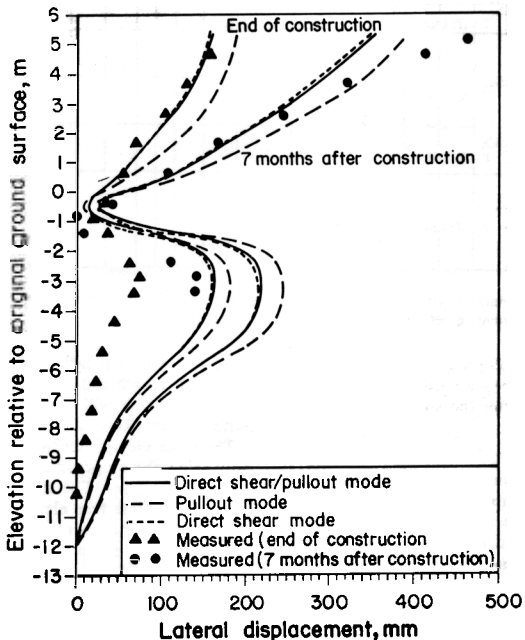|	Interface cohesion, c (kPa)	Interface friction angle, ϕ (deg.)	Shear stiffness number, k_i	Shear stiffness exponent, n_i	Failure ratio R_{fi}	Reloading shear stiffness number, k_{ir}
Soil–steel grid	60	12.5	10 600	0.72	0.85	13 000

FIG. 9. Comparison of lateral-displacement profiles with different soil–reinforcement interaction modes.

1991). The wall–embankment was constructed on a Bangkok clay deposit with base length of 26 m and fill thickness of 5.8 m. It has three sloping faces with 1:1 slope and one vertical face (wall). The subsoil profile at the site consists of the topmost 2.0 m thick layer of weathered clay overlying a soft clay layer, which extends to a depth of about 8 m below the existing ground. The soft clay layer is underlain by a stiff clay layer. The finite element mesh used for the analysis is shown in Fig. 8 together with the boundary conditions. The subsoil properties are also indicated in Fig. 8. The whole embankment was constructed within 30 days.

The hyperbolic shear stress – displacement model parameters for the direct-shear soil–reinforcement interaction mode are listed in Table 3; these were determined from direct-shear tests. The hyperbolic pullout interaction model is a bit complicated (Chai 1992). However, the most important parameters are fill properties, which are given in Table 1.

Figure 9 shows the calculated lateral displacements at the inclinometer, which was on the wall face, using the different soil–reinforcement interaction modes and their corre-

sponding properties. It can be observed that the pullout interaction mode gives the largest lateral displacements and the direct-shear interaction mode gives the smallest lateral displacements. The direct-shear interaction mode has a stronger tangent shear stiffness, and the whole reinforced mass deformed more like a rigid body. On the other hand, the pullout interaction mode has a weaker tangent stiffness, making the wall–embankment deform more easily. Figure 10 shows the influence of the different soil–reinforcement interaction modes on the foundation-deformation pattern. From the figure, it can be seen that the difference between the settlement profiles calculated by using direct-shear and pullout interaction modes is evident. Using the pullout interaction mode, the largest settlement occurred under the center line of the reinforced mass, whereas for the direct-shear interaction mode the largest settlement occurred under the toe of the wall face. In both Figs. 9 and 10, the direct-shear – pullout interaction mode means that the interface properties are selected according to the shear displacement pattern of the pair elements.
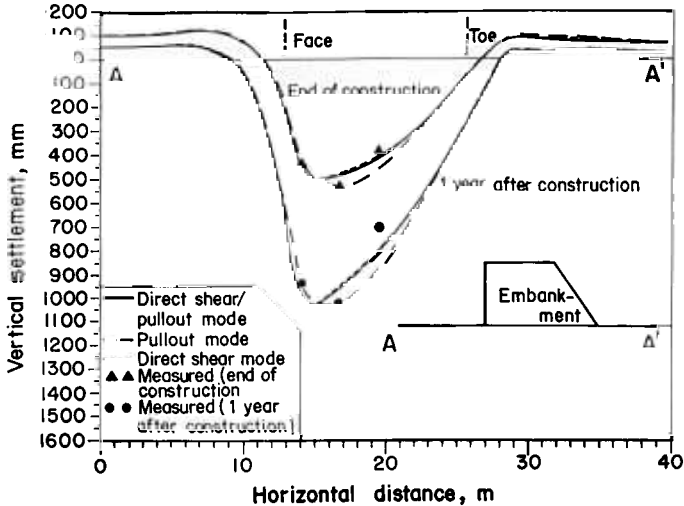
FIG. 10. Comparison of settlement patterns for different soil–reinforcement interaction modes.

5. Conclusions

The techniques presented in this study for simulating the actual construction process, considering the permeability variation of the soft ground, and modelling the different soil–reinforcement interaction modes, can improve the accuracy of the finite element predictions of the behavior of an embankment on soft ground. The following points can be made regarding each of the techniques discussed in this note.

(1) For embankments constructed on soft ground, considerable settlement may occur during the construction process. For the finite element analysis, the technique proposed in this study allows the coordinates of the embankment elements above the current construction level to follow the deformation of the existing structure and guarantees that the applied fill thickness is the actual field value.

(2) Using constant values of the foundation soil permeability cannot simulate the whole process of the foundation response during and after construction. It is necessary to consider the permeability variation of soft ground in a finite element analysis. The results of example analyses also indicate that the permeability of the natural ground drastically changed before and after the soil yielded.

(3) In the case of a reinforced embankment, in order to properly estimate the effects of the reinforcements, the soil–reinforcement interface properties should be selected according to the interface shear displacement patterns, namely, direct shear or pullout.

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