FE analysis of steel grid reinforced embankment on soft Bangkok clay

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(abstract) The behavior of a full scale test reinforced embankment on Bangkok clay has been predicted by finite element method. In the numerical modelling, two aspects have been emphasized: (1) selecting proper soil/reinforcement interface properties according to the relative displacement pattern (direct shear or pullout) and upper and lower interface elements between soil and reinforcement; and (2) simulating the actual construction process by updating the node coordinates including those of the embankment elements above the current construction level which ensures that the applied fill thickness simulate the actual field value. Finite element results were compared with the field data in terms of excess pore pressures, settlements, lateral displacements, and tension forces in the reinforcements. It was found that the acceptable results can be obtained from finite element analysis. Several influence factors, such as permeability variation of the foundation soil and compaction effects of embankment fill have also been investigated. Furthermore, some of the difficulties in finite element modelling are also discussed.

1 Introduction

The behavior of the reinforced embankments on soft ground have been analyzed by several investigators using finite element methods (e.g. Hird and Pyrah, 1990). Although different soil models, such as non-linear hyperbolic and modified Cam clay, have been used to represent the stress/strain behavior of the soils, the soil/reinforcement interface properties and actual construction process are not properly simulated. All these factors influence the ability of using finite element method to predict the behavior of reinforced embankment on soft ground.

The interface properties are usually determined by direct shear or pullout tests. However, for grid reinforcements, the different soil/reinforcement interaction modes (direct shear or pullout) yields different interface properties. In order to properly simulate the soil/reinforcement interaction behavior, it should be considered in the numerical modelling to use different properties for different interaction mode (Rowe and Mylleville, 1988).

In finite element analysis, the incremental embankment load is applied by one of the following methods: (1) by applying the surficial loading; (2) by increasing the gravity of the whole or part of embankment elements; and (3) by placing a new layer of wall or embankment elements. Obviously, the option (3) of loser to actual construction process. However, if the fill thickness is specified, the node coordinates of the embankment elements above the current construction level 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.

In this paper, the concepts of considering the different soil/reinforcement interaction modes and simulating the actual construction process are presented first. A full scale test reinforced embankment is then analyzed by the proposed finite element method. Consequently, the finite element results are compared with the field data. Finally, some of the difficulties in finite element modeling are discussed.

2 Numerical Modelling

2.1 General aspects

The reinforced embankment on soft ground system has been modelled by finite element method under plane strain condition. The face panel of reinforced wall is modelled by 3-node beam elements with axial, shear and bending stiffness. The reinforcement is modelled by 3-node bar elements. The soil/reinforcement and soil/wall face interfaces are modelled by 6-node zero thickness joint elements. The soil elements are modelled by 8 or 6-node solid elements with or without pore pressure degree of freedoms. Finally, nodal links are used at the free end of reinforcement to allow a realistic vertical stress condition (Collin, 1986).

The behavior of the soft foundation soil is controlled by modified Cam clay model (Roscoe and Burland, 1968). The linear elastic/perfect plastic model is used to model the heavily overconsolidated clay, and the yielding is controlled by Mohr-Coulomb criterion.
The backfill soil is modelled by hyperbolic constitutive model (Duncan et al., 1980). The compaction operation is modelled by bi-linear hysteretic loading/unloading model (Duncan and Seed, 1986). The consolidation process of soft ground is simulated by coupled consolidation theory (Biot, 1941).

In modelling the soil/reinforcement interface behavior, two interaction modes are considered, namely: pullout and direct shear modes. The hyperbolic shear stress/shear displacement model (Clough and Duncan, 1971) is used to represent direct shear interaction mode. Pullout resistance of grid reinforcement consists of skin friction in the longitudinal members and bearing resistance in the transverse members. The skin friction is modelled by linear elastic-perfect plastic model and the pullout bearing resistance is simulated by a hyperbolic bearing resistance model which is only valid for grid reinforcements (Chai, 1992). It is assumed that the pullout resistance is uniformly distributed over the entire interface area.

2.2 Modelling different soil/reinforcement interaction modes

Soil/reinforcement interaction mode can either be direct shear or pullout. The interface elements above and below reinforcement work as pair elements. The sign of the shear stress of the pair interface elements are used to determine either the direct shear (same sign) or pullout (different sign) soil/reinforcement interaction mode is acting mode.

2.3 Simulating the actual construction process

In order to make sure that the fill thickness applied in finite element analysis is the specified value, the embankment elements above the current construction level are allowed to follow the foundation settlement during the incremental analysis. The following assumptions are used to update the node coordinates of the embankment elements above the current construction level: (a) for the finite element mesh above the construction level, the original vertical lines are kept at vertical direction, and the horizontal lines remained straight; (b) the incremental displacements of the nodes above current construction top surface are linearly interpolated from the displacements of the two end nodes of current construction top surface according to their x-coordinates (horizontal direction).

3 TEST REINFORCED EMBANKMENT AND INPUT PARAMETERS

3.1 Test reinforced embankment

A full scale welded steel grid reinforced test embankment was constructed on the campus of Asian Institute of Technology (AIT). The original embankment was 5.8 m (19.5 feet) above the existing ground surface with about 26.0 m (87 feet) base length. It has three sloping faces with 1:1 slope and one vertical front face (wall). The length of reinforcement was 5 m and the vertical spacing between the reinforcements was 0.45 m. The wall face was formed by bent-up the steel grid. The subsoil profile at the site consists of the topmost 2.0 m thick layer of dark-brown weathered clay overlying a blackish-grey 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 and the boundary conditions are shown in Fig. 1. The bar and beam elements are indicated by darker solid line. For clarity, the interface elements are not shown in the mesh.

![Fig. 1 Finite element mesh used for AIT test reinforced embankment](image)

3.2 Model parameters

The linear elastic-perfectly plastic model parameters for topmost 1.0 m thick weathered clay layer and modified Cam clay parameters for soft and medium stiff clay layers are shown in Table 1. The parameters were determined based on actual test data (Balasubramaniam et al., 1978; Asakami, 1989). Since there is uncertainty of the permeability of the foundation soil, 3 sets of permeabilities, namely: high, middle, and low permeabilities, were determined based on existing information (Asakami, 1989; Bergado et al., 1990) and indicated also in Table 1. The top 2 m weathered clay is overconsolidated with an average overconsolidation ratio (OCR) of 5 and the underlying soil layers are slightly overconsolidated with an average OCR of 1.2.

The hyperbolic, non-linear elastic soil model parameters for compacted lateritic fill material (middle section of the embankment) are tabulated in Table 2, which were determined based on triaxial unconsolidated undrained (UU) test results (Bergado et al., 1988).

The interface hyperbolic direct shear model parameters were determined from direct shear test results of the fill material (Shivashankar, 1991). The adopted parameters were: friction angle, $\phi$, of 32.5 degrees, cohesion, $C$, of 60 kPa, shear stiffness number, $k_b$, of 10500, shear stiffness exponent, $n_b$, of 0.72, and failure ratio, $R_f$, of 0.85. The skin friction parameters between reinforcement frictional surface
Table 1. Soil parameters of Bangkok clay

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
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<tbody>
<tr>
<td>Depth, (m)</td>
<td></td>
<td>0-1</td>
<td>1-2</td>
<td>2-6</td>
<td>6-8</td>
<td>8-12</td>
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<tr>
<td>Kappa</td>
<td>(n)</td>
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<td>0.11</td>
<td>0.07</td>
<td>0.04</td>
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<tr>
<td>Lambda</td>
<td>(\lambda)</td>
<td>0.18</td>
<td>0.51</td>
<td>0.31</td>
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<td>Slope</td>
<td>(M)</td>
<td>1.1</td>
<td>0.9</td>
<td>0.95</td>
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<td>Gamma ((P=1) kPa)</td>
<td>(\Gamma)</td>
<td>3.0</td>
<td>5.12</td>
<td>4.0</td>
<td>2.9</td>
<td></td>
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<tr>
<td>Poisson's Ratio</td>
<td>(\nu)</td>
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<td>0.25</td>
<td>0.30</td>
<td>0.30</td>
<td>0.25</td>
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<td>Modulus, (kPa)</td>
<td>(E)</td>
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<tr>
<td>Friction Angle, ((^\circ))</td>
<td>(\phi)</td>
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<tr>
<td>Cohesion, (kPa)</td>
<td>(c)</td>
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<td>Unit Weight, (kN/m(^3))</td>
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<td>17.5</td>
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<tr>
<td>High</td>
<td>(k_h)</td>
<td>69.4</td>
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<td>Middle</td>
<td>(k_m)</td>
<td>34.7</td>
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</tr>
<tr>
<td>Low</td>
<td>(k_l)</td>
<td>13.9</td>
<td>13.9</td>
<td>2.1</td>
<td>2.1</td>
<td>13.9</td>
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<td>Low</td>
<td>(k_l)</td>
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<td>17.4</td>
<td>2.6</td>
<td>2.6</td>
<td>17.4</td>
</tr>
</tbody>
</table>

NOTE: High: \(k_h\) = 50 times of estimated average test value; Middle: \(k_m\) = 25 times of estimated average test value; Low: \(k_l\) = 10 times of estimated average test value.

Horizontal permeability is always 2 times of the vertical value.

Table 2. Hyperbolic soil parameter used for lateritic backfill material

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Cohesive modulus</th>
<th>Friction angle</th>
<th>Modulus number</th>
<th>Modulus exponent</th>
<th>Failure ratio</th>
<th>Bulk modulus number</th>
<th>Bulk modulus exponent</th>
<th>Unit weight, (kN/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>C, (kPa)</td>
<td>4.0</td>
<td>(k)</td>
<td>(n)</td>
<td>(R_c)</td>
<td>(k_s)</td>
<td>(m)</td>
<td>(s(kN/m^3))</td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>60</td>
<td>32.5</td>
<td>1018</td>
<td>0.24</td>
<td>0.96</td>
<td>1020</td>
<td>0.24</td>
<td>20.0</td>
</tr>
</tbody>
</table>

and lateritic soil were determined from test results of Shivashankar (1991) with adhesion of 50 kPa, and skin friction angle of 9 degrees. The spacing between grid reinforcement bearing member was 225 mm and the diameter of the bearing member was 5.4 mm. For both direct shear and pullout models, the normal stress of the interface was defined as 10\(^3\) kN/m\(^2\) for compression case and 10\(^2\) kN/m\(^2\) for tension case.

For welded wire reinforcement including the wall face, the Young’s modulus was 2.0 \cdot 10^4 kPa and the cross-sectional area of longitudinal bar per meter width was 180 mm\(^2\). For the reinforcement, the yielding stress was 6.0 \cdot 10^5. For the wall face, the shear modulus was 8.3 \cdot 10^7 kPa, and the moment of inertia of cross sectional area was 45 mm\(^3\) which was the sum of the moment of inertia of individual bars within 1.0 m width. The shear and normal stiffness for nodal link were assigned as 1.5 \cdot 10^9 kN/m and 5.0 \cdot 10^9 kN/m, respectively.

The parameters adopted for hysteretic compaction model (Duncan and Seed, 1986) were: at-rest lateral earth pressure coefficient, \(k_s\), of 0.55, friction component of limiting coefficient of at-rest lateral earth pressure, \(k_{1,4}\), of 2.21, cohesion under dynamic load, \(C_d\), of 50 kPa, at-rest lateral earth pressure coefficient for unloading and reloading, \(k_{1,5}\), of 0.15, and softening depth of 0.4 m. The peak compaction induced lateral stress used in the analysis are obtained by elastic solution (Duncan and Seed, 1986). The light compactor (Ingersollrand, D 23) was used. During construction there was a 0.3 m gap between the wall face and the soil being compacted which was later filled up during the placement of next reinforcement layer.

3.3 Finite element analysis

Totally 6 analyses have been conducted. The first three of analyses were using 3 sets of the permeabilities listed in Table 1, respectively. Analysis number 4 used middle permeability but with compaction effect. Both analyses numbers 5 and 6 were conducted with two different options of varying the permeability. For analysis number 5 (varied I), the permeability was varied with the formula proposed by Taylor (1948) as follows:

\[ k = k_s \cdot 10^{-(\varepsilon - \varepsilon_0)C_p} \]

(1)

where \(\varepsilon_0\) is the initial void ratio; \(\varepsilon\) is the void ratio at the condition under consideration; \(k\) is the permeability; \(k_s\) is the initial permeability; and \(C_p\) is constant, which is equal to 0.5 \(\varepsilon_0\) (Tavenas et al., 1983). The initial value of permeability was the middle permeability.

For analysis number 6 (varied II), the permeability variation was also controlled by Eq. 2. However, the values of permeability of the soft soil were different before and after yield with much higher value before yield. The ratio of the permeability before and after soil yields is 5:1. The yielding is controlled by modified Cam clay model. The high permeability values (Table 1) were used for before soil yield condition. A computer program named CRISP-AIT which was developed by modifying the CRISP computer program (Britto and Gunn, 1987), was used for the analyses.

4 PREDICTED RESULTS AND COMPARISON WITH FIELD DATA

Since the prediction is class C type prediction, the predicted results are presented together with the field data. The data included excess pore pressures, vertical settlements, lateral displacements, and tension forces in the reinforcements. The finite element results obtained by using middle permeability with compaction effect that are mainly used as predicted values. The results of using high and low permeabilities (Table 1) did not predict the field data well. For the sake of clarity they are omitted from the presentation. However, some of the results from varied permeability analyses and using the middle permeability without compaction effect are also included for discussion.
4.1 Excess pore pressures

Figure 2 shows the typical predicted excess pore pressure variations with different assumptions of the foundation permeability together with the field data at piezometer point 7 m below the ground surface. The agreement between predicted and measured data is fair. It can be seen that all the analyses overpredicted the excess pore pressure at the end of construction. However, the varied permeability analyses predict the excess pore pressure dissipation process slightly better.

4.2 Settlements

The predicted and measured surface settlements under the center point of reinforced mass are compared (Fig. 3). It can be seen that the predicted values have remarkable agreement with measured data. However, the varied permeability analyses yielded higher settlement rate at the early stage of construction and lower settlement rate during the consolidation process.

The settlement profile on the cross-sectional lines on the ground surface is plotted in Fig. 4. The comparisons are given for both immediately after construction and one year after construction conditions. It can be seen that the agreement between the predicted and measured data are reasonably good.

4.3 Lateral displacements

Lateral displacement is one of the most difficult items to predict. Figure 5 is the comparison of predicted and measured lateral displacement profiles for both end of construction and 7 months after construction cases. For lateral displacements in the foundation soils, the measured data up to 7 months after construction only reach down to 3 m depth because the inclinometer probe could not be inserted into the deformed casing below 3 m depth. At the end of construction, the predicted wall face lateral displacements agreed well with the measured data. However, the predicted subsoil lateral displacements are twice as large as that of measured data. At 7 months after construction, the predicted subsoil and wall face lateral displacements reasonably agreed with the measured values. However, at the top of the wall face, the predicted values are less than the measured ones and the predicted maximum subsoil lateral displacements are still larger than the field data. It also can be seen that compaction effect increased wall face lateral displacement by about 10% at end of construction even with the light compactor. This effect became less significant at one year after construction. There are two reasons for the differences obtained between the measured lateral displacements and those predicted by the finite element analyses, namely: (1) the deficiency of the analytical method (Poulos, 1972); and (2) the influence of inclinometer casing stiffness which may result in relative displacements between the soil and
the casing because it is difficult for casing to freely follow the "S" shape deformation pattern (see Fig. 5).

4.4 Tension forces in reinforcements

The predicted maximum tension forces in reinforcements are immediately after construction and one year after construction are shown in Fig. 6, together with the measured data at immediately after construction. As shown is the at-rest earth pressure line. The agreement between predicted and field data for immediately after construction is quite good. The data are presented in terms of per meter width and per reinforcement layer (0.45 m). The measured data one year after construction was not included because of too much scatter. Both the predicted and measured data showed that at the end of construction, the maximum tension forces in the reinforcements at the top half of the wall are much larger than k_s line. At the middle wall height, the data are closer to the k_s line. At the bottom of the wall, the data are much higher than k_s line again. For reinforced wall on soft ground, under the wall loading, the soft soil tends to squeeze out of the embankment base which causes large relative movement between the reinforcement and the soil. Therefore, large tension force can be developed in the reinforcements. The maximum tension forces in the reinforcements increased during the foundation soil consolidation process. The figure also shows that the compaction effect may cause the tension force in reinforcement at the top of the wall increase significantly. The location of the maximum tension force was within 1.0 m from the wall face.

5 DIFFICULTIES IN PREDICTION

5.1 Difficulties related to determine the input parameters

As mentioned previously, the correctness of finite element results depends largely on both the constitutive model and the value of model parameters used. Although most of the model parameters can be determined from high quality laboratory test results with confidence, some parameters, such as soft soil permeability, are very difficult to determine. The laboratory permeability test can be subjected to error resulting from the size of sample, temperature, and the large difference between the hydraulic gradient in the field and in the laboratory. For Bangkok clay, as reported by Bergado et al (1990), the laboratory test values underestimated the field permeability significantly. Field permeability measurements such as the piezometer method can be affected by the clogging of the filters and disturbance of the soil during the equipment installation. The values derived from back analysis of existing case histories are of great help for determining the permeability values. Another point is the variation of the permeability. In order to precisely predict the behavior of the embankment on soft ground, it is necessary to consider the variation of the permeability of soil.
5.2 Difficulties in modelling

The stress/strain behavior of the soil is influenced by several factors, such as elasto-plastic behavior, non-homogeneity, anisotropy, and structure of the soil, the stress path followed by the soil, etc. Even the sophisticated soil model, such as the modified Cam clay model cannot consider all these factors. At present, the settlement can be predicted reasonably well, but the agreement between predicted and measured lateral displacements is fair to poor. This phenomenon had been discussed by Poulos (1972). Normally, the maximum lateral displacement occurs at the vicinity of the embankment toe, where considerable principal stress rotation occurs during the embankment construction. It is difficult for soil model to consider all these factors and some of them are still not well understood, such as the effect of the principal stress rotation on the behavior of soil.

6 CONCLUSIONS

The finite element modelling techniques presented in this paper improved the ability of finite element method to predict the behavior of the reinforced earth structure on soft ground. The modelling demonstrates that the soil/reinforcement interaction properties can be properly selected according to the relative displacement pattern between soil and reinforcement (direct shear or pullout), and the construction process can be closely simulated.

Comparing the predicted and measured data indicates that the acceptable results regarding to the performance of the reinforced embankment on soft ground can be obtained by finite element analysis using the foundation permeability values based on back analyzed from case histories and by considering the large deformation phenomenon. It has been found that the predicted foundation settlements and wall face lateral displacements agreed reasonably well with the field data, and the agreement between predicted excess pore pressures, tension force in reinforcements, and foundation lateral displacements is fair.

REFERENCES

tation, Asian Institute of Technology, Bangkok, Thailand.