Finite Element Analysis for Geogrid Reinforced Embankment

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Abstract: The finite element analysis greatly enhances the understanding of the observations while the measurements give the finite element analysis substance. A full-scale test embankment reinforced with Tenax TT 201 polymer geogrids and weathered Bangkok clay as backfill material has been carried out. Finite element method has been employed to reanalyze all the measured data from the above test embankment. The agreement between the finite element results and the field data is quite good. The results indicate that the overconsolidation ratios affect much on the consolidation settlement. The consolidation has a strong effect on lateral displacement and strain of the reinforcement. The predicted results and measured data showed that the direct shear mode is suitable for large deformation and low stiffness reinforcements like Tenaxgeogrid.

1 INTRODUCTION

A geogrid reinforced full-scale test embankment was constructed at the western part of the campus of the Asian Institute of Technology (AIT) in Bangkok Thailand. The embankment has been installed with strain gages (on reinforcements) piezometers, inclinometer casings, settlement gauges and earth pressure cells.

The field data have been obtained during and after the construction. In this paper, the embankment system has been analyzed by finite element method using the computer program CRISP90. The main purpose of this analysis is to evaluate the interaction behavior between the reinforced mass and the soft foundation soil; and the capacity of the proposed model to predict the response of the reinforced embankment using polymer geogrid on soft ground.

2. DESCRIPTION OF EMBANKMENT

To see the relationship between the model and the full-scale test embankment, a brief description is given as follows. The reinforced wall/embankment system with polymer geogrid reinforcement was constructed at the western part of the campus of the Asian Institute of Technology (AIT) in Bangkok Thailand. A schematic diagram of the instrumented earth reinforced wall/embankment is shown in Fig. 1

The specified soil reinforcement was Tenax TT 201 SAMP polymer geogrids, which are manufactured with a unique extrusion technology using high quality polymers. This technology creates a product with tensile modulus, great interlock and junction strength as well as superior long term design strength and durability. The geogrid has a maximum tensile strength of 55 KN/m. The embankment has a total height of 6m with 5.7 above the existing ground surface and 0.3 m excavated below the natural ground surface. It has a vertical face in front and sloping face of 1 horizontal to 1 vertical at the back.

The embankment was constructed and reinforced with the polymer geogrid reinforcements (Tenax TT 201 SAMP) which were laid out into 7 mats. The weathered clay backfill was placed over the geogrid and compacted in equal lifts of 0.15 m to ensure efficient compaction. At the same time the following types of instruments were installed to monitor the wall/embankment: Inclinometer, pneumatic and hydraulic piezometers, surface and subsurface settlement plates, dummy piezometers and ground water observation well, pneumatic earth pressure cells.

The locations of the various types of the instrumentation are shown in Fig. 2

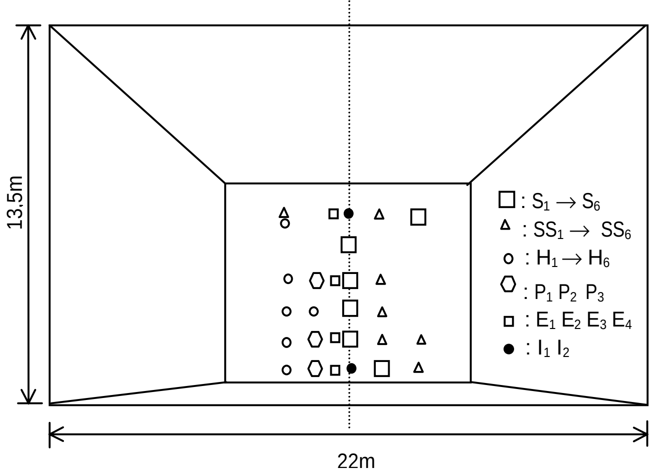

Fig. 1. Instrumentation Layout Plan of Geogrid Test Embankment

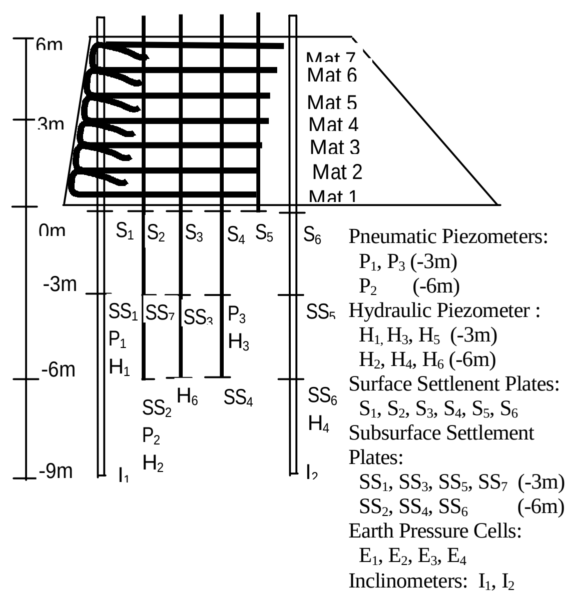

Fig. 2. Schematic Diagram of The Instrumented Geogrid Test Wall Embankment.

3. PARAMETER EVALUATION FOR CONSTITUTIVE MODELS.

3.1 Foundation soil

Based on the existing test data of Bangkok clay (Balasubramaniam et al, 1978) for the purpose of numerical modeling, the foundation soil was divided into 5 layers. The topmost heavily overconsolidated layer behaves more or less elastically. For simplicity, the linear elastic-perfectly plastic model was used for this layer. The underlying layers that are normally consolidated or slightly overconsolidated are modeled by modified cam clay model. The attempt to use the modified cam clay model for topmost heavily overconsolidated weathered clay has failed because the model produced extensive yielding on dry side of the critical state line where the validity of the model is doubtful.

Based on the existing test data 2 sets of permeability were proposed for the prediction namely: middle permeability and low permeability. The vertical permeability k_v were 25, and 10 times of the estimated representative test values, respectively. The horizontal permeability was 2 times of the corresponding vertical value for all cases (Table 1).

Table1. Soil Parameters of Bangkok Clay

Parameters	Soil layers					
	Symbol	1	2	3	4	5
	Depth, m	0-1	1-2	2-6	6-8	8-12
Kappa	κ		0.04	0.11	0.07	0.04
Lamda	λ		0.18	0.51	0.31	0.18
Slope	M		1.1	0.9	0.95	1.1
Gamma	Γ		3	5.12	4	2.9
Poisson's ratio	μ	0.25	0.25	0.3	0.3	0.25
Modulus, kPa	E	4000				
Friction angle, Degree	ϕ	29				
Cohesion, kPa	C	29				
Unit weight, kN/m^3	γ	17.5	17.5	15	16.5	17.5
Horizontal	High	69.4	69.4	10.4	10.4	69.4
Permeability, m/s	Medium	34.7	34.7	5.2	5.2	34.7
(10^{-4})	Low	13.9	13.9	2.1	2.1	13.9
Vertical	High	34.7	34.7	5.2	5.2	34.7
Permeability, m/s	Medium	17.4	17.4	2.6	2.6	17.4
(10^{-4})	Low	6.9	6.9	1	1	6.9

3.2 Wall/Embankment Structure

The linear elastic model was used to represent the embankment structure. For this analysis the embankment system was treated as a plane strain problem. The parameters for the elastic model are defined from a series triaxial unconsolidated undraind tests conducted by Bergado et al (1988a). However, this model has some limitations. Creep phenomenon that occurs during and after the construction makes lateral earth pressure coefficient higher and Poisson's ratio varied with time. This rheological behavior has much effect on the strain and tension of the reinforcement, but has not been considered in CRISP90 program.

3.3 Reinforcement

The analysis carried out in calculating the strain of the reinforcement is still based on a linear stiffness formulation with Young modulus equal to 3.10^4 kN/m^2 The cross-sectional area of longitudinal reinforcement per meter width was $4.5.10^{-4}$ m^2.

The reinforcement can be treated as a bar element. In this analysis, there's no wall face so the reinforcement members are not connected to any facing and permitted to slide relatively to each other

3.4 Soil/Reinforcement Interface

The two parameters that define the behavior of interface are shear stiffness, k_s, and normal stiffness, k_n. For k_s, two interface models were involved in modeling this interface behavior, namely: direct shear test mode and pullout test mode. The values of k_n in the range 10^5 to 10^7 were found to be suitable. The $k_n=3.2*10^4$ kN/m^2 and $k_s = 5*10^2$ were used in the analysis.

Hyperbolic shear stress/shear displacement model was used to represent the direct shear interaction mode. In this analysis the reinforcement members are not connected to any facing and permitted to slide relatively to each other; so the parameters were determined from direct shear test results of the corresponding soil. A series of direct shear test was done for Bangkok weathered clay and Tenax Samp 20 by Menil (1992) which was used to calculate the model parameters.

3.5 Reinforcement/Reinforcement Interface

The properties for the reinforcement/reinforcement interface were estimated, as there was no test information available. The properties are the same values as soil/reinforcement interface.

4. FINITE ELEMENT ANALYSIS

The finite element mesh used in this analysis is rectangular. The width of the discretized zone may be at least 3 times larger than the thickness of the ground; and the depth is limited to the underlain layer which is more than 500 to 1000 times stiffer than the overlain layer. The boundary excess pore pressure was chosen as follows: zero excess pore pressure are for the upper and lower surface of the soft clay in the case of consolidation due to external load (embankment fill). On the basis of the construction sequence, three blocks simulated the wall/embankment above the ground surface. For each block the soil weight load was applied (coupled with the consolidation) by fifty increments. After the construction, the consolidation was simulated into eight blocks. For each block of ten increments, the usual practice is to select a fixed number of time steps within a log cycle. The small incremental loading is necessary, because for soft foundation soil, the expansion of the yield locus should be within 5% at any integration point as suggested by Britto & Gunn (1987) for use of the cam clay model.

4.1 Finite Element Results And Comparison With Field Data

To evaluate the models used in finite element analysis it is necessary to compare the predicted results with measured data in the field. This comparison will draw some conclusions as well as experience, which are useful for further researches and designers. The comparison included excess pore pressures, vertical settlements, wall face and subsoil lateral displacements, strain in reinforcements and the wall/embankment base pressures.

4.2 Settlement

The locations of settlement plates are shown in Fig 2. Six surface settlement plates (S1 to S6) were placed below the reinforced soil mass on the general ground surface. Seven subsurface settlement plates were placed beneath the embankment. Four and three plates were located at 3m and 6m below the ground surface, respectively. Predicted and measured settlements at three surface points are compared in Fig. 3.

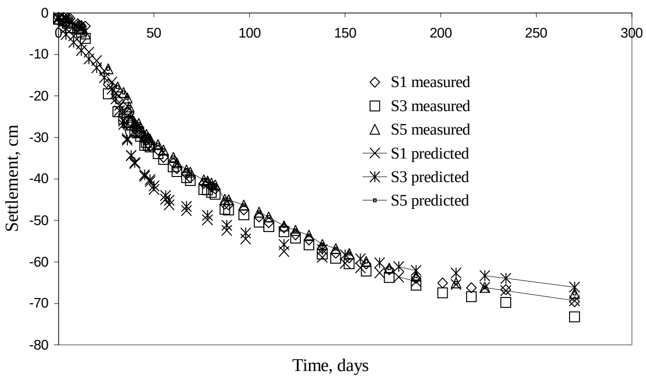

Fig. 3. Predicted and Measured Surface Settlements versus Time

The low permeability was recognized to yield good agreement between the predicted and the measured surface settlement. However, at the depth 6m, the predicted settlement fit the measured settlement only during the construction. Therefore, the important thing is to find out the relationship between the permeability and void ratio for the soil in the research and to take creep behavior into account. Fig. 4 shows the comparison between the measured settlement and predicted settlement of every point (S1 to S6) at the different times on the same plot.

The shape of settlement dish depends on the rigidity of the embankment and the type of soil. The influence of the reinforcement stiffness has affected the foundation settlement that was studied by several investigators (e.g. Hird & Kwok). The predicted and measured surface settlement profiles are plotted in Fig. 4. It is recognized that the measured settlements tend to be greater at the center than at the face or back during the time of consolidation. However, the difference is insignificant. The predicted settlements also tend to be greater at the center and smaller at the face or back. But at the end of consolidation the tendency for the settlements is on the contrary.

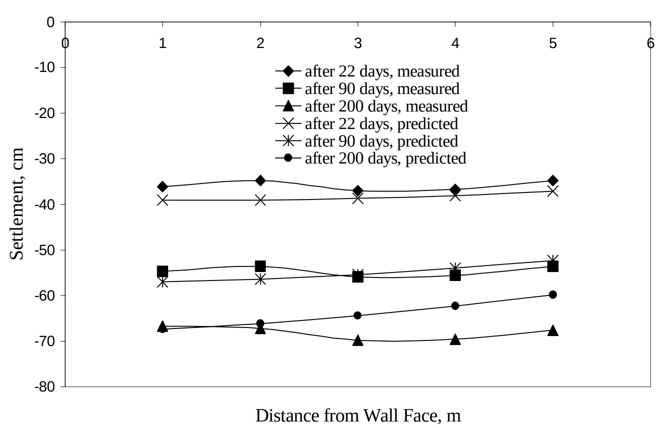

Fig. 4. Predicted and Measured Surface Settlements versus Time and Location

4.3 Lateral Displacement

The distribution of the horizontal movements at the face and at the back both in the embankment and subsoil were monitored by means of biaxial inclinometer. Because the inclinometer casing were installed before the embankment construction, some relative settlement between the soil and the casings occurred due to foundation settlement (Chai, 1993). Actually, a certain reference point is used to determine coordinates at which lateral displacements are measured; and no correction is considered for the measurement and calculation.

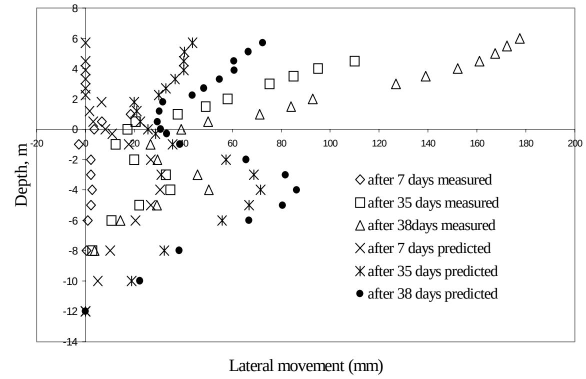

Fig. 5. Predicted and Measured Lateral Movements at Face versus Time During Construction

The lateral displacement at face during construction is shown in Fig.5. The comparison shows that the predicted lateral displacements of the embankment face are smaller than the measured ones.

The lateral displacements at the face after construction are shown in Fig. 6. The recognition is the same as above.

However, to the embankment face the agreement in displacement is good. At the longer time the the agreement is better for both the embankment face and the subsoil, especially in the case of constant OCR.

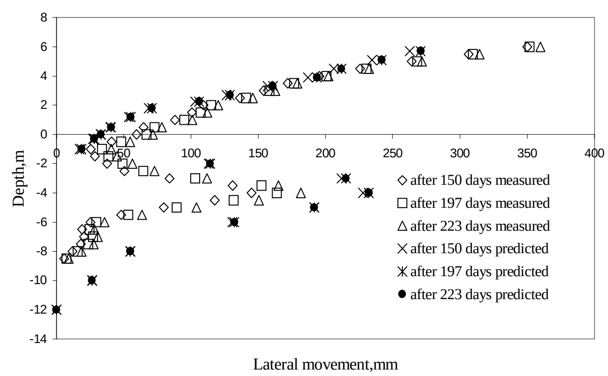

Fig. 6. Lateral Movement at Face After Construction and at Different Time After Construction

4.4 Pore Pressure:

Once parts of foundation have become normally consolidated, their consolidation characteristics are reduced by a factor 10 to 50. Maybe the above behavior can be used to explain the difference

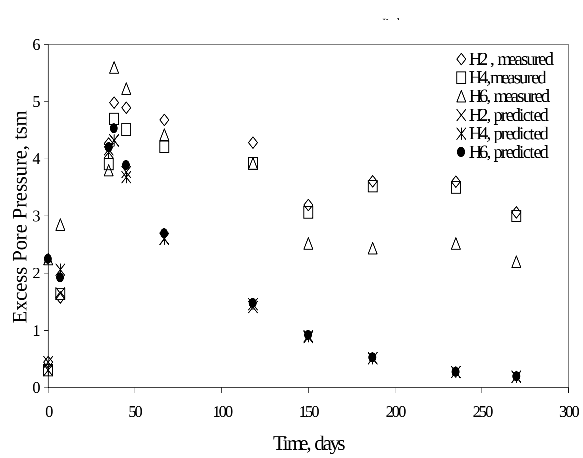

Fig. 7. Variation of Excess Pore Pressure at 6m depth by Measurement and Prediction

between the pore pressure by measurement and by prediction, (Fig 7). During the construction, the agreement in pore pressure is good. However, compared to the measured data, the predicted pore pressure decreases sharply after the construction because at this time the effective pressure exceeded the precompression pressure. That means the soil became normally consolidated. Unexpectedly, the measured pore pressure of Menil (1994) changed little after the construction. After the construction the permeability actually decreases. Therefore, the best way is to find out a relationship between the void ratio and permeability for Bangkok clay and modify the program.

4.5 Strain

Creep behavior takes much effect on strain especially for polymeric material like geogrid reinforcements. However, from the measured strains it is recognized that the maximum strain amounts to 0.5% only, that means measured strains are much smaller than the strains at yield points. It is very reasonable to model the reinforcement as linear material with limiting yield stress in the analysis. The results of the predicted strains are in good agreement with the measured one for mat 1 through mat 7 (Fig. 2) at the end of construction as shown for mat 1 and mat 2 (Fig. 8a,b). In mat 1 the predicted strain does not exist near the toe of the embankment while the measured strain does. The explanation for this difference is that when the embankment settles, the at rest earth pressure acting on mat 1 increases and balances the passive pressure and causes no tension in the part of

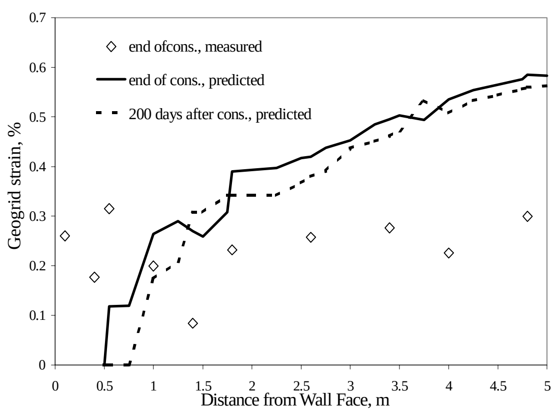

Fig. 8a. Predicted and Measured Strains of Mat 1

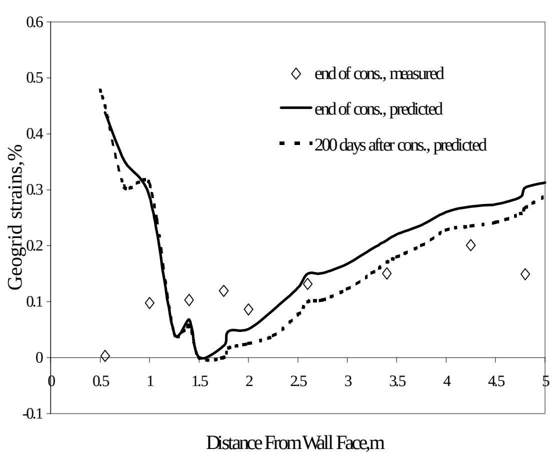

Fig. 8b. Predicted and Measured Strains of Mat 2

the reinforcement near the toe. In the field, the foundation soil at the toe has become plastic so the earth pressure can not balance the earth pressure due to the embankment soil so the strain still exists.

5 CONCLUSIONS

This research presents the results of the finite element analysis of a model with free slippage between two overlapped reinforcements and interface elements at the two free ends of reinforcements. The following conclusions can be drawn from the study:

1) The predicted and observed distribution of strains of the reinforcement showed that in this case the shear stiffness of interface could be determined by direct shear mode test.

2) In practice the elastic model can be used for the reinforcement.

3) Constant OCRs gave predicted lateral displacements which agreed with measured data but overestimated settlements. On the contrary varied OCRs gave predicted settlements which were in good agreement with measured data and predicted lateral displacements which were in fair agreement with measured ones in case of low permeability.

4) The low permeability is strongly recommended for the analysis.

5) The shear stiffness between the two reinforcements affects the strain of the reinforcements and lateral displacements of the embankment.

6) The FEM solution predicted incorrectly the subsoil displacements due to consolidation because there is a difficulty in the current critical state soil model in CRISP.

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