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3D NUMERICAL MODELING OF HEXAGONAL WIRE MESH REINFORCED EMBANKMENT ON SOFT BANGKOK CLAY

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ABSTRACT

The numerical modeling of the full scale test embankment reinforced with hexagonal wire mesh was analyzed using finite difference method under three-dimensional (3D) conditions to reflect the actual embankment dimensions. In the analysis, the 3D finite difference simulation using 5 times of vertical laboratory permeability can reasonably predict its behavior on soft foundation. In comparison with the field measurements, the predicted results from 3D analysis reasonably agreed with measured data including vertical settlement, excess pore pressures and lateral displacements. Thus, the actual embankment geometry and the selected permeability influenced the behavior of the reinforced embankment constructed on soft ground foundation. The simulated maximum tension lines in the reinforcements tend to follow the coherent gravity failure plane. The maximum tension at the bottom of the reinforced embankment occurred near midpoint portion away from the facing due to the settlement profile of the soft soil foundation.

INTRODUCTION

Generally, the usual technique to analyze and design reinforced structures can conducted using limit equilibrium method (LEM) which does not consider the interaction between the backfill and reinforcement. The LEM can normally be adopted to investigate the performance of reinforced embankment under ultimate strength condition. The plane strain finite element (2D-FEM) approach has been applied in many research studies, to investigate the behavior of reinforced embankment and interaction between soil and reinforcement material under working stress condition (e.g. Chai and Bergado, 1993; Kapurrapu and Bathurst, 1995; Bergado et al., 2001; Alfaro, 1996; Rowe and Ho, 1997; Youwai, 1999). The numerical analyses enable the investigation of various assumptions to predict the consequences of complex situations, and assess the viability of conceptual model. Although the 2D-FEM is convenient to simulate the problem, this method does not reflect the actual boundary conditions and geometry of the structure with finite boundaries. Therefore, three-dimensional finite element analysis is performed to capture the effect of boundary conditions.

Until now, the numerical simulation of the reinforced embankment under three-dimensional condition (3D) has not been thoroughly investigated. In this study, the finite difference analysis using FLAC^{3D} has therefore been adopted to simulate the behavior of three-dimensional reinforced earth structure. Subsequently, the results obtained from the finite difference analysis were compared with the measured field data.

HEXAGONAL WIRE MESH REINFORCED EMBANKMENT

The hexagonal wire mesh reinforced embankment was constructed on soft Bangkok soil. This embankment was divided into two sections along its length. Each section was constructed with different types of hexagonal wire mesh, namely: zinc-coated and PVC-coated. The facing unit inclined at 10 degrees with respect to vertical was constructed using gabion form. The backfill was compacted in 0.167 m lifts to a total thickness of 0.50 m with the maximum dry density of 95% of the standard Proctor compaction using the combination of roller and manual compaction. The instrumentation such as settlement plates, piezometers, and inclinometer were installed. After completion, the hexagonal wire mesh reinforced embankment was 6.0 m high, 6.0 m long, 6.0 m wide at the top and the base width of embankment was 18 m. Additional 1 m high surcharge was then added on the top of the existing embankment using one thousand plastic sand bags. Each bag was filled with 40 kg of sand and laid in one cubic meter of gabion cage. Thus, the pressure of additional surcharges load approximately is equal to 16.7 kN/m^2. The instrumentation program and cross section for this test embankment were illustrated in Fig. 1.

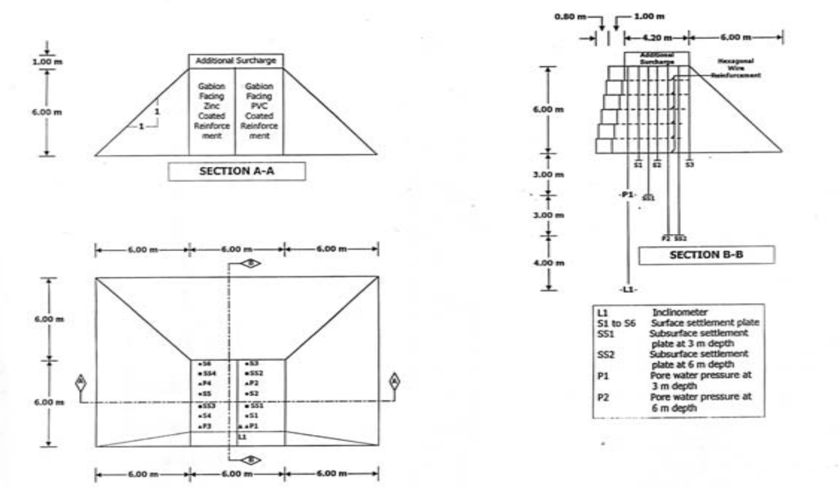

Figure 1. Schematic plan view layout of field instrumentation (Voottipruex, 2000)

3D FINITE DIFFERENCE ANALYSIS OF HEXAGONAL WIRE MESH REINFORCED EMBANKMENT

Finite difference mesh set-up with interface

The numerical modeling by $FLAC^{3D}$ has the advantages in obtaining more realistic staged construction and finite boundary conditions (Itasca, 2005). Owing to the symmetry of the embankment structure, only the half section of the reinforced embankment and the soil foundation were applied to reduce the numbers of the degree of freedom, which caused the time consuming in the calculation steps The soil and gabion structure materials were consisted of brick shaped elements (eight Nodes and six sides). The material properties are summarised in Table 1. The dimensions of the soil foundation were equal to 42 m long, 24 m wide and 12 m. deep. The similar soil profiles were utilized same as the previous case mentioned in 2D analysis. The dimensions of embankment facing were modeled to be equal to 3.0 m high, 6.0 m long, 6.0 m wide at the top and 12 m long 9 m wide at the base with the inclined side slope of 45 degrees.. The dimensions of gabion structure were 3.0 m wide, 1.0 m long and 6.0 m high. The uniform vertical spacing of hexagonal wire mesh reinforcement in reinforced embankment was 0.5 m. The reinforcements simulated by the shell structural elements in the embankment were 4 m long, 3 m wide and 0.003 m thick. The interface elements were attached to provide the sliding plane for the reinforcement and surrounding soil. The interaction coefficient was also set to be 0.9 same as the 2D case (see Fig. 2).

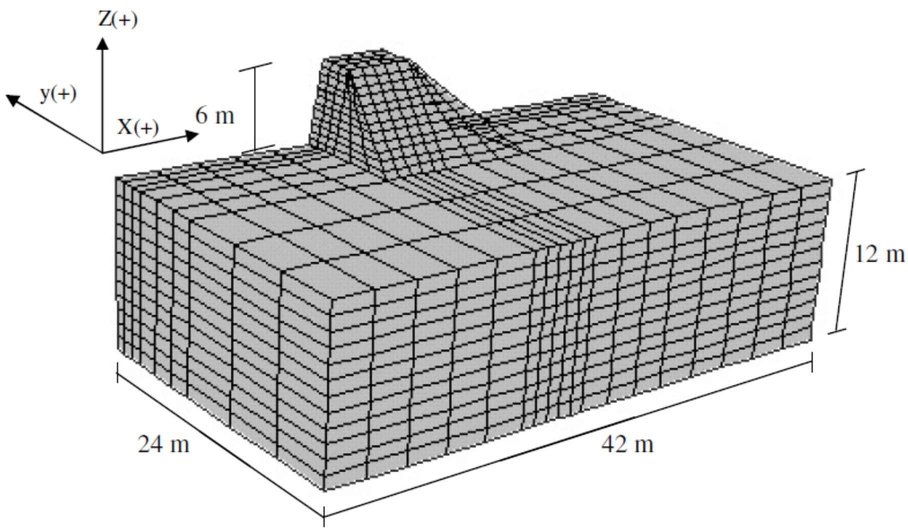

Figure 2. 3D Finite Difference Mesh of Hexagonal Wire Mesh Reinforced Wall

Table 1 Selected parameters for hexagonal wire mesh reinforced embankment in FDM analysis

Parameter	Soil Layer						Gabion	Backfill
	Symbol	1	2	3	4	5		
	Depth, (m)	0-1	1-2	2-6	6-8	8-12		
Model		MC	MCC				MC	
Slope of Elastic Swelling line	κ		0.04	0.11	0.07	0.04		
Slope of Normal Consolation Line	λ		0.18	0.51	0.31	0.18		
Frictional Constant	M		1.1	0.9	0.95	1.1		
Specific Volume at Reference Pressure (1Pa)	ν_λ		4.256	8.879	5.996	4.168		
Reference Pressure (1Pa)	p1		1	1	1	1		
Poisson's Ratio	ν	0.25	0.25	0.3	0.3	0.25	0.33	0.30
Maximum Elastic Bulk Modulus ($x10^7$ Pa)	κ_{max}		12.5	2.88	4.86	9.6		
Preconsolidation Pressure ($x10^4$ Pa)	p_{c0}		13.0	6.00	8.50	10.2		
Elastic Bulk Modulus ($x10^6$ Pa)	K	1.6					5.88	5.83
Elastic Shear Modulus ($x10^6$ Pa)	G	4000					2.26	2.69
Friction Angle, (degree)	ϕ'	29					45	25
Cohesion, ($x10^3$ Pa)	c’	29					20	10
Total density (kg/m^3)	ρ_τ	1750	1750	1500	1650	1750	1800	1800
Dry density (kg/m^3)	ρ_δ	1750	1750	803	1050	1226	1800	1800
Porosity	n	0.545	0.545	0.697	0.600	0.524		
Permeability ($x10^{-12}$ m/s)	$25.0k_v$	17.4	2.6	2.6	2.6	17.4		

Note: (1) MC = Elastic Perfectly Plastic Mohr-Coulomb Model
(2) MCC = Modified Cam-Clay Model

The boundary conditions along both sides of the soil foundation were assigned by the fixed displacement boundary in x-direction while the other displacement boundary in z direction was allowed. The horizontal and vertical fixed displacements were attached to the bottom of the soil foundation. The boundary conditions also were shown in Fig 2. The distribution of initial hydrostatic pore-pressure was designated below the ground water table at the depth of 2 m below the ground surface. The closed flow boundaries are attached both sides of the foundation and the bottom of the foundation to ensure that no flow across the boundary. The axial stiffness, EA, of 900kN/m was adopted in the modeling of the reinforcement as average value for reinforcement (Bergado et al., 2000). The interface coefficient of 0.9 was adopted for all analysis cases.

The sequences of the earth reinforced embankment construction were also considered by dividing the height of embankment into 12 stages. Within 60 days, the full height of reinforced soil embankment was raised up to 6 m high. The coupled analysis, undrained and consolidation analyses, was also considered. After 405 days, the applying surcharge pressure of 16.7 kN/m^2 was added.

RESULTS AND DISCUSSIONS

The effects of the finite size boundary of reinforced embankment under three-dimensional condition were conducted using FDM program, FLAC^{3D}. The coupled analysis using 25 and 5 times vertical laboratory permeabilities, was considered for all cases. The comparisons between measured field data and predicted results e.g. vertical settlements, excess pore pressures, and lateral displacements are mainly discussed on the following sections.

Surface settlements

The comparisons between the predicted results and the measured field data from subsurface settlement plates at the depth of 3 and 6 m below the ground surface are shown in Figs. 3 and 4, respectively. From FDM analyses under three-dimensional analysis with 25 times of vertical laboratory permeability (25k_v), the computed subsurface settlements at 3 m depth overestimated when compared with the field results. Using 5 times of vertical laboratory permeability (5k_v) as back-calculated permeability, the predicted subsurface settlements agreed with the field measurements.

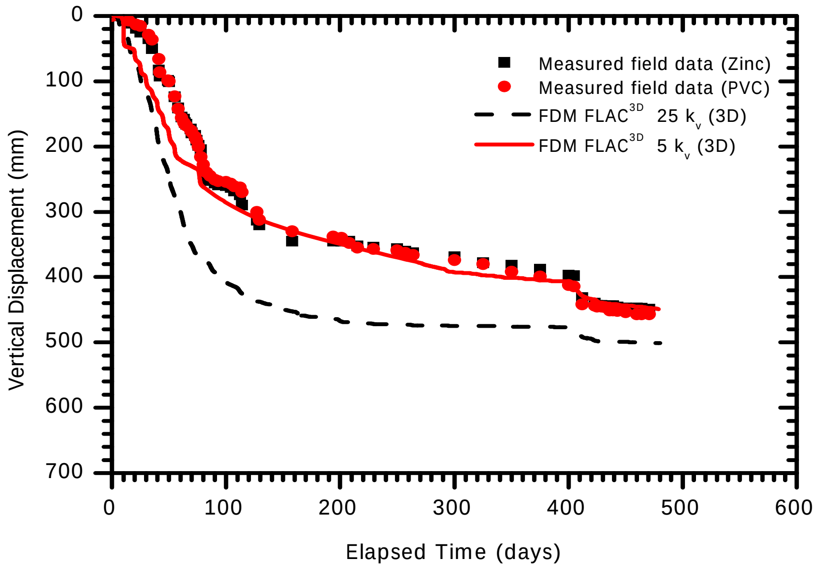

Figure 3. Comparison of predicted surface settlement of hexagonal wire mesh reinforced soil Embankment under 3D condition at plate S1 (0.45 m deep at front)

For the subsurface settlement plates at 6 m depth (see Fig. 4), For the three-dimensional stresses field condition with both 25 times and 5 times of vertical laboratory permeability ($25k_v$ and $5k_v$), the predicted results were less than the measured field data. It is noted that the piezometric draw down occur starting from the depth of 6 m from the ground surface due to the excessive groundwater pumping that caused ground subsidence. However, during the construction period utilizing 5 times of vertical laboratory permeability ($5k_v$), the predicted subsurface settlement from the numerical simulation under 3D condition is consistent with the field measurements.

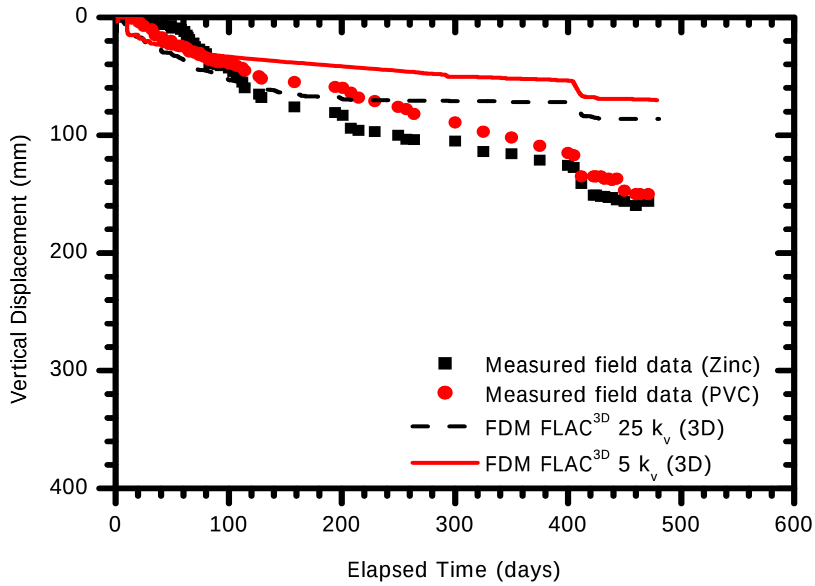

Figure 4. Comparison of predicted subsurface settlement of hexagonal wire mesh reinforced soil embankment under 3D condition at settlement plate SS2 (6 m deep at middle)

Excess pore pressures

The comparisons between predicted and measured results of excess pore pressure at the depth of 3 m below the ground surface are shown in Fig 5. The maximum of predicted excess pore pressures occurred at 60 days and gradually decreased with the time due to consolidation. However, after the additional surcharge was added at 405 days, the excess pore pressures rise can be observed and then started decreasing. It can be seen that, under 3D condition, the maximum excess pore pressure by using 5 times of vertical laboratory permeability ($5k_v$) was higher than 25 times of vertical laboratory permeability. ($25k_v$) Owing to the applied higher permeability value in the numerical analyses, the excess pore pressures can dissipate faster than the lower permeability case during the embankment construction. The prediction of the excess pore pressures after the end of construction could have some different results compared with the filed measurements because of the loading conditions and the permeability of the foundation soils. There are also many uncontrollable factors which could not be simulated by the numerical analyses, such as piezometric draw down due to excessive groundwater pumping (reduction of the piezometric level), the precipitation during rainy season (increase the piezometric level).

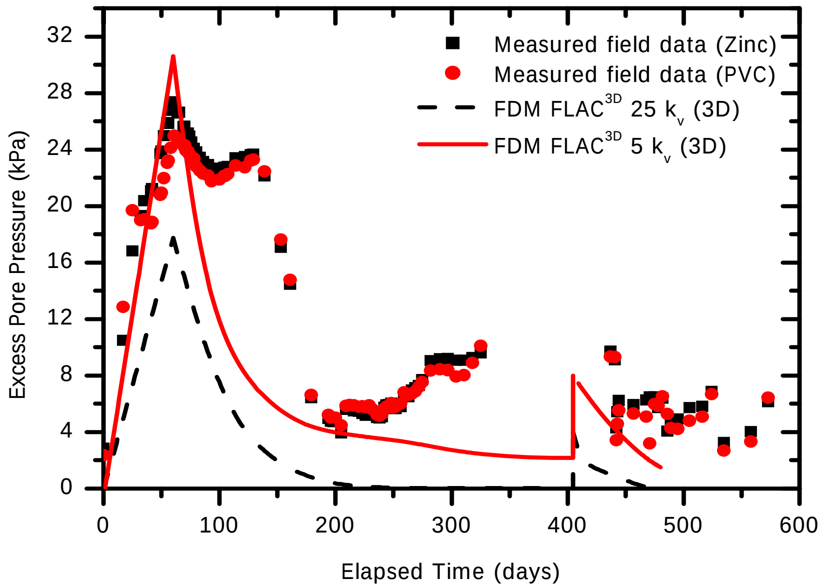

Figure 5. Comparison of predicted excess pore pressure of hexagonal wire mesh reinforced soil embankment under 3D condition (3 m Deep at middle**)**

Lateral displacements

The comparisons of predicted and measured lateral displacements are given in Fig. 6 at 493 days. The predicted lateral movement using 25 times of laboratory permeability (25 k_v) was slightly more than using 5 times of laboratory permeability (5k_v). In 3D simulation using 5 times of vertical laboratory permeability (5k_v) for the foundation soils, the predicted results agreed well with the field data. However, for the reinforced embankment zone, the lateral displacements underestimated because of many factors such as, creep behavior and anisotropic condition, which were not included in the analysis.

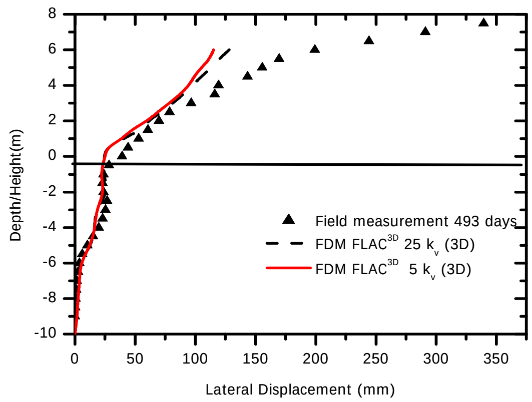

Figure 6. Comparison of measured and predicted lateral displacement profiles of hexagonal wire mesh reinforced soil embankment under 3D condition

Tension forces in the hexagonal wire mesh reinforcement

The distributions of tension forces along hexagonal wire reinforcements in the reinforced embankment obtained from FDM under 3D condition using 5 times of vertical laboratory permeability ($5k_v$) zone are shown in Fig. 6. The predicted results showed logical results in which the tension forces of 240 days are larger than that of 60 days. For Mat no. 1 (the 1st layer of reinforcement close to the ground surface), the maximum tensile forces were located at 3 to 4 m from the facing unit because the compression of the soft foundation. This is quite different mechanism comparing with the reinforced embankment constructed on the rigid foundation. As reported by Rowe and Ho (1998), the maximum tensile forces on the reinforcement are located nearly to the wall face in the rigid foundation case. For Mat no. 2 and 3, the maximum tensile force are located at the front of the embankment because of the rotations and the horizontal movements of embankment. The maximum tension line from numerical analyses tends to agree with the coherent gravity failure plane as illustrated in Fig 7.

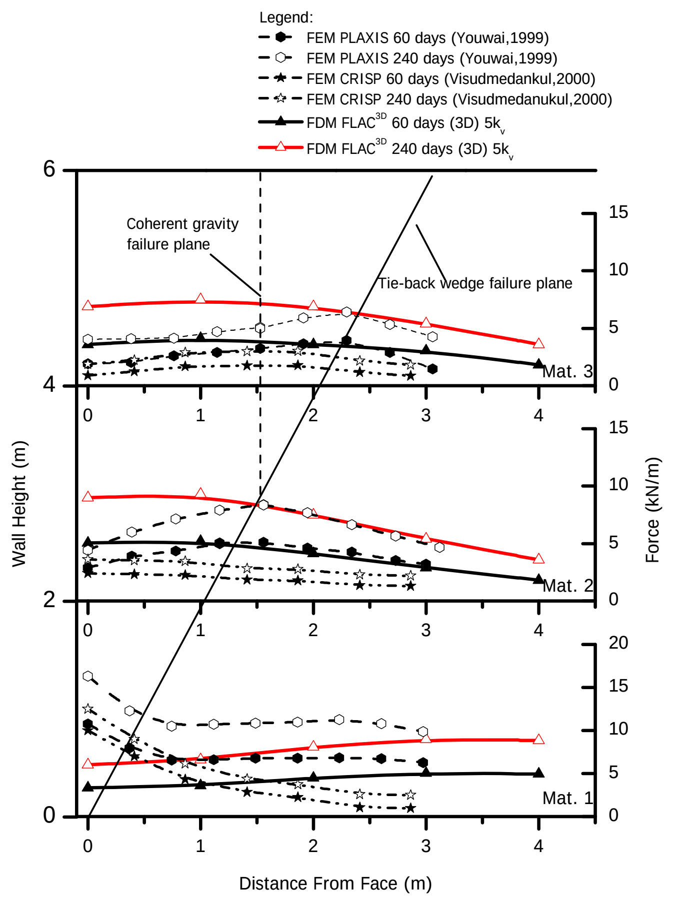

Fig. 7 Comparison of predicted forces in hexagonal wire mesh reinforcement with previous studies

CONCLUSIONS

The numerical simulations based on finite difference method (FDM) were carried out to investigate the actual behavior of full-scale hexagonal wire mesh reinforced embankment on soft ground foundation. FLAC^{3D} program was adopted to analyze reinforced embankment under 3D field condition. Based on the numerical results and the comparison with the field measurements, the following conclusions can be drawn;

- The numerical simulations of hexagonal wire mesh reinforced embankment were conducted under 3D condition with 5 and 25 times measured foundation soil permeability ($5k_v$, $25k_v$). The 3D finite different simulation using 5 times of vertical laboratory permeability ($5k_v$) reasonably agreed with the measured field data.
- The simulated maximum tension line in the reinforcements tends to follow the coherent gravity failure plane. The maximum tension at the bottom of the reinforced embankment occurred near midpoint portion away from the facing due to the settlement of the soft soil foundation.
- The tensions in the reinforcement increase from K_a-line to K_0-line with increasing vertical settlements and the lateral displacements of the wall.
- The factors affecting on the numerical simulation were the stages of the construction, the boundary conditions in the field, the variation of soil permeability in the soft soil foundation, and the selection of appropriate model as well as the properties of the interface between the backfill soil and the reinforcement material corresponding to their interaction mechanism.

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