NUMERICAL ANALYSIS OF EMBANKMENT ON SUBSIDING GROUND IMPROVED BY VERTICAL DRAINS AND GRANULAR PILES

ANALYSE NUMERIQUE D'UNE LEVEE DE TERRE SUR SOLEN VOIE D'AFFAISEMENT A TENUE AMELIOREE PAR CANALISATIONS VERTICALES D'EVACUATION ET PIEUX GRANULAIRES

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SYNOPSIS In this paper, the use of Finite Element Method (FEM) based on Revised Cam clay model for 2-D consolidation analysis of two test embankments constructed on soft Bangkok clay improved by granular piles and prefabricated vertical drains are presented. In 2-D plane strain model, the vertical drains and granular piles were converted into continuous walls. The 3-D radial flow in the improved grounds is converted into 2-D laminar flow by introducing a new permeability conversion. The results of FEM were compared with the other methods and observed data. Excellent agreements between the observed and predicted time-settlements were obtained. Using the ratio of horizontal to smeared zone permeability (k_h/k_s) is 10, the ratios of d_s/d_w = 2.5 and d_s/d_p = 2 are very good fitting parameters, where d_s, d_w and d_p are the diameters of smear zone, equivalent drain and granular pile, respectively. The stress concentration ratio of granular piles varied in a wide range from 2.4 to 8 depending on the degree of consolidation and location within the piles. The subsidence effects on improved grounds have also been evaluated separately. The results shown that to penetrate the drains or granular piles down to the stiff clay layer where the pore pressure dropped considerably, additional subsidence may occur.

INTRODUCTION

In Southeast Asia, soft clays are fairly widespread, and some of these deposits exist extensively in the vicinity of many big cities. A number of ground improvement techniques for deposits of soft clay have been developed over the past years, in which, vertical drains and granular piles improvements have gained wide interest and recognition. A review of research results on soft Bangkok clay improved by granular piles and vertical drains can be found in Bergado et al (1991, 1992). In an attempt to study the capability of improving soft Bangkok clay by using granular piles and prefabricated vertical drains, two full-scale test embankments were constructed at the Campus of the Asian Institute of Technology (AIT) in December, 1986. The soft, subsiding ground was improved by compacted granular piles in one test embankment, and in the other, by prefabricated Mebra drains. Both granular piles and vertical drains were arranged in triangular pattern at 1.5 m spacing. Analysis of settlements, slope stability, and other behaviors of these test embankments using analytical methods have been made by several researchers. This study aims to proposed a two-dimensional (2-D) model of granular piles and vertical drains so that a finite element program using elasto-plastic, stress-strain relationship, namely CON2D (Duncan et al, 1981), can be applied for consolidation analysis of these test embankments. The results of the numerical solution is subsequently compared with the observed data and the analytical results obtained by previous researchers. Most of the data presented in this paper were derived from the work of Long (1992).

SUBSOIL CONDITIONS AND AIT TEST EMBANKMENTS

The subsoil profile of the site together with index properties are shown in Fig. 1. The AIT campus, where the test site is located, is situated on a flat, deltaic marine deposit called Chao Phraya Plain in Central Thailand. The ground subsidence of Chao Phraya Plain is attributed to the reduction in piezometric pressure, and subsequent consolidation of the clay layers due to excessive groundwater pumping for water supply through deep wells. In AIT campus, from observations of Taesiri (1976) and Khaw (1986), the surface of groundwater table dropped about 1 m and the pore pressure drawdown in the first sand layer was about 4.5 m at 18 m depth during this 10-year period and has now stabilized (Bergado et al., 1986).

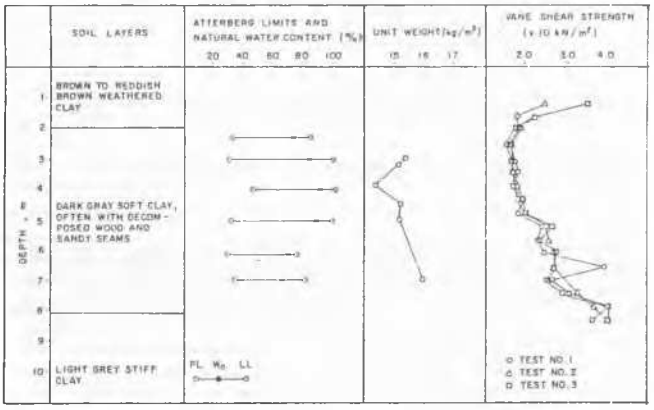

Fig. 1 Soil profile and soil properties

The cross-section of embankment on granular piles (EGP) and location of instrumentation are shown in Fig. 2. The embankment was constructed in two stages, initially 2.4 m high, and was later raised up to a height of 4 m after 345 days. The granular piles (GP) were arranged in triangular pattern with spacing of 1.5 m, each pile having a diameter of 0.3 m and a length of 8 m. The granular material used to construct the granular piles consists of whitish-gray, crushed limestones and was poorly-graded with maximum size of 20 mm. The cased borehole method (Bergado et al., 1991) was employed in constructing the granular piles. Unit weight after construction varied from 17 to 18.1 kN/m³. Direct shear test results gave values of friction angles ranging from 39 to 45 degrees.

The cross-section of embankment on prefabricated vertical drains (EVD) is shown in Fig. 2. The vertical drains (VD) were installed in triangular pattern at 1.5 spacing by means of a special mandrel down to 8 m depth. The size of the mandrel was minimized to reduce the smear effect. The rectangular-shaped mandrel had the inner dimensions of 2.8 x 13.3 cm, and the outer one of 4.5 x 15 cm, just enough to contain the 0.3 x 9.5 cm prefabricated Mebra drain. A drainage pad of 0.3 m thick consisting of clean sand was laid between the embankment and the top of drains.

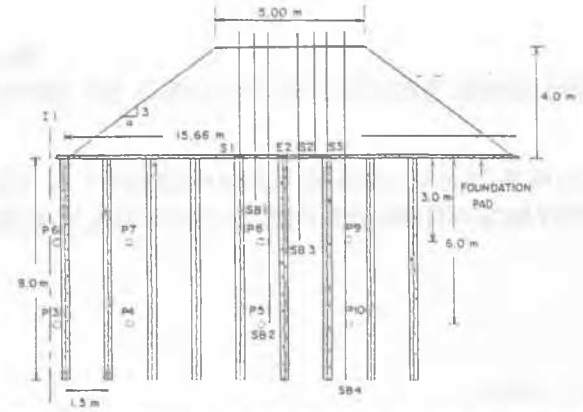

Fig. 2 Cross section of test embankment on granular piles (EGP)

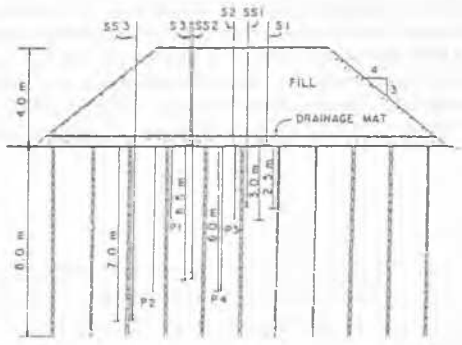

Fig. 3 Cross section of test embankment on vertical drains (EVD)

2-D MODELLING OF GP and VD-IMPROVED GROUNDS

Analytical methods developed so far for consolidation analysis employ the concept of a "unit cell", wherein, a circular domain of influence of a single drain or granular pile is analyzed. The assumption is that each unit cell works independently and all strains within the soil mass occur in the vertical direction only (Barron, 1948; Hansbo, 1981). This assumption is strictly valid only for an infinitely wide loaded area and, thus, face other serious restrictions concerning boundary conditions, taking into account plastic flow in the soil, non-homogenous behavior of subsoils etc.. In fact, the assumption of no lateral displacements is not reliable particularly for the case of embankment on soft ground, wherein lateral deformation can occur being one of the important signals indicating the instability of the ground. Therefore, it seems more reasonable that the GP or VD-ground system should be analyzed as a whole using numerical treatment. The GP or VD-improved ground is a 3-D problem. However, finite element calculations with discrete modelling of granular piles or vertical drains in 3-D turn out to be very complicated for routine analysis. Thus, 2-D analysis seems more practical.

An "equivalent material" model of improved ground was introduced by Swcheiger & Pande (1988) for 2-D plane strain analysis of embankment on soft clay stabilized with stone columns. In this model, the improved zone was treated as an "equivalent material" with the "equivalent parameters" of strength and stiffness. The flow parameters were not considered, then only undrained or fully drained cases could be analyzed. Finite element method using elasto-plastic Cam clay model has been used by Asaoka et al (1991) for analysis of undrained failure of embankment improved by sand compaction piles, wherein, the sand compaction piles were transformed into a number of sand walls. For consolidation analysis, it is necessary to convert the spatial flow in actual case into the laminar one in 2-D plane strain model. One such converted permeability was introduced by Shinsha et al (1982) for vertical sand drains. Shinsha (1982) based on his assumption that the required time for 50 % degree of consolidation in both schemes (in actual case and in 2-D model) are equal. Then, the simple expression was obtained as shown below:

$$k_m / k = (L/D_e)^2 \, T_{h50} / T_{r50} \qquad (1)$$

where L is half the distance between two sand walls in 2-D model, T_{h50} = 0.197 is dimensionless time factor at 50 % consolidation of laminar flow and T_{r50} is corresponding radial flow in actual case, k_m and k are horizontal permeability coefficients of 2-D model and actual case, respectively. Comparisons of 3-D with 2-D analysis using Shinsha et al (1982) method, Cheung et al (1991) obtained the higher pore pressures in the 2-D model and concluded that this method should be used with cautions.

In this paper, for 2-D consolidation analysis of embankment on vertical drains and granular piles, the VD and GP are transformed into continuous walls with the same spacing as that of the actual case. The following determinations of model parameters are in general form for both vertical drains and granular piles. If a_s is the ratio of granular piles area to the total improved area and the same value of a_s is used for both schemes, then:

$$a_s = tS/DS = t/D \quad \text{or} \quad t = a_s D \qquad (2)$$

where t is the thickness of the walls in 2-D model while D and S are the row spacing and pile spacing of actual case, respectively.

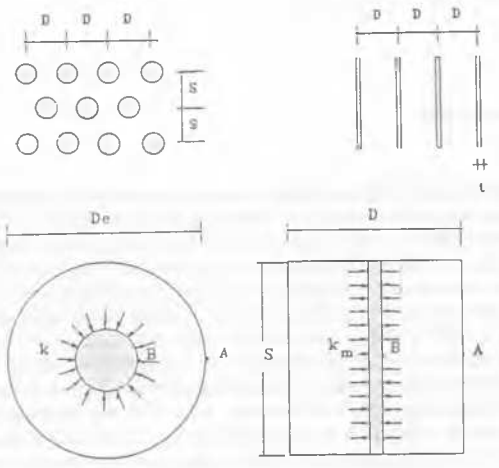

Fig. 4 Granular piles in actual case and in 2-D plane strain model

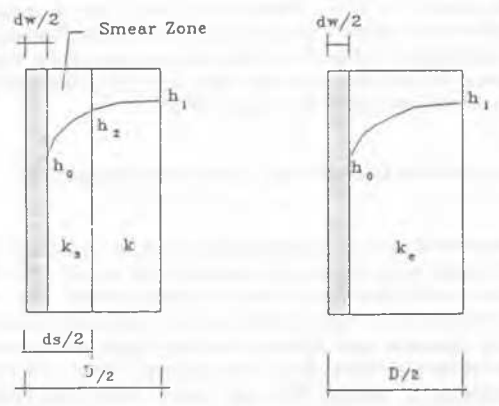

Fig. 5 Equivalent permeability for smear effect

The converted permeability including smear effect is introduced, based on condition of the equal discharge rate in both schemes, with the assumption that the coefficient of permeability is independent on state of seepage flow. In this approach, the permeability of the soil between drain walls in 2-D model is adjusted to make the same discharge between the actual case and 2-D model. The vertical flow in both schemes can be assumed to be the same. Then, only the horizontal flow needs to be converted. Taking the same head boundaries A and B in Fig. 4, then from the condition of the same discharge between both schemes in steady state flows one can get Eq. (3), wherein, n = D/d, $\alpha = D_e/D$; S = D and α = 1.05 for square pattern; S = 0.866D and α = 1.13 for triangle pattern.

$$k_m = \frac{\pi\,(1-a_s)\,D}{2S\log_e(\alpha n)}\,k \qquad (3)$$

If the smear effect is taken into account, the value of k in Eq. (3) can be replaced by k_e, where k_e is the equivalent permeability including smear effects. The value of k_e can be obtained from the condition of the same discharge rate of steady state flows with the same head boundaries (Fig. 5) as given below:

$$k_e = \frac{k\log_e(\alpha n)}{\log_e\left(\frac{\alpha D}{d_s}\right) + R_s\log_e\left(\frac{d_s}{d_w}\right)} \qquad (4)$$

where $R_s = k/k_s$, k and k_s are horizontal permeabilities in undisturbed and smeared zone, respectively; α, n, D are previously defined and the others are shown in the Fig. 4. Replacing k in Eq. (3) by value of k_e from Eq. (4) one can get the value of horizontal converted permeability for the 2-D model, k_m, in terms of the horizontal permeability of unimproved soil, k, as shown below:

$$k_m = \frac{\pi D(1-a_s)k}{2S\left[\log_e\left(\frac{\alpha D}{d_s}\right) + R_s\log_e\left(\frac{d_s}{d_w}\right)\right]} \qquad (5)$$

FINITE ELEMENT MODELLING AND SOIL MODEL

The finite element modelling for consolidation analysis of embankment on vertical drains consisted of 62 elements with 192 nodes as shown in Fig. 6. The 8-node quadri-lateral elements with the same number of nodes for both pore pressure and displacement were used for subsoils. The incompressible boundary was chosen at depth of 8 m, at the top of the stiff clay layer. Zero excess pore pressure boundaries were set at ground surface and at the nodes on vertical drains. Embankment on granular piles was discretized in to 94 elements with 251 nodes for consolidation analysis due to embankment load as shown in Fig. 7. The incompressible boundary was chosen at depth of 8 m, where the stiff clay appears. Zero excess pore pressure boundaries were set at ground surface and at nodes of granular material.

Since pore pressure drawdown due to excess groundwater pumping on any drains may be assumed to be the same, axi-symmetric model for a single drain with its surrounding soil could be used for subsidence analysis separately. Based on the observed data from Khaw (1986), the boundary conditions can be set up for each time step of the calculation as demonstrated by Long (1992).

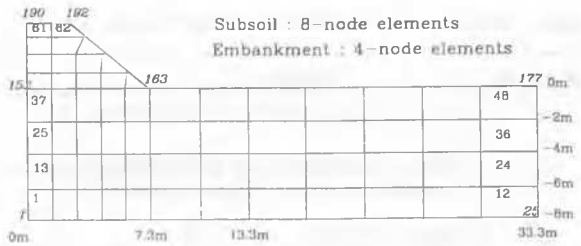

Fig. 6 Finite element discretization of EVD

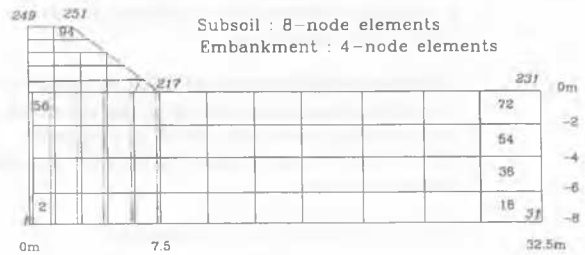

Fig. 7 Finite element discretization of EGP

Programs CON2D (Duncan et al, 1981) and CONSAX (D'Ozario et al, 1982) were used for consolidation analyses due to embankment loading and pore pressure drawdown, respectively. CON2D is a finite element programs for analysis of consolidation in saturated and partly saturated earth masses. The program treats the couple problem of deformation and fluid flow. It can be used to calculate movements and pore pressure variations under undrained, partly drained and fully drained conditions. CONSAX is a modification of CON2D for axi-symmetric case. Soil model used in these programs, namely Revised Cam Clay model (Duncan et al, 1981), is a revision from Modified Cam clay model which was generalized by Roscoe and Burland (1968). To make Modified Cam clay model more suitable for presenting the stress-strain characteristics of over consolidated clays and compacted soils, the failure surface and yield surface were modified in revised version as shown in Fig. 8. Soil parameters required for this model are: κ, λ, ν, e_o, p_r, p_p, and p_{break} which are defined in Table 1 and Fig. 8.

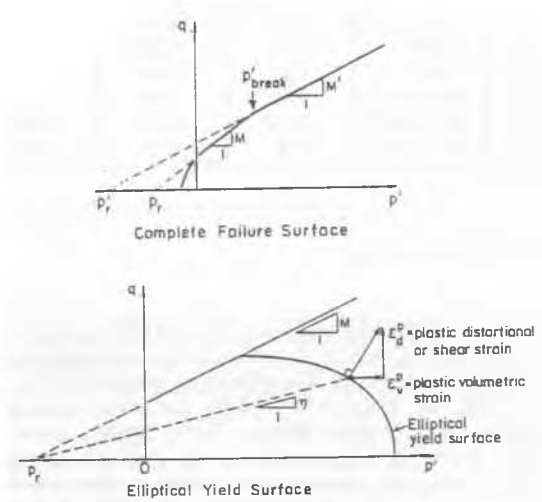

Fig. 8 Failure surface and yield surface of Revised Cam clay model (Duncan et al, 1981)

Table 1 Soil parameters required for Revised Cam Clay Model

Parameters	Definition
M	Slope of failure surface that is shown in Fig. 8
λ, κ	Slopes of compression and swelling lines of isotropic consolidation test in e-$\log_e$ plot, respectively
e_o	Initial void ratio
ν'	Drained Poissons ratio
p_r	Cohesion intercept that can be calculated by $p_r = c'\cot\phi'$, where c' and ϕ' are drained cohesion and friction angle, respectively.
p_p	Intersection of yield surface and p' axis calculated from the maximum stress state (p', q) that the soil has been experienced in the past, where p', q are mean effective and deviatoric stresses, respectively, in triaxial-isotropic condition.
p_o	Mean effective stress at current condition.
p'_{break}	Value of p' at the intersection of the bi-linear failure surface as shown in Fig. 8.

Soil parameters used in this analysis is presented in Table 2, in which k_h, k_v are horizontal and vertical permeabilities of untreated ground, respectively, and k_m is the converted horizontal permeability of the 2-D plane-strain model. The other parameters are shown in Fig. 8.

Table 2 Soil parameters used in this study

Para-meters	Unit	Weathered Clay	Soft Clay	Medium Clay
γ	t/m^3	1.85	1.55	1.62
e_o		1.22	2.30	1.68
κ		0.03	0.06	0.04
λ		0.30	0.50	0.35
ν		0.33	0.38	0.38
M		1.10	1.00	1.10
p_r	t/m^2	5.71	0.00	0.00
p_p	t/m^2	8.00	5.04	7.65
k_h	10^{-4} m/day	2.00	1.00	1.00
k_v	10^{-4} m/day	1.00	0.50	0.50
k_m	10^{-4} m/day	0.46	0.23	0.23

RESULTS OF ANALYSES

Observed and predicted total settlements of EGP and EVD are shown in Figs. 9 and 10, respectively. Comparison of primary settlements deleting subsidence with the other methods and the observed data are presented in Figs. 11 and 12. Only FEM and Asaoka (1978) method gave good results for both embankments. The Skempton-Bjerrum (1957) method overestimated considerably the settlement of EGP. One of the reasons is that the stress concentration ratio used in conventional method is often conservative, particularly in case of low displacement ratio. It is noted that the smear effects were considered by using the previous findings of Bergado et al (1992), wherein, k_h/k_s = 10 and d_e/d_w = 2.5 for prefabricated vertical drains. Also using k_h/k_s = 10 for embankment on granular piles, the value of d_e/d_p = 2 has been found to be very good fitting parameter in this case of study.

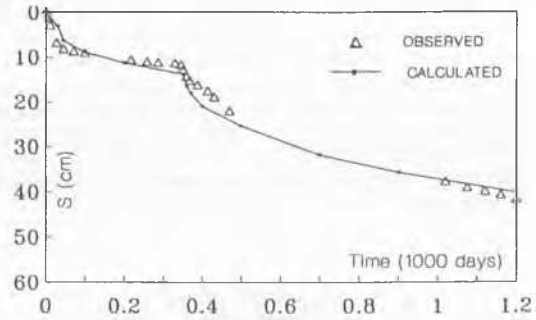

Fig. 9 Comparison of total settlements of EGP

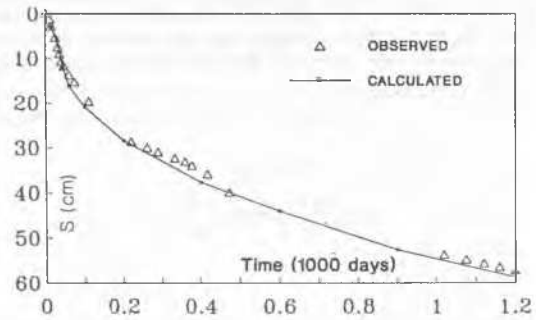

Fig. 10 Comparison of total settlements of EVD

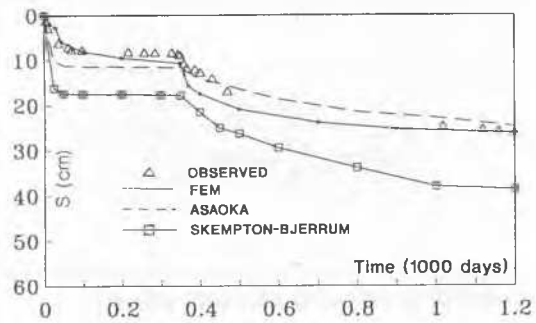

Fig. 11 Comparison of primary settlements of EGP

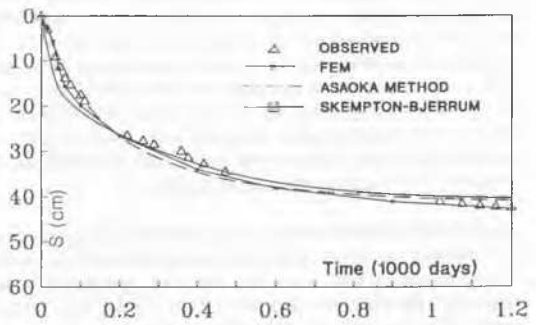

Fig. 12 Comparison of primary settlements of EVD

Variations of pore pressures vs time at depths of 3 and 6 m are shown in Figs. 13 and 14. The results of FEM fitted better than that of the analytical methods (Henkel, 1960; Skempton-Bjerrum, 1954). The analytical methods overestimated at the depth of 3 m but underestimated at depth of 6 m. This may be explained that the vertical drainage of the multi-layer subsoil cannot be verified in these analytical methods, particularly the higher permeability of the weathered clay crust.

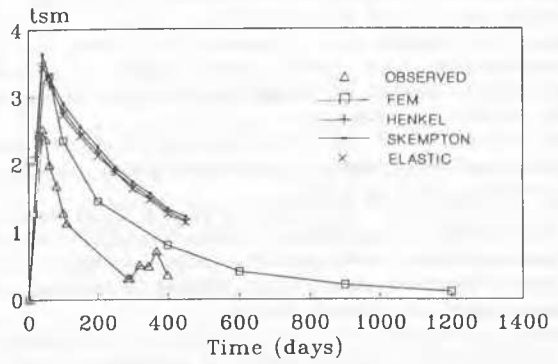

Fig. 13 Comparison of pore pressure at depth of 3 m of EVD

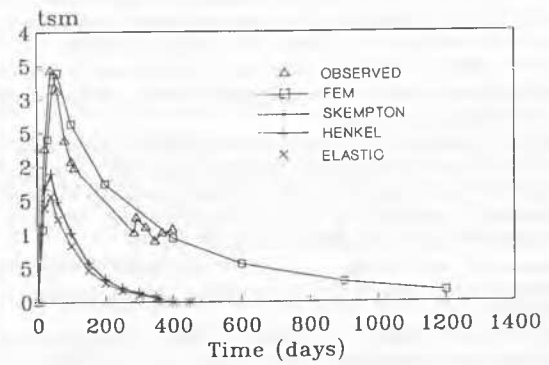

Fig. 14 Comparison of pore pressure at depth of 6 m of EVD

The reduction of consolidation settlement and the increase of bearing capacity are considered to be caused by the stress concentration on the granular piles. The stress concentration factor, n, vs time and depth obtained in this analysis is shown in Figs. 15 and 16, respectively. Observed data was only recorded at one location on ground surface level. Comparison with the predicted values at the depth of 1 m, the results shown that the trends are the same but the lower observed values may be due to the lateral yielding of the piles at the ground surface. Inaccuracies of the total earth pressure cells due to arching effects caused by the presence of such instruments may also have effected the results.

The calculated shear stress-to-shear strength ratio, q/q_{full}, in EGP and EVD grounds are presented in Fig. 17. The values of this ratio increased with degree of consolidation in granular pile material but oppositely in surrounding improved soft clay. For the case of EVD this ratio decreased about 1.6 to 2 times in one year after construction, and almost remained constantly from 600 days after construction.

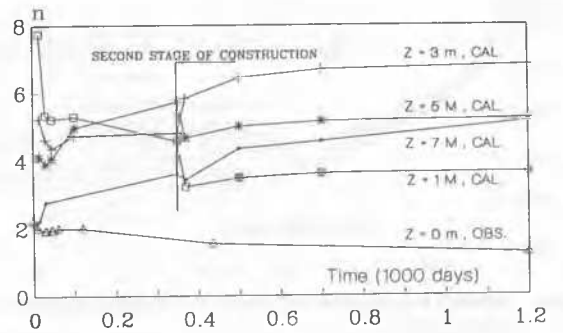

Fig. 15 Stress concentration ratio, n, versus time of EGP

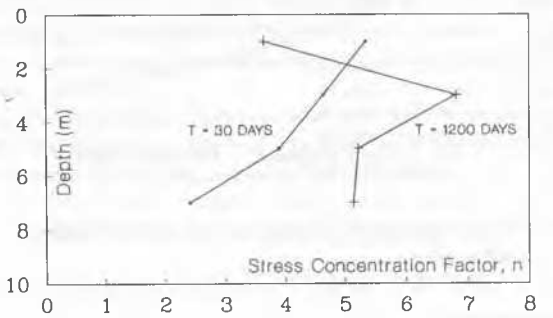

Fig. 16 Stress concentration ratio, n, versus depth of EGP

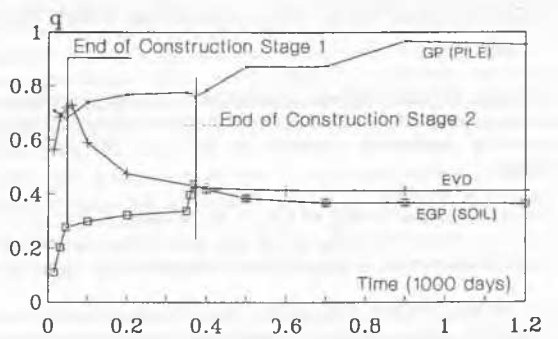

Fig. 17 Shear stress-to-shear strength ratio, q/q_{full}, vs time

Consolidation settlements due to pore pressure drawdown have been estimated separately. Figures 18 and 19 show the predicted settlements with and without subsidence for different lengths of vertical drain and granular pile, respectively. The results indicated that in the case of full improvements down to the stiff clay layer at depth of 8 m, where the pore pressure dropped about 1.3 m compared with the current hydrostatic pore pressure, additional subsidence were obtained for both embankments. Hence, it is necessary to consider the condition of pore pressure drawdown in choosing the depth of ground improvement in subsiding area. With the conditions of the test site, in terms of total settlement, it is better to use 6 m length down to the medium stiff clay layer rather than 8 m length for both vertical drain and granular pile improvements.

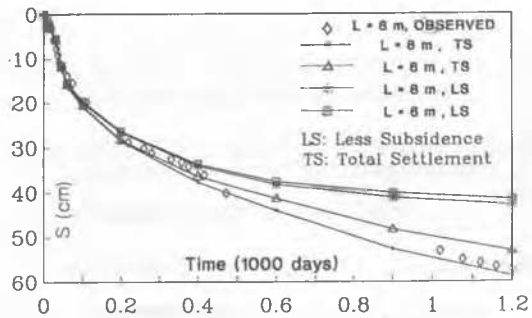

Fig. 18 Observed and predicted settlements with different lengths of vertical drains

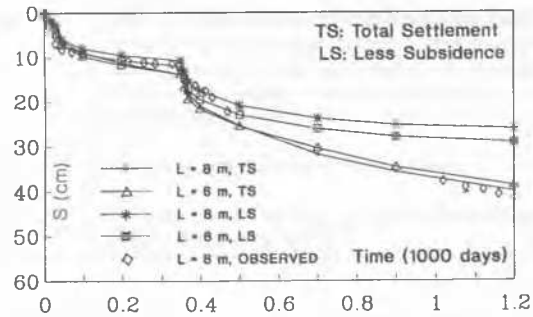

Fig. 19 Observed and predicted settlements with different lengths of granular piles

CONCLUSIONS

An elasto-plastic finite element program has been applied for 2-D consolidation analysis of embankment on soft ground improved by vertical drains and granular piles. The following conclusions can be made from this study:

(1) The good agreements between observed and predicted time-settlements and pore pressures indicated that the 2-D plane strain model with the converted permeability introduced in this paper can yield reliable results.

(2) Using the previous finding of k_b/k_s = 10, the value of d_s/d_w = 2.5 and d_s/d_p = 2 have been found to be very good fitting parameters for prefabricated vertical drain and granular compaction pile, respectively.

(3) The predicted stress concentration factor, n, ranges from 2.4 to 8 depending on the degree of consolidation and location within the granular piles. The value of n is decreased with time at shallow depth but it is increased with time at deeper depth.

(4) For the condition of the test site, penetration of vertical drains or granular piles to the stiff clay layer at 8 m deep, where there exists pore pressure drawdown about 1.3 m from the current hydrostatic condition, additional subsidence may occur. In terms of total settlement, it is considered more economical to use 6 m length only down to the top of the medium stiff clay layer rather than using 8 m length for both improvements.

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