INVERSE ANALYSIS OF GEOTECHNIC PARAMETERS ON IMPROVED SOFT BANGKOK CLAY

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Reprinted from

Journal of Geotechnical Engineering, Vol.118, No.7

1992, pp.1012~1030

Inverse Analysis of Geotechnic Parameters on Improved Soft Bangkok Clay

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Abstract: Deformation, strength, and flow parameters were determined by inverse analyses on geotechnical data obtained from two full-scale test embankments constructed on improved soft Bangkok clay. One site was improved with prefabricated vertical drains (PVD), and an adjacent site was improved with compacted granular piles (CGP). In the PVD site, the ratio of c_h (field) to c_h (lab) ranged from 1.5–4, and the c_h value was 4.90 m²/year. The value of k_h/k_s was about 10, with d_s/d_w of 2.5. In the CGP site, the ratio of c_v (field) to c_v (lab) ranged from 1.3–12. The smeared zone was considered by reduction to CGP diameter by one-fifth. The ratio of E_u to S_u was determined to be 150. The observed final settlements in the CGP site was 30–35% lower than those in the PVD site. Using back-analyzed soil parameters, the settlements of the test embankments were predicted well. Both ground improvement schemes are effective in improving the soft Bangkok clay.

Introduction

Recently, back or inverse analyses in both laboratory models and field prototypes have indicated promising results, and they have become an accepted model of validating theories and evaluating soil parameters for use in available and accepted methods. The accuracy with which an inverse analysis is carried out depends on the working assumptions and knowledge of the advantages and limitations of the theoretical models being used and validated. This study aims to evaluate the deformation, strength, and flow parameters of the soft Bangkok clay improved by prefabricated vertical drains (PVD) and compacted granular piles (CGP) in conjunction with embankment preloading, by back calculation of soil parameters using available field data. The detailed description of the improvement and preliminary data were reported elsewhere (Bergado et al. 1988a, b).

Site Location and Soil Profile

The test site is on the campus of the Asian Institute of Technology, about 42 km north of Bangkok on the central plain (Chao Phraya) of Thailand. The subsoil profile reveals the three topmost sublayers of Bangkok marine clay deposits underlain by alternating layers of very dense sand and stiff

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Note. Discussion open until December 1, 1992. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 8, 1990. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 118, No. 7, July, 1992. ©ASCE, ISSN 0733-9410/92/0007-1012/$1.00 + $.15 per page. Paper No. 26640.

clay extending to depths of more than 500 m. The Bangkok clay at the site consists of the uppermost 14–15 m, which can be divided into three distinct layers: namely, weathered clay, soft clay, and stiff clay (Fig. 1). The depth of the water table varies seasonally from 0.5–2.5 m, averaging about 1.5 m. The index properties of the compressible soft clay are shown in Fig. 1. The stress history at the site is depicted in Fig. 2. Field vane tests were carried out using a Geonor vane apparatus at depths of 8 to 9 m. Typical results also are shown in Fig. 1.

Ground Improvement by Prefabricated Vertical Drains

Barron (1948) presented a solution to the differential equation for problem of consolidation of a soil cylinder containing a central drain. The differential equation formulated in terms of polar coordinates is expressed as follows:

$$\frac{\partial u}{\partial t} = c_h \left(\frac{1}{r}\frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) \quad \text{.......... (1)}$$

where u = the excess pore pressure at any point and at any time by virtue of radial flow only; r = the radial distance of the considered point from the center of the drained soil cylinder; t = the time after an instantaneous increase of the total vertical stress; and c_h = the horizontal coefficient of consolidation. From (1), the general expression for average degree of consolidation due to horizontal drainage derived by Barron (1948) was modified by Hansbo (1979) as follows:

$$\bar{U}_h = 1 - \exp\left(\frac{-8T_h}{F} \right) \quad \text{.......... (2)}$$

where

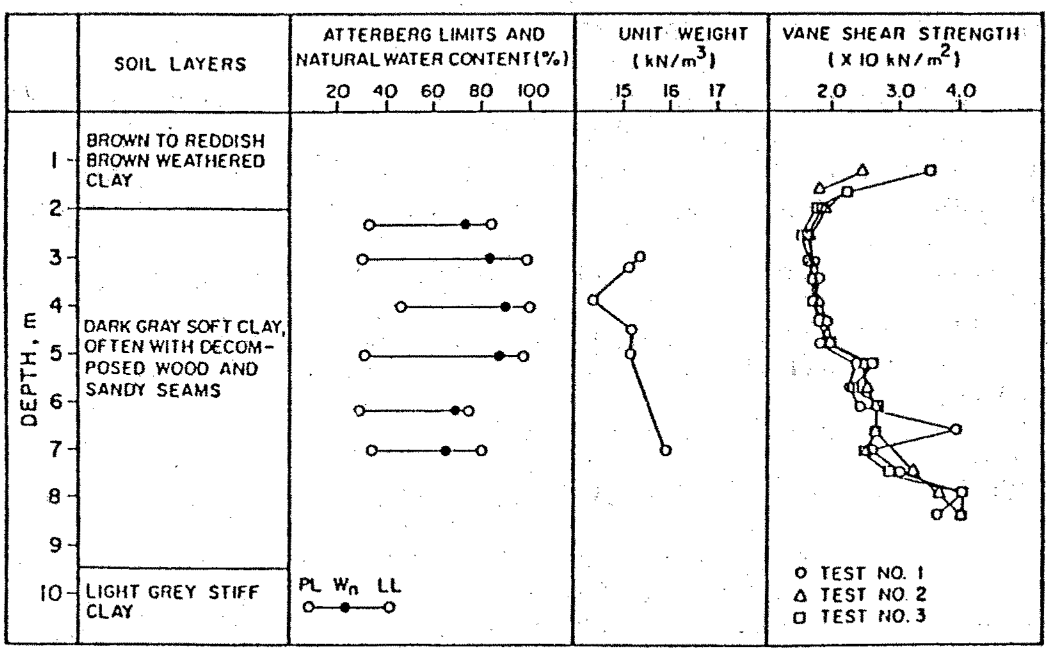

FIG. 1. Subsoil Profile at Site with Corresponding Index Properties and Undrained Shear Strength

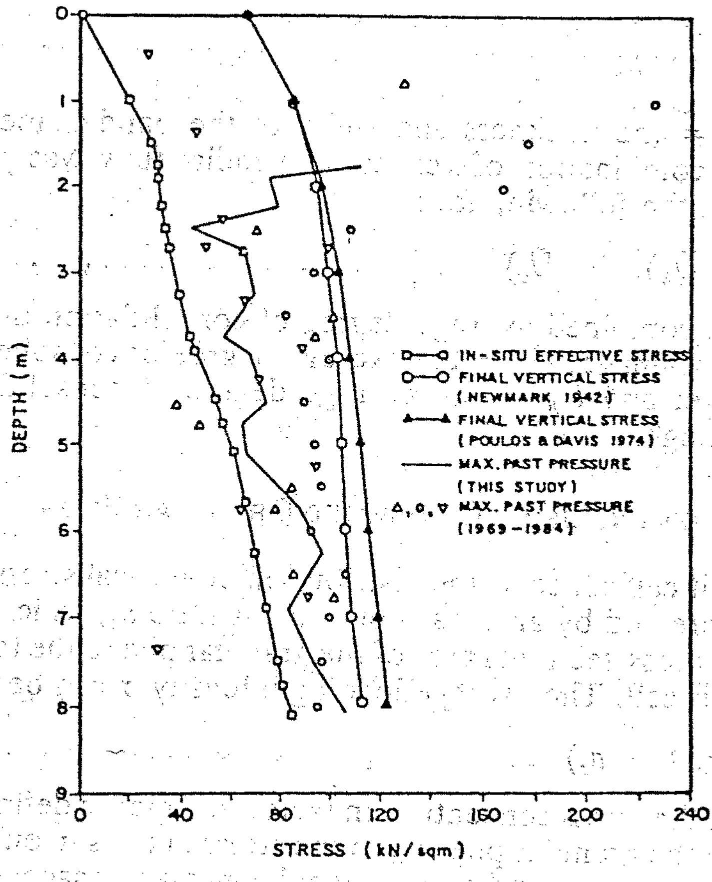

FIG. 2. Stress History at Site with Results of Previous Investigations

$$T_h = \frac{c_h t}{D_e^2} \qquad (3)$$

and D_e = the diameter of the equivalent soil cylinder; F = the factor that expresses the additive effect due to the spacing of drains $F(n)$, the smear effect F_s, and well resistance F_r. Hansbo (1979) gave the following expressions for the spacing, well resistance, and smear factors, respectively:

$$F(n) = ln\left(\frac{D_e}{d_w}\right) - \frac{3}{4} \qquad (4)$$

$$F_r = \pi z(L - z)\left(\frac{k_h}{q_w}\right) \qquad (5)$$

$$F_s = \left(\frac{k_h}{k_s} - 1\right) ln\left(\frac{d_s}{d_w}\right) \qquad (6)$$

$$F = F(n) + F_r + F_s \qquad (7)$$

where d_s = the diameter of the smeared zone; L = the length of the drain when opened at one end only; z = the vertical distance from open end of drain; k_s = the permeability of the disturbed zone in the horizontal direction; and q_w = the discharge capacity of the drain. Recently, FEM analysis (Rixner et al. 1986) showed that the equivalent diameter of the drain (d_w) preferable for use in practice can be obtained as

$$d_w = \frac{a + b}{2} \qquad (8)$$

where a and b = the thickness and width of the band-shaped drain, respectively. The combination of vertical and radial flow was presented by Carillo (1942) in the following form:

$$\bar{U} = 1 - (1 - \bar{U}_h)(1 - \bar{U}_v) \qquad (9)$$

where $\bar{U}$ = the combined average degree of consolidation due to vertical and horizontal drainage; $\bar{U}_v$ = the average degree of consolidation due to vertical drainage; and $\bar{U}_h$ = the average degree of consolidation due to horizontal drainage.

Ground Improvement Using Compacted Granular Piles

From the unit cell concept, the distribution of vertical stress within a unit cell can be expressed by an area replacement ratio a_s, which is defined as the ratio of the cross-sectional area of the granular pile to the total composite area of the unit cell. The average loading intensity σ can be calculated as

$$\sigma = \sigma_s a_s + \sigma_c(1 - a_s) \qquad (10)$$

Introducing the stress concentration factor n, which is defined as the ratio of the stress in the granular pile, σ_s, to the stress in the surrounding cohesive soil, σ_c, the stresses in the clay and granular pile are, respectively, given as

$$\sigma_c = \frac{\sigma}{1 + (n - 1)a_s} = \mu_c \sigma \qquad (11)$$

$$\sigma_s = \frac{n\sigma}{1 + (n - 1)a_s} = \mu_s \sigma \qquad (12)$$

The equilibrium method (Aboshi et al. 1979) of estimating the settlement of composite ground, which was based on conventional, one-dimensional consolidation theory, was found to be less realistic and theoretically sound than the finite element method. The finite element method incorporates material nonlinearity, interface slip, and suitable boundary conditions. Solutions in terms of nonlinear finite element design curves to estimate the settlement of the composite ground were presented by Balaam et al. (1977).

Procedures

A 4.0-m-high test embankment preloading was constructed on improved ground with prefabricated vertical (PV) drains to study the effectiveness of PV drains on soft Bangkok clay. The rate of embankment loading is shown in Fig. 3. The drains were installed in triangular pattern at 1.5-m center-to-center spacing by means of a special mandrel to a depth of 8.0 m. A rectangular-shaped mandrel was used, which had inner dimensions of 28.0 mm by 133.0 mm and outer dimensions of 45.0 mm by 150.0 mm, just large enough to contain the 3.0-mm by 95.0-mm Mebra prefabricated vertical drains made of geotextiles. The embankment plan and section views, including the layout of prefabricated vertical drains, are shown in Figs. 4(a) and (b).

A 2.4-m-high test embankment was constructed on improved ground with granular piles. After 345 days, the test embankment was raised to a height of 4.0 m to provide a meaningful basis of comparison with the performance of a nearby test embankment constructed on improved ground using prefabricated vertical drains (Fig. 5). The granular piles were 300.0 mm in diameter and 8 m long, fully penetrating the soft clay layer. The granular piles were arranged in a triangular pattern with a center-to-center spacing of 1.5 m. They were installed by first driving an open-ended steel casing. A large auger was then used to remove the clay inside the casing. The granular materials, consisting of 20-mm maximum size sandy gravel, were carefully added in layers and compacted by means of dropping a 1.6-kN hammer at a height of 0.6 m to a steel disc placed on the surface of the granular material as the casing was withdrawn. Fifteen blows were applied to each layer, compacting the thickness of each layer by about 0.60 m. The friction angles of compacted gravel were obtained from direct shear tests using a shear box dimension of 100 mm × 100 mm × 60 mm, and they were found to vary from 39–45°, with compacted densities ranging from 17–18.1 kN/m³. The plan and cross section of the embankment, including the layout of field instrumentation, are illustrated in Figs. 6(a) and (b).

The solution suggested by Olson (1977) for time-dependent loading was not employed, because the rate of loading was fast enough to be considered instantaneous, as demonstrated in Figs. 3 and 5.

Tests and Instrumentations

The locations of surface and subsurface settlement gauges, piezometers, and inclinometers for PVD and CGP sites are shown in Figs. 4(a) and (b) and 6(a) and (b), respectively.

Additional field tests and sampling were made and additional monitoring instruments are installed, as described in the following. Three boreholes were drilled down to a 9.0-m depth. Two of these boreholes (BH1 and BH2) were made through the 4.0-m-high embankments on compacted granular piles and prefabricated vertical drains, respectively. The third borehole (BH3) was drilled on natural unimproved ground. A 3-in.-diameter (76.0-

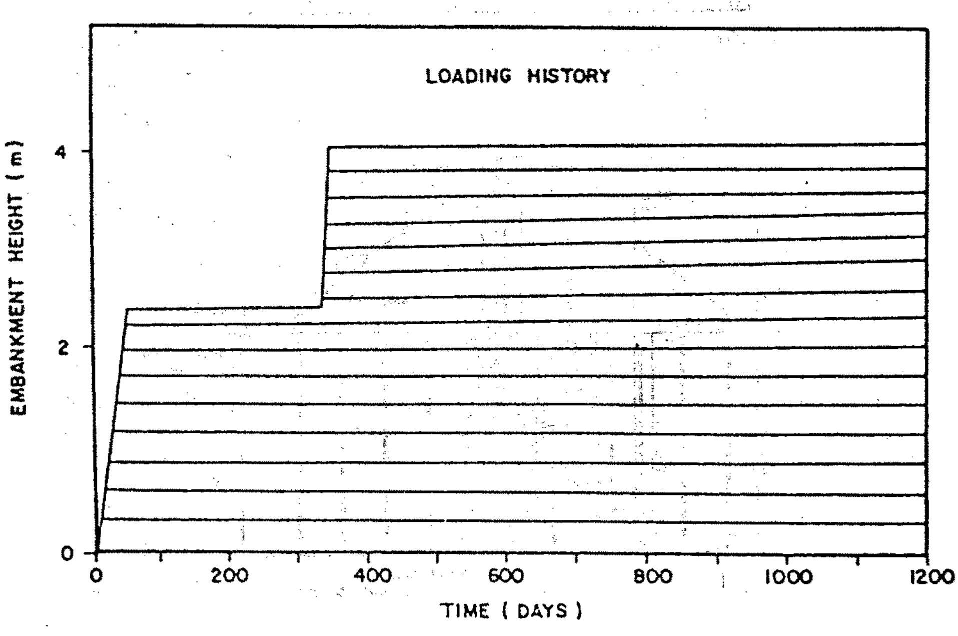

FIG. 5. Embankment Loading History at CGP Site

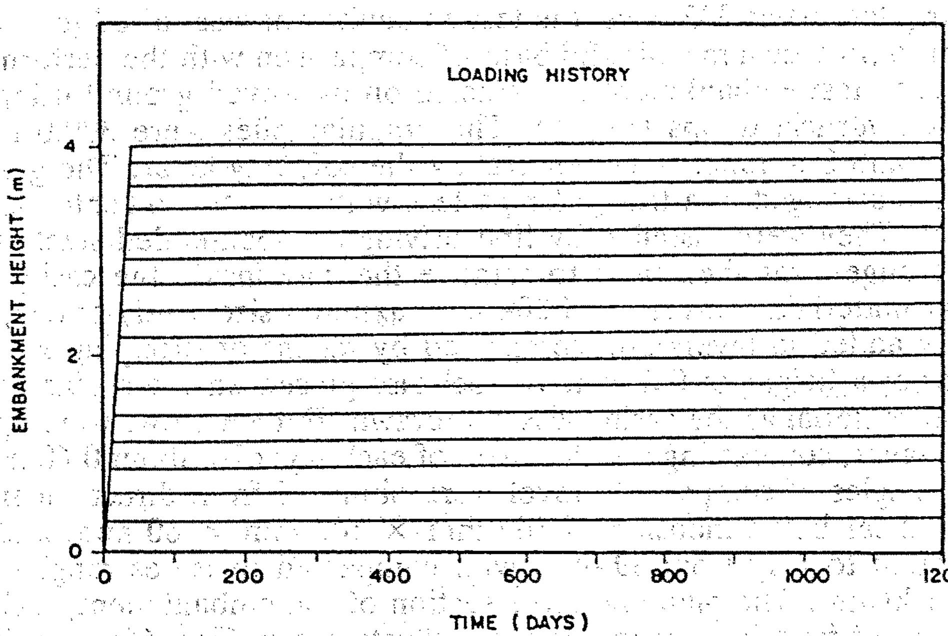

FIG. 3. Embankment Loading History at PVD Site

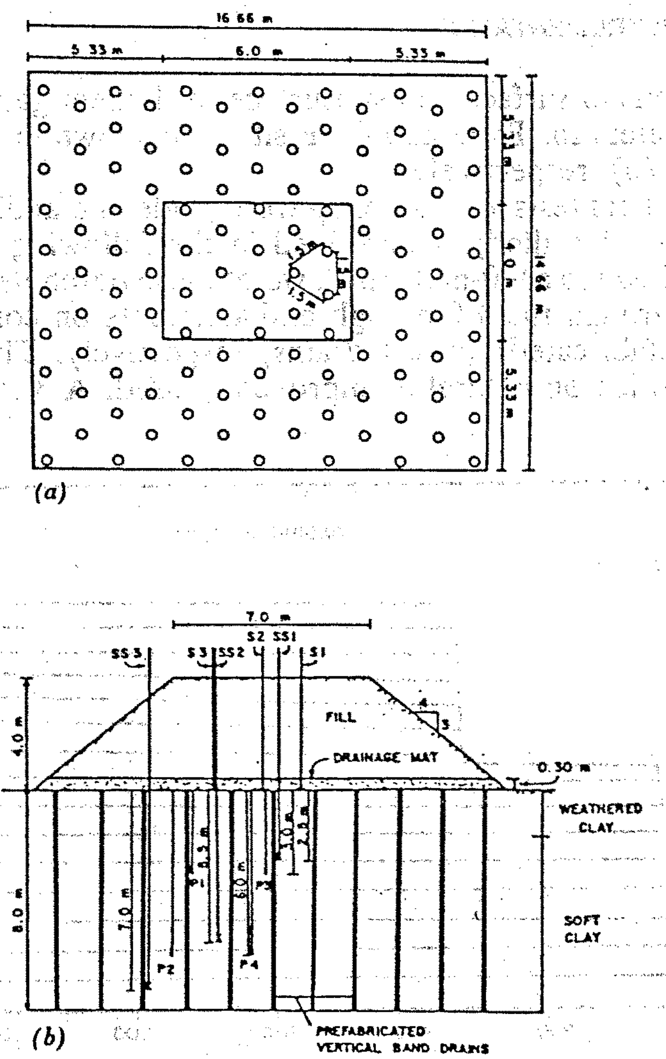

FIG. 4. Embankment Plan, Section View, and Layout of Field Instrumentation at PVD Site

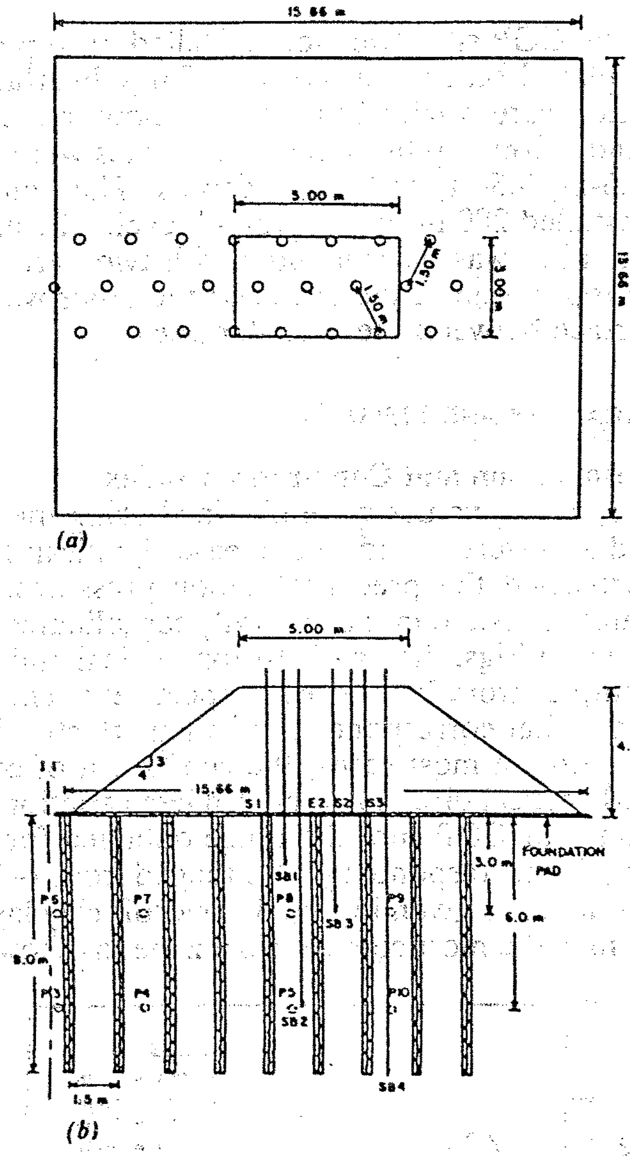

FIG. 6. Plan and Embankment Cross-Section Including Layout of Field Instrumentation at CGP Site

mm) thin-walled tube piston sampler was used, and continuous sampling was done by pushing manually. Fourteen conventional consolidation tests were conducted, consisting of six sets for samples under each of the two embankments and two sets for the unimproved samples. In the horizontal oedometer tests, samples were cut at orientations perpendicular to their original directions in the field, as suggested by Laminen and Rathmeyer (1981). A total of six field vane shear tests were also conducted to depths of 8-m at the site: two each on the subsoil under both embankments and two on natural unimproved ground. Aside from the original eight, previously installed, closed hydraulic piezometers, an additional seven open standpipe piezometers were installed to monitor the variation of pore pressures within and outside of the test embankments. Two were installed in each embankment of 3.0-m and 6.0-m depths. The other three were installed 10 m away at the same depths as dummy piezometers. The fluctuations of ground-water levels were monitored by means of an observation well consisting of perforated pipes. Inclinometers had been installed 12.0 m deep for the PVD site and 13.0 m deep for the CGP site to measure subsurface lateral displacements. There were three surface settlement plates installed in each embankment. For the CGP site, two were installed on top of the granular piles and one was placed on natural ground. For subsurface settlements, four settlement plates were originally installed below the CGP site at 1.5-m, 3.0-m, 6.0-m, and 8.0-m depths, while three plates were installed below the PVD site at 2.5-m, 5.5-m, and 7.0-m depths. The bench mark was a deep well casing installed 200 m below ground level. The embankment on compacted granular piles was instrumented with two earth pressure cells; one was placed on top of one of the piles and the other was placed on the original ground surface between the granular piles.

ANALYSIS OF LABORATORY AND FIELD DATA

Coefficients of Consolidation and Compression Index

The laboratory c_h and c_v values were obtained using the square-root-of-time fitting method and were found to decrease significantly as the consolidation stress approached the preconsolidation pressures. They then remained approximately constant in the normally consolidated region. Typical results are illustrated in Figs. 7(a) and (b) for vertical and horizontal oedometer tests on samples from PVD sites, respectively. The selected values of c_v and c_h are those that correspond to the final effective vertical stresses anticipated in the field. In most cases, the coefficient of consolidation for both horizontal and vertical directions showed less variation in the normally consolidated region. For CGP site, the range obtained for (c_h/c_v) is about 1–4. For PVD site, the corresponding ratio ranged from 1–7. The variation of the compression index is given in Fig. 8. The soft clay layer between the depths of 2.5–5.0 m is the most compressible zone, as shown in this figure.

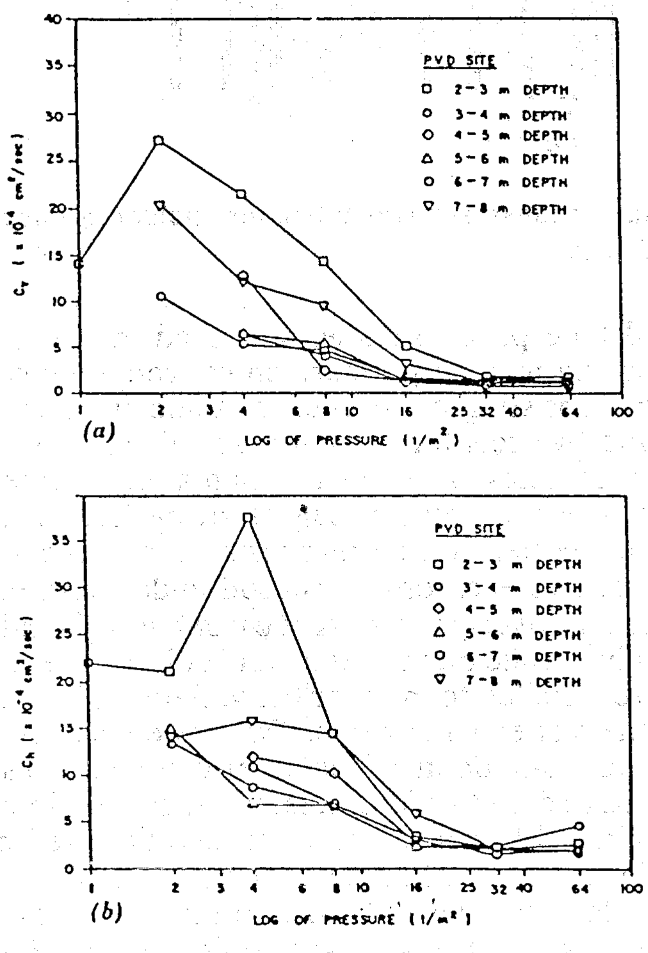

FIG. 7. Values of c_v and c_h on Samples Obtained from PVD Site

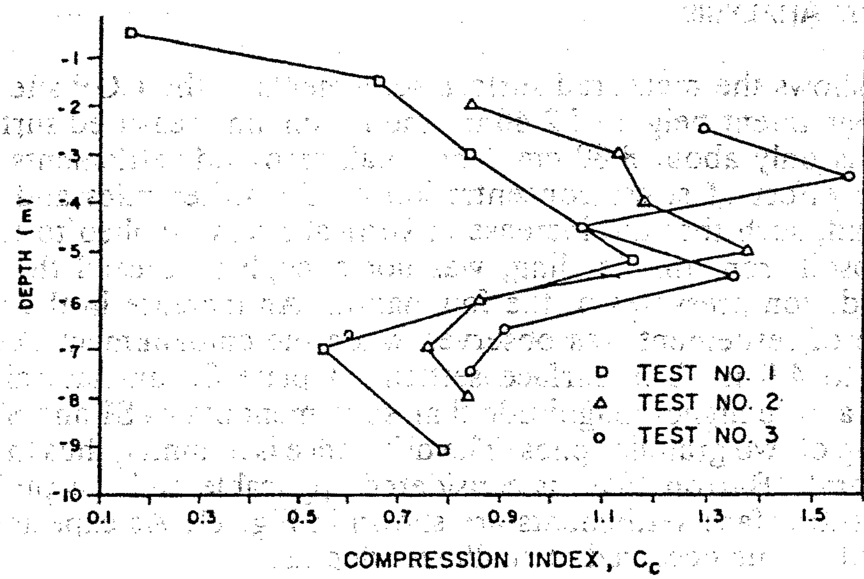

FIG. 8. Variations of Compression Index with Depth

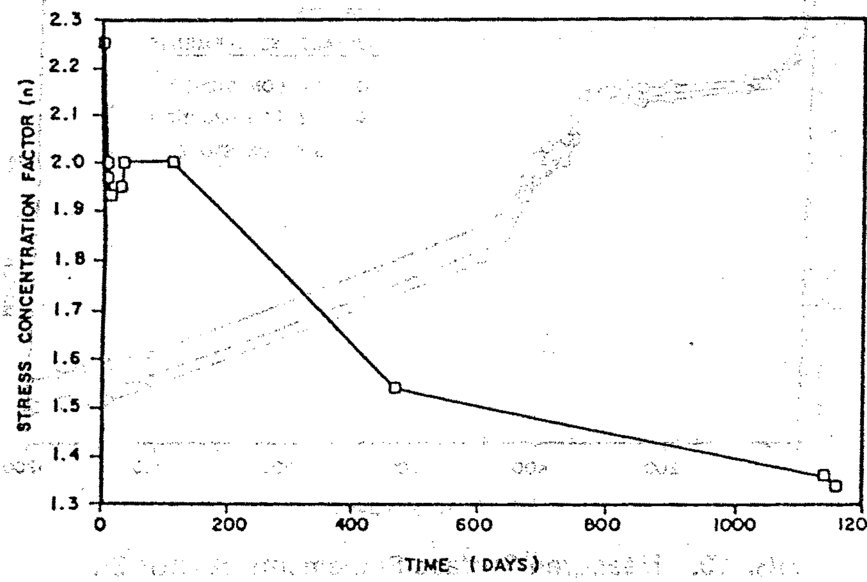

FIG. 9. Measured Stress Concentration Factor (n) with Time

Stress Concentration Factor (n)

The values of the stress concentration factor, n, as measured from the two total earth pressure cells installed on top of the granular pile and in between the piles, are shown in Fig. 9. They initially increased at the initial stage of loading. This initial increase is partially due to the passive earth resistance of the soil, which is progressively mobilized to restrain the lateral dilatancy of the granular piles. As the surrounding soil consolidated, the load on the piles increased. In this study, low area replacement ratio, a_s, was used and the measured value of n decreased to 1.34 (Fig. 9). This result is contrary to that reported in the literature (Juran and Guermazi 1988), in which the value of the stress concentration factor increased with time (i.e., when the surrounding soil consolidated). The possible reasons for these variations in observation include lateral yielding of the piles resulting from the load transfer from the piles to the surrounding soil, the effects of low replacement ratio, and inaccuracy of the total earth pressure cells due to arching effects caused by the presence of such instruments.

SETTLEMENT ANALYSIS

Fig. 10 shows the measured surface settlements at the CGP site. At the initial embankment height of 2.40 m, the maximum measured surface settlement was only about 8.80 cm. The small recorded settlements may be due to the effect of stress concentration on the stiffer piles and the low applied load, such that the increase in vertical stress, applied to the foundation subsoil from this loading, was not enough to exceed the existing preconsolidation pressures of the foundation. An increase in the rate and magnitude of settlement was observed when the embankment height was increased to 4.0 m. The surface settlement plate S2 on natural ground registered a little higher magnitude than settlement plates S1 and S3, which were on top of two granular piles. The difference is minimal, thus the "equal strain theory" (Barron 1948) is considered applicable and acceptable. The observed subsurface settlements are shown in Fig. 11. As expected, much higher settlements occurred at shallower depths.

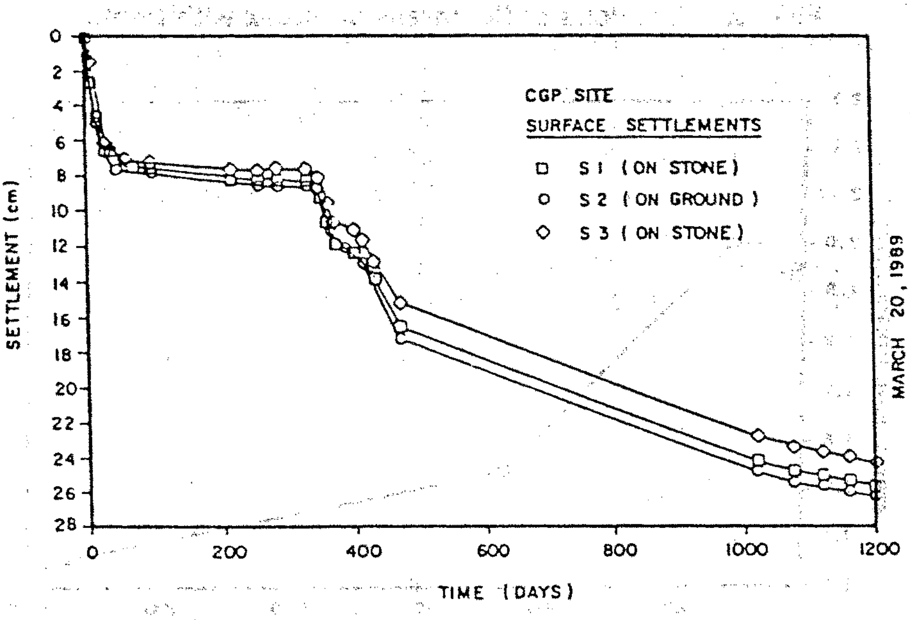

FIG. 10. Measured Surface Settlements at CGP Site

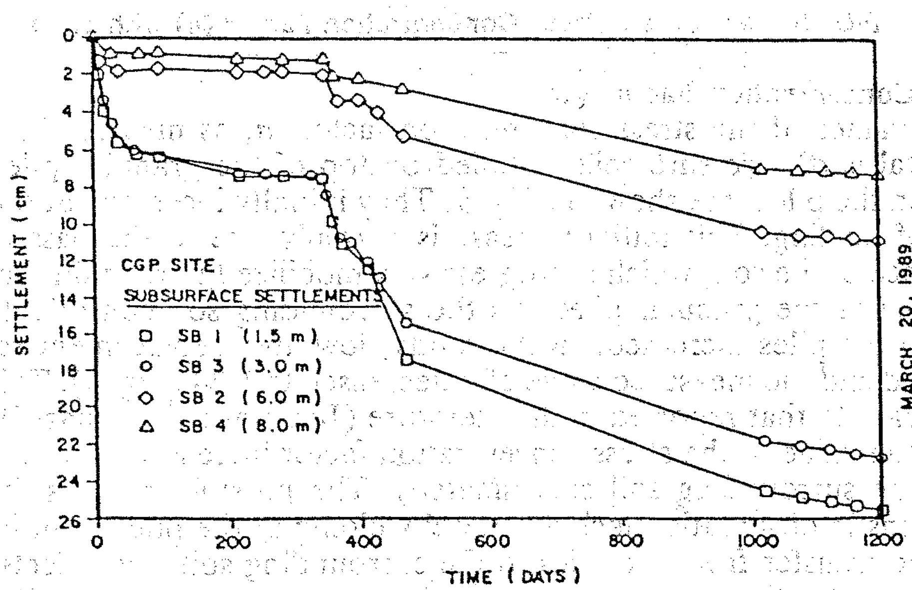

FIG. 11. Measured Subsurface Settlements at CGP Site

Fig. 12 shows the measured surface settlements at the PVD site. Higher settlements occurred in S2, which was at the center of the embankment. Fig. 13 shows the measured subsurface settlements at the PVD site. The maximum subsurface settlement obtained, which included some effect of secondary compression, was about 292.0 mm after 1,200 days. This was derived from SS1, which was near the center of the embankment at a shallow depth of 2.50 m below the ground surface.

Asaoka (1978) suggested a graphical method to determine the final settlement. The observed time-settlement curve plotted to an arithmetic scale was divided into equal time intervals, Δt (usually between 10 and 100 days). The settlements ρ_1, ρ_2, . . . , corresponding to times t_1, t_2, . . . , are read off and plotted as points (ρ_f, ρ_{f-1}) in the coordinate system, as shown in Figs. 14 and 15 for PVD and CGP sites, respectively. A straight line is fitted through the points. The slope of this line is β_1, and its intercept with the ordinate axis is β_0. The 45° line with $\rho_f = \rho_{f-1}$ is also plotted. The point where the line intersects the 45° line gives the final settlement, ρ_f.

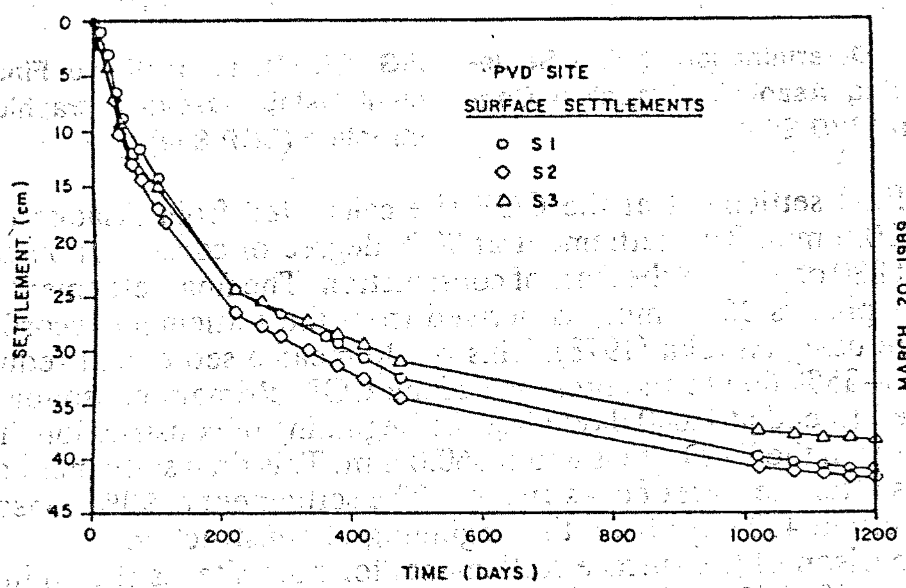

FIG. 12. Measured Surface Settlements at PVD Site

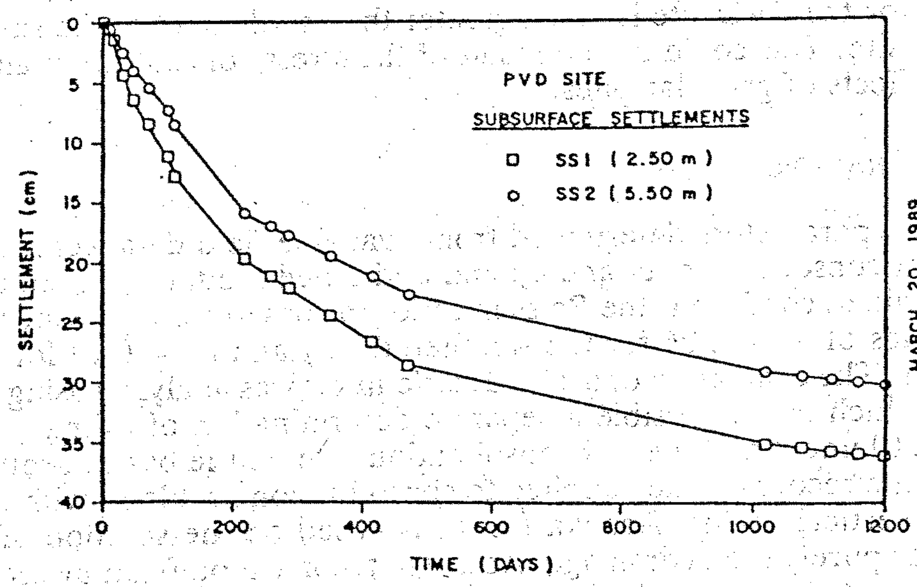

FIG. 13. Measured Subsurface Settlements at PVD Site

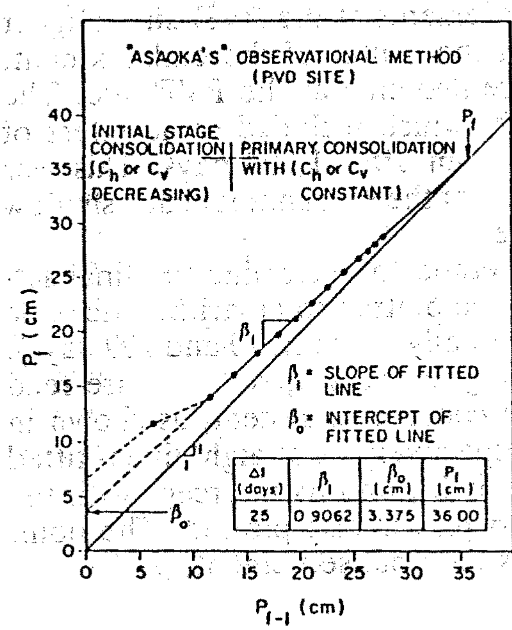

FIG. 14. Determination of Final Settlement Using Asaoka's Graphical Construction (PVD Site)

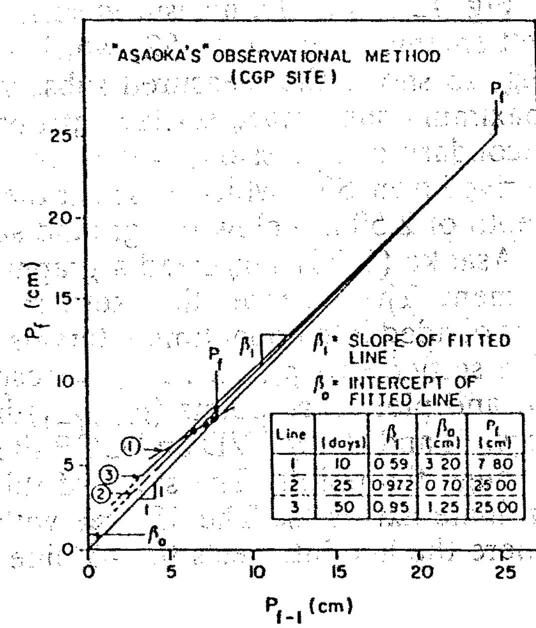

FIG. 15. Determination of Final Settlement Using Asaoka's Graphical Construction (CGP Site)

The final settlement at the CGP site calculated from Asaoka (1978) is about 250.0 mm. The settlement at 90% degree of consolidation occurred at about 880 days from the start of construction. The final settlement without granular piles is 360.0 mm, as derived from the settlement records of the PVD site using Asaoka (1978). This would mean a settlement reduction of about 30–35% due to the presence of the CGP. Primary consolidation was estimated to end at 1,040 days after the beginning of construction. The final settlement in the PVD site is about 360.0 mm. This final settlement occurred 610 days after the start of construction. The settlement at 90% consolidation was observed 430 days from the beginning of construction.

Comparison of the surface settlements for both sites is shown in Fig. 16. There was 30–35% reduction of settlement by using granular piles. For the case of subsurface settlements, the comparison for both embankments is depicted in Fig. 17. It is interesting to note that the settlement at the 5.50-m depth for the PVD site is much greater than settlement at the surface of the CGP site. This could be the result of the stress concentration and reinforcing effects of granular piles.

Inverse Analysis

The soil parameters determined from available field data were the coefficients of consolidation, c_h and c_v, and undrained modulus, E_u. In the case of the drained condition, the Poisson ratio was based from previous studies with values of $\nu' = 0.26$ for the weathered clay and $\nu' = 0.24$ for the soft clay layer. The evaluation of drained settlements was analyzed using Asaoka (1978), which made possible a separate determination of primary consolidation settlement and the soil consolidation rate on the basis of settlement-versus-time records. When applied to consolidation problems with the presence of vertical drains, Asaoka (1978) is based on the solution of Barron (1948) for purely radial drainage. As a result, some modifications were made in this study to introduce Hansbo's equation (Hansbo 1979), which considers the smear effect, into Asaoka's equation.

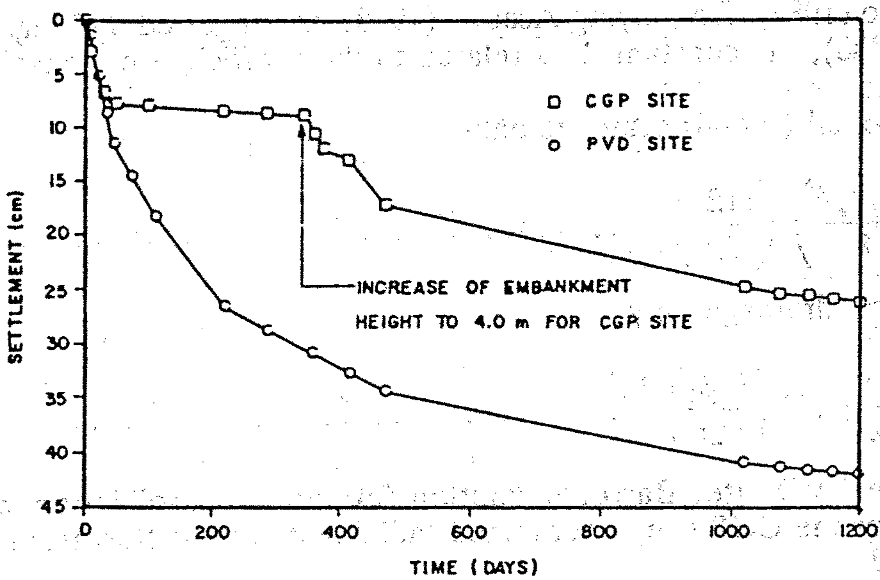

FIG. 16. Comparison of Maximum Surface Settlements for Both Sites

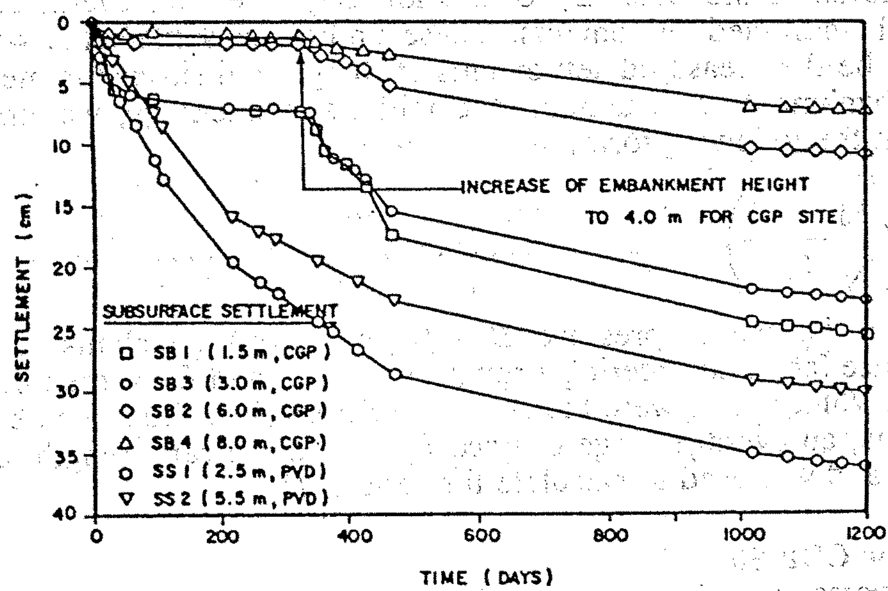

FIG. 17. Comparison of Maximum Subsurface Settlements for Both Sites

The data for back analysis were obtained from field and laboratory results. Correlations of these results were also made. For the PVD site, back analyzed parameters were applied to predict the soil behavior using oedometer, one-dimensional (Skempton and Bjerrum 1957; Asaoka 1978) methods. For the case of the CGP site, evaluation of settlement reductions was made by employing the equilibrium method (Aboshi et al. 1979) and design charts from the finite element formulation of Balaam et al. (1977).

Evaluation of c_h or c_v from Settlement Data

From time-settlement curves obtained from field measurements, the total settlements comprising immediate settlement ρ_i, primary consolidation, ρ_c, and secondary compression, ρ_{sec} were determined. The total settlement, ρ_t, consisting of immediate and consolidation settlement, was then calculated by subtracting the effects of secondary compression. Since only conventional oedometer tests were conducted in this study, the magnitude of secondary compression was estimated based on $C_\alpha = 0.04^* C_c$, as suggested by Mesri and Castro (1987). Employing Asaoka (1978), as suggested by Magnan and Deroy (1980), the constant β_1 is related to the coefficient of consolidation as follows:

For horizontal (radial) drainage only

$$c_h = \left(\frac{-D_e^2 F}{8}\right)\left(\frac{ln\beta_1}{\Delta t}\right) \qquad (13)$$

For vertical drainage only

$$c_v = \left(\frac{-5}{12}\right) H^2 \left(\frac{ln\beta_1}{\Delta t}\right) \qquad (14)$$

For the PVD site, Barron's solution for radial consolidation was employed; for the CGP site, a combined radial and vertical flow from (9) was computed.

Evaluation of Undrained Modulus (E_u) and E_u/s_u Ratio

The undrained modulus, E_u, of the soft clay layer was evaluated from the initial undrained settlements. These undrained settlements were assumed to be the measured settlements at the ground surface immediately after embankment construction. The initial settlement, ρ_i, was then expressed in the following form:

$$\rho_i = qBI\left(\frac{1-\nu^2}{E_u}\right) \qquad (15)$$

where q = the loading pressure; B = the width of the embankment; I = an influence factor considering a rough rigid base; and E_u = the undrained modulus. Values of E_u were then computed from (15). From the field vane test results, an average range of uncorrected vane shear strength, S_u, was obtained and was used to calculate the ratio E_u/S_u.

Results for CGP Site

The inverse analysis involves trial and error, especially at the CGP site. The complexity of three-dimensional flow, two-stage embankment loading, and the changing coefficients of consolidation are some of the factors that complicated the analyses. Using Asaoka (1978), the final settlement at the final stage of loading was 250.0 mm. A reanalysis from this study showed that the final stress from the 2.40-m-high embankment was within the overconsolidation range of the underlying soil. This analysis was supported by the rapid dissipation of excess pore pressure and by observed settlement magnitudes (Fig. 10). The coefficients of consolidation tend to be very high in the overconsolidated range as a result of the clay having very low compressibility for very small load increments (Bjerrum 1972). These observations were proven correct from consolidation test results and from back calculation of settlement data from the CGP site. Thus, in the back analysis, different c_h and c_v values were derived depending upon the applied load on the soil. Smaller applied load from the 2.40-m embankment height gave much higher coefficients of consolidation than the load from the 4.0-m-high embankment. These back analyzed parameters were then used in the prediction of the rate and amount of settlement, as shown in Fig. 18.

The initial settlements were calculated using the modified method of

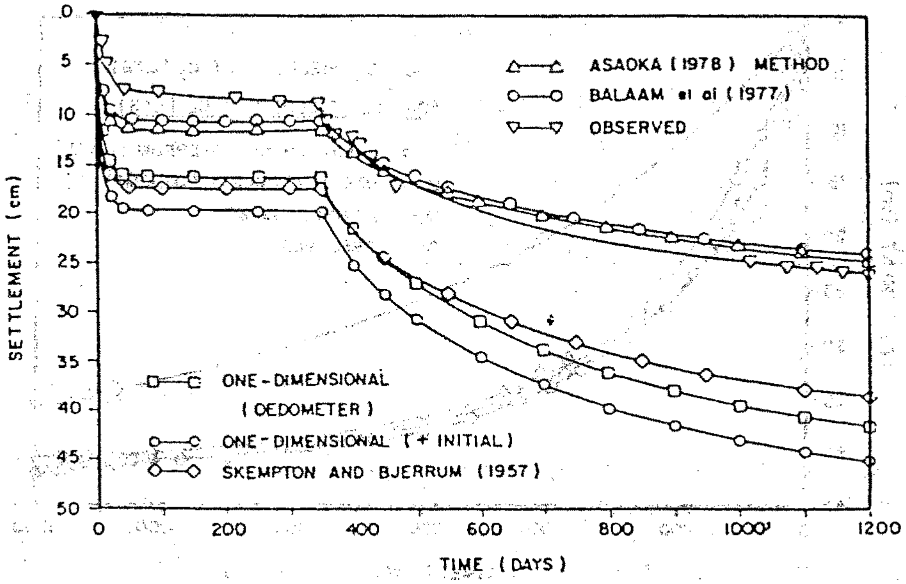

FIG. 18. Comparison Between Predicted and Observed Settlements for CGP Site

Janbu (Christian and Carrier 1978). In the absence of data, the smeared zone of the soil surrounding the granular piles during construction was taken into account by reducing to one-fifth the constructed final diameter of the granular piles as the effective diameter used in the analysis. This assumption is in accordance with the range tentatively recommended by Barksdale and Bacchus (1983) for the effective diameter of granular piles, which varies from one-half to one-fifteenth of the actual diameter. Asaoka (1978) and Balaam et al. (1977) appeared to be satisfactory in predicting the rate and amount of settlements, although overprediction during the initial stages of loading was observed. Excellent agreements were obtained at the final stage of consolidation using back-analyzed parameters from this loading. The equilibrium method using one-dimensional consolidation theory as well as the Skempton and Bjerrum (1957) method resulted in overestimation of the rate and amount of settlements.

Results of PVD Site

In the PVD site, some parameters were assumed and tried, then compared with the actual behavior of the subsoil. A number of trials were made based on different values of k_h/k_s. From these, a value of $k_h/k_s = 10$ was found to be the most appropriate for the case under consideration with $d_s/d_w = 2.5$, based on the equivalent diameter of the mandrel recommended by Hansbo (1987) and Rixner et al. (1986). The smear effect was found to be equally important, if not more, than the effect of drain spacing, which is always present. This, however, also depends on the chosen value of k_h/k_s, which has the dominant effect on the value of the back-calculated c_h. The wide variation in c_h could greatly affect the average degree of consolidation in the radial direction, $\bar{U}_h$, and hence the prediction of the rate of settlement. This analysis was based on the assumption that Asaoka's method can correctly predict the final settlement from one to two years of postconstruction settlement data, or when about 60% consolidation has already occurred. The back analyzed value of c_h was about 4.90 m^2/year.

Fig. 19 shows the comparison between the predicted and observed rates of settlement by different methods using the back-analyzed parameters. Except for the one-dimensional method (without initial settlement), generally all the methods predicted well the rate and amount of settlements.

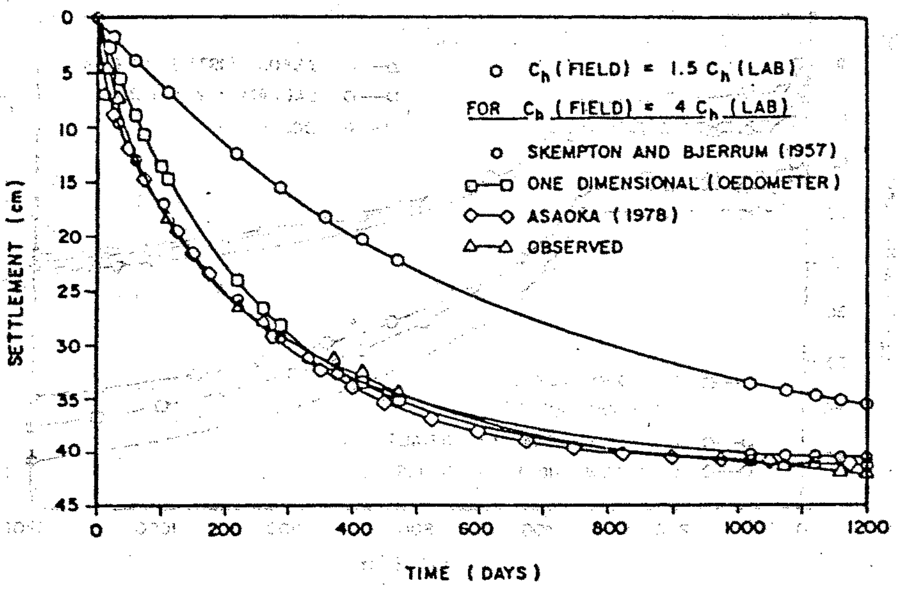

FIG. 19. Comparison Between Predicted and Observed Settlements for PVD Site

Without the inclusion of initial settlements, the one-dimensional method underestimated both the rate and amount of settlements at the initial stage of consolidation. Asaoka (1978) and Skempton and Bjerrum (1957) yielded very good predictions. The one-dimensional method with the inclusion of initial settlements overestimated the rate and amount of settlements.

From these analyses, it was found that c_h (field)/c_h (lab) ranged from 1.5–4 for the PVD site, while a wider range of 1.3–12 for c_v (field)/c_v (lab) was calculated for the CGP site, based on the final stage of loading and consolidation. The back-analyzed value of E_u was about 3,000 kPa, giving an average E_u/S_u ratio of 150, where S_u is the uncorrected vane shear strength. This E_u/S_u ratio confirmed the earlier results presented by Bergado and Khaleque (1986) and Bergado et al. (1990).

Conclusions

This study attempted to use inverse analysis to validate accepted theories by evaluating various parameters of the soft Bangkok clay that had been improved by prefabricated vertical drains and compacted granular piles. On the basis of this evaluation, the following conclusions can be drawn:

1. Based on the observed preconsolidation pressures, the final settlement for the PVD site was calculated at 360.0 mm, based on Asaoka (1978). The 90% and 100% consolidation settlements occurred at 430 days and 610 days, respectively, after the beginning of construction. For the CGP site, the expected final consolidation settlement based on the observational procedure of Asaoka (1978) was 250.0 mm. This magnitude implies reduction of about 30–35% in the total final settlement using granular piles. Also, the 90% and 100% consolidations occurred at about 880 days and 1,040 days, respectively, after the beginning of construction.
2. For the PVD site, excellent agreements were attained in the predicted rate and amount of settlements using back-analyzed parameters by the methods of Asaoka (1978) and Skempton and Bjerrum (1957). At the CGP site, good agreements for the rate and amount of settlement resulted from the adoption of Asaoka (1978) and Balaam et al. (1977), except at the initial stage of loading.

3. The back-analyzed c_h (field)/c_h (lab) for the PVD site ranged from 1.5–4.0, with a back-analyzed c_h of 4.90 m²/year and a back-analyzed E_u/S_u ratio averaging 150. For the CGP site, the range of back-calculated c_v (field)/c_v (lab) was about 1.3–12, based on the final stage of loading and consolidation.

4. For the PVD site, a value of k_h/k_s = 10 was found to be most appropriate with d_s/d_w = 2.5. For the CGP site, the effective diameter of the granular piles was taken as one-fifth of the constructed final diameter to account for the smeared zone.

APPENDIX I. REFERENCES

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Appendix II. Notation

The following symbols are used in this paper:

a = thickness of band-shaped drains;
a_s = area replacement ratio;
B = width of embankment;
b = width of band-shaped drains;
C_α = coefficient of secondary compression;
c_h = horizontal coefficient of consolidation;
c_v = vertical coefficient of consolidation;
D_e = diameter of equivalent soil cylinder;
d_s = diameter of smeared zone;
d_w = equivalent diameter of drain;
F = $F(n) + F_r + F_s$;
F_r = factor to account for well resistance;
F_s = factor to account for smear;
$F(n)$ = factor to account for drain spacing;
H = thickness of treated ground;
I = influence factor;
k_h = permeability along horizontal direction;
k_s = permeability at smeared zone;
L = drain length;
n = stress concentration factor;
q = loading intensity;
q_w = discharge capacity of drain;
r = radial distance;
S_t = settlement of composite ground;
$\bar{U}$ = combined average degree of consolidation due to vertical and horizontal drainage;
$\bar{U}_h$ = average degree of consolidation due to horizontal drainage;
$\bar{U}_v$ = average degree of consolidation due to vertical drainage;
u = excess pore pressure due to radial flow;
z = distance from open end of drain;
β_1 = slope of settlement records;
Δt = time interval;
ν = undrained Poisson ratio;

ν' = drained Poisson ratio;
ρ_c = primary consolidation settlement;
ρ_i = immediate settlement;
ρ_n = settlement at time t_n;
ρ_t = net total settlement;
ρ_{sec} = secondary settlement;
σ = average loading intensity;
σ_c = stress intensity in cohesive soil; and
σ_s = stress intensity in granular piles.