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Reinforcement of soft Bangkok clay using granular piles

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ABSTRACT: To study the effects of granular piles on the settlement of the soft Bangkok clay, a full scale embankment was constructed at the campus of the Asian Institute of Technology. Fully penetrating granular piles of 0.30 m in diameter, were arranged in triangular pattern at a spacing of 1.50 m and a 2.40 m high, well-instrumented embankment was constructed. To compare the settlement performance of the embankment with the nearby existing 4.0 m high test embankment on vertical drains, the height of the embankment was added up to 4.0 m after 345 days. Results show that after 900 days, the settlement of the embankment on granular piles was about 62% of the settlement observed on vertical drains. Comparison with past investigations indicates that granular piles reduced the settlement of soft clay foundations by as much as 20 to 40%. This implies that granular piles function as reinforcement to the clay rather than drains. The stress concentration factor measured in the granular pile foundation decreases with the increasing applied load.

1. INTRODUCTION

The use of granular piles as a soil improvement technique, is preferable when moderate increase in bearing capacity and/or moderate reduction, yet uniform rate of settlements, are required for foundations on soft clays. Bergado et al (1984b, 1987b) indicated that the bearing capacity of the soft Bangkok clay using granular piles increased by 4 times as much, the total settlements reduced by at least 30% and the slope stability factor of safety increased by at least 25%. Almost equal vertical strains were measured and the stress concentration varied from 2 to more than 5 between the pile and the surrounding clay.

To further study their effects on the settlement of a soft clay foundation, a full scale test embankment, 2.4 m high was constructed by Sim (1986) on a granular pile-improved foundation, and was raised to a height of 4.0 m by Panichayatum (1987) to provide a meaningful basis of comparison with the performance of the nearby test embankment constructed on a Mebra vertical drain-improved soil by Singh (1986). This paper presents the behaviour of the two test embankments to compare their effects on the settlement performance of the soft clay foundation and to assess the most suitable soil improvement techniques in the soft Bangkok clay. This study is the first of its kind in the subsiding environment in the Chao Phraya Plain in Thailand.

2. SITE LOCATION AND SOIL PROFILE

The test site is located inside the campus of the Asian Institute of Technology, 42 km north of Bangkok, Thailand. The soil profile is given in Fig.1. The uppermost subsoil is divided into 3 distinct layers as follows:

(a) heavily overconsolidated, weathered clay layer of about 2.0 m thick with reddish-brown color;

(b) grayish soft clay layer underlying the weathered clay of about 6.0 m thick; and

(c) highly overconsolidated reddish-brown stiff clay layer of about 6.0 m thickness.

The groundwater table varies with the season and an average value of 1.50 m below

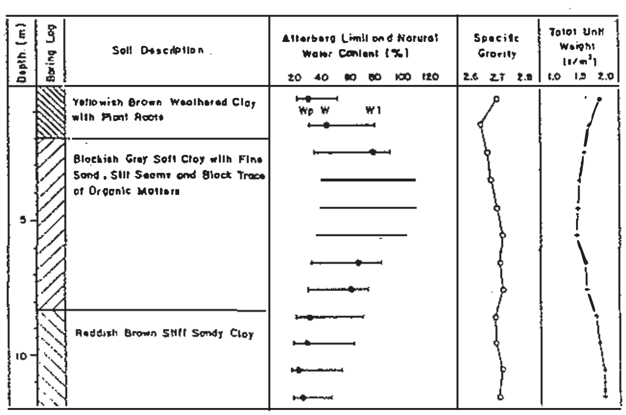

Fig.1 Soil profile and basic soil properties at the site.

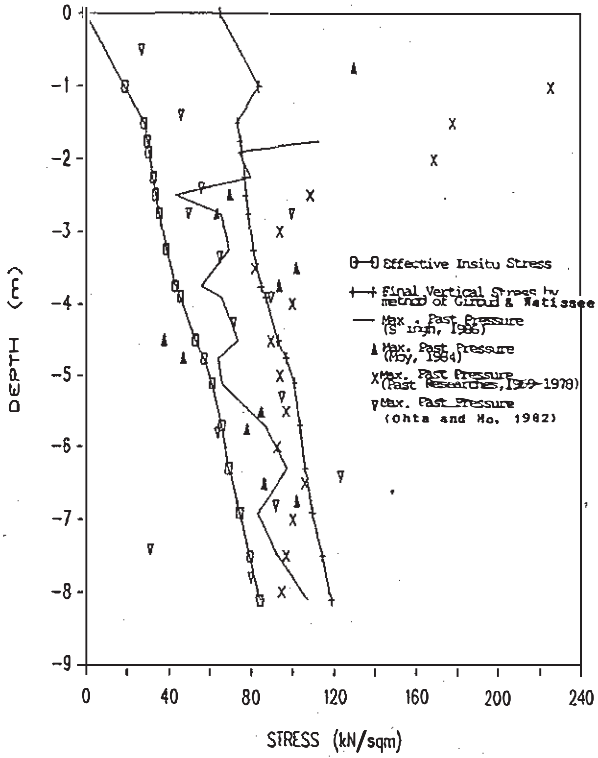

Fig. 2 Effective overburden and maximum past pressure beneath the test embankment.

the ground surface is assumed. From the bore-hole samples, the presence of fine sands and silt lenses were observed in the soft clay layer with few decomposed organic matters. Bergado and Danzuka (1988) studied the macro-fabric of AIT subsoil and obtained 2 domains, namely: (a) almost homogeneous clay layer with scattered fine sand and silt lenses (mean = 0.2 % by area) from 2 to 7 m depth; (b) almost heterogeneous clay layer containing higher density of fine sand and silt lenses (mean = 20 %) from 7 to 9 m depth.

3. SOIL PROPERTIES

The index properties of the subsoil are shown together with the soil profile in Fig. 1. The maximum past pressure, p_c, are plotted in Fig. 2, together with the values obtained from previous investigations (Ohta and Ho, 1982; Moy, 1984). The horizontal coefficient of consolidation C_h was obtained from consolidation tests with samples cut perpendicular to the horizontal direction. The variations of the vertical and horizontal coefficients of consolidation, C_v and C_h, respectively, are plotted in Fig. 3, showing that C_h is about 3 to 4 times larger than C_v. The resulting coefficients of vertical and horizontal permeabilities were found to be higher than the values obtained from past studies (Ohta and Ho, 1982).

4. THE SUBSIDING ENVIRONMENT

The campus of the Asian Institute of Technology (AIT) is situated on a flat, deltaic marine deposit called the Chao Phraya Plain in Central Thailand. The ground subsidence of the Chao Phraya Plain is caused by the excessive groundwater pumping for water supply through deep wells (AIT, 1982). Presently, in AIT campus alone, 1,500 cubic meters of water per day is pumped through deep wells from 200 m depth (Bergado et al, 1984a). The ground subsidence effects include differential settlements between structures on shallow foundations and on pile foundations, differential settlements in pavements and pathways, and localized ground depressions in open field areas. Ground movements and building settlements were monitored from precision levelling surveys (Bergado et al, 1987b). An average subsidence rate of 2.4 cm and a maximum of 8 cm per year were estimated. Nutalaya and Rau (1982) calculated an average subsidence rate of 6 cm per year and a maximum of 10 cm per year in Bangkok area.

5. SOIL IMPROVEMENT BY GRANULAR PILES

Granular piles are preferable when moderate increase in bearing capacity and/or a moderate reduction of settlements are required for foundations on soft clay subsoils. Moreover, their recharging effects to minimize the problems caused by differential settlements such as cracking of concrete and asphalt pavements, as well as their effects of increasing the slope stability factor of safety, has assumed position of greater significance and interest for the past years. Barksdale and Bacchus (1983) presented results of their numerical models and case studies on various projects. Enoki (1987) presented the comparison of the experimental and analytical results on the behavior of composite ground

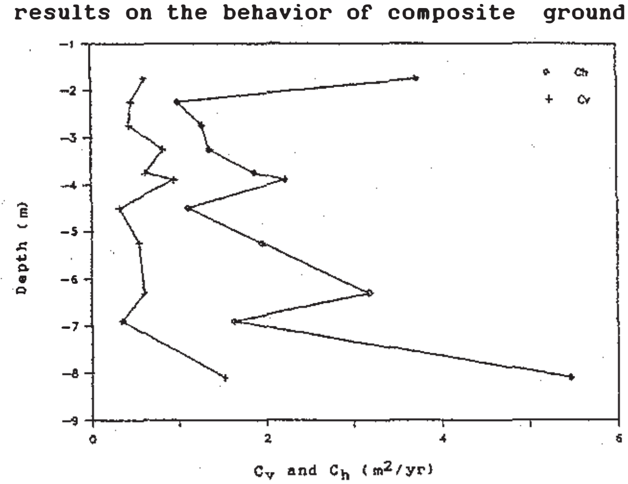

Fig. 3 Variation of C_v and C_h with depth.

consisting of granular pile and the surrounding clay.

The gravel materials used to construct the granular piles consisted of whittish-gray, crushed limestones. The gravel was poorly graded with a maximum size of 20 mm. The compacted density varied from 1.70 to 1.81 t/m^3. The friction angles obtained from direct shear tests varied from 39 to 45 degrees. The gravel materials and the procedure of installation of the granular piles were similar to that used by Bergado and Lam (1987). The triangular pattern was used. Full scale load tests were made by Bergado et al (1987b) using rigid plate and a stress concentration factor of more than 5 was calculated. However, in the test embankment, a stress concentration factor of 2 was measured. Previous studies have shown that the bearing capacity of the clay subsoil increased by as much as 4 times in the improved ground (Bergado et al, 1987a).

6. TEST EMBANKMENT ON GRANULAR PILES

A full scale test embankment was constructed on granular pile foundation 10 days after the installation of the last pile. The granular piles were arranged in triangular pattern with spacing of 1.50 m. The granular piles have diameter of 0.30 m and length of 8.0 m fully penetrating the soft clay layer. Prior to the construction of the test embankment, the test site was instrumented and the initial readings of all instruments were recorded. The field instruments consisted of piezometers, settlement plates, inclinometers, etc. The embankment was rectangular in plan with a first stage height of 2.4 m and a second stage height of 4.0 m. It has dimensions of 13.7 m by 15.7 m at the base and 1.3 m by 9.3 m at the top. A drainage blanket of 0.25 m thick consisting of clean sand was laid at the base on top of the granular piles. The embankment was compacted in layers by a light vibrating plate tamper and was found to have an average density of 2.0 t/m^3. The cross-section of the test embankment is shown in Fig. 4. For the second stage, the embankment height was increased to 4.0 m after 345 days.

7. SOIL IMPROVEMENT BY VERTICAL BAND DRAINS

The principal alternative to large diameter drain is the much smaller band-shaped drains first employed by Kjellman (1948). A prefabricated band drain consists of a central core, whose function is to act as free draining channel and to withstand buckling stresses, enclosed by a thin filter sleeve, which prevents the fine soil particles from entering the central core but allows free entry of pore water into the core. The band drains are manufactured from man-made polymer fabrics such as polyethelenes, polypropylenes, etc. Currently, there are a number of available band drains. Mebra drains were used in this study.

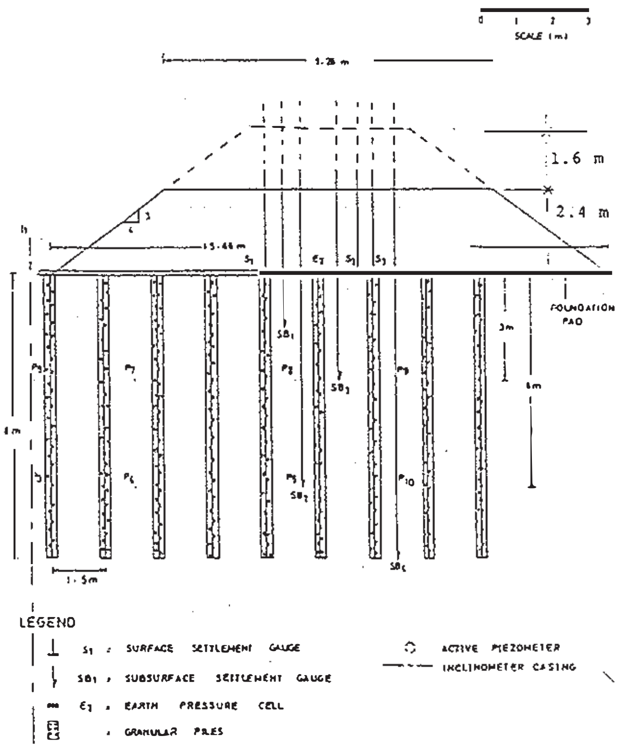

Fig. 4 Cross-section of test embankment with granular piles and location of field instruments.

Barron (1948) presented a solution to the problem of the consolidation of a soil cylinder containing a central drain. The problem formulated in terms of polar coordinates yield the following differential equation:

$$\frac{\partial u}{\partial t} = C_h \left[\frac{1}{r}\frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right] \qquad (1)$$

where C_h is the horizontal coefficient of consolidation, r is the radial coordinate, u is the excess pore water pressure and t represents time. Barron (1948) suggested two types of solutions based on either free strain or equal vertical strain. Jamiolkowski et al (1983) compared these two solutions and indicated that almost similar values of degree of consolidation were obtained for the usual values of drain spacing and soil compressibility. This justifies the use of the less complicated equal vertical strain solution. The average degree of consolidation with respect to the radial flow is given as:

$$U_h = 1 - \exp(-8\, T_h/F(n)) \qquad (2)$$

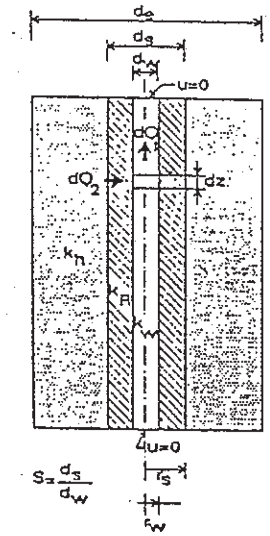

Fig. 5 Consolidation problem with well and smear effects (after Hansbo, 1979).

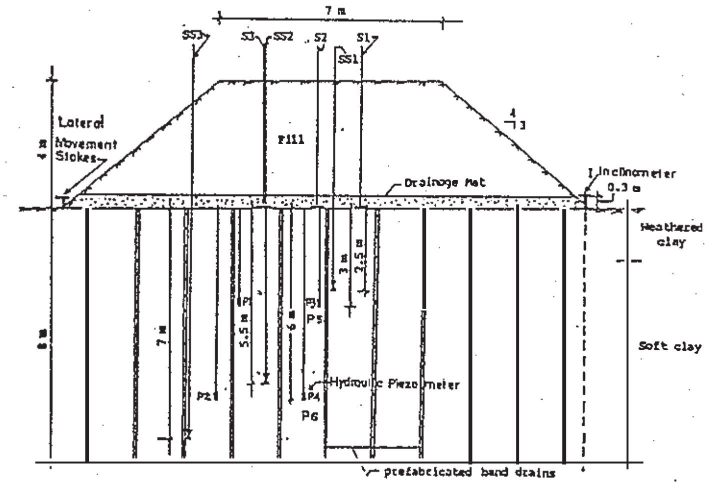

Fig. 6 Cross-section of test embankment with drains and location of field instruments.

In reality, the installation of the drain causes a disturbance around the drain depending on the size of the mandrel, the manner of installation, and the macrofabric of the clay which is called the smear effeot. Also, headloss ooours in the flow along the drain due to the well-resistance. Solutions have been presented by Barron (1948) considering the effects of smear and well-resistance. Hansbo (1979) presented simplified solutions considering the effects of smear and well-resistance. The remoulding around the drain is included in the analysis by assuming an annulus of smeared clay with an internal diameter, d_s, surrounding the drain. Within the annulus, the soil is strongly remoulded and its coefficient of permeability, K_h', is lower than K_h, which characterized the original clay. Combining the effects of smear and well-resistance, F(n) takes the form:

$$F(n) = \ln(n/s) - (K_h/K_h') \ln(s) - \frac{3}{4} + Z(2\ell - Z)\, K_h/q_w \qquad (3)$$

where n is the ratio of the diameter of equivalent soil cylinder, d_e, to the equivalent nominal diameter of the drain, d_w, s is the ratio of smeared diameter, d_s, to the drain diameter, d_w, and the other terms are defined in Fig. 5.

8. TEST EMBANKMENT ON VERTICAL DRAINS

The test site is located inside the campus of the Asian Institute of Technology, 42 km north of Bangkok. The soil profile is as given in Fig. 1. The soil properties have been given in Figs. 2 and 3.

Before the installation of the drains, trenches of 0.6 m wide and 1.6 m deep were excavated in the topmost weathered clay layer to eliminate the arching effect. Mebra drains were installed in triangular pattern at 1.5 m center to center spacing. The drains were installed by means of a special mandrel down to 8.0 m depth to fully penctrate the soft Bangkok clay layer at the site. The size of the mandrel was minimized to reduce the smear effect. The rectangular shaped mandrel had inner dimensions of 2.8 by 13.3 cm and outer dimensions of 4.5 by 15.0 cm enough to contain the 0.3 by 9.5 cm Mebra drains. Disposable shoes were installed at the bottom of each drain for anchorage. Field instruments such as piezometers, surface and subsurface settlement gages, inclinometers, and lateral movement stakes were installed to monitor the behaviour of the embankment foundation. Subsequently, a 4.0 m high test embankment with side slopes of 4H: 3V was constructed. The base dimensions consist of 14.6 by 16.6 m while the top dimensions were 5.0 by 7.0 m. The embankment construction lasted for one month. The vertical stress at the ground surface was calculated to be 65.0 kN/m^3 due to the embankment loading. The detailed locations of the instrumentation and the typical section of the embankment is shown in Fig. 6. The result of this full scale loading test have been published elsewhere (Bergado et al, 1988).

9. COMPARISON OF SETTLEMENTS OF EMBANKMENTS ON GRANULAR PILES AND ON VERTICAL DRAINS

The observed settlements of test embankments on granular piles and on vertical drains after subtracting the effects of ground subsidence are shown in Fig. 7.

At a height of 2.4 m, the maximum observed settlements for embankment on granular piles, 110 days after construction, was

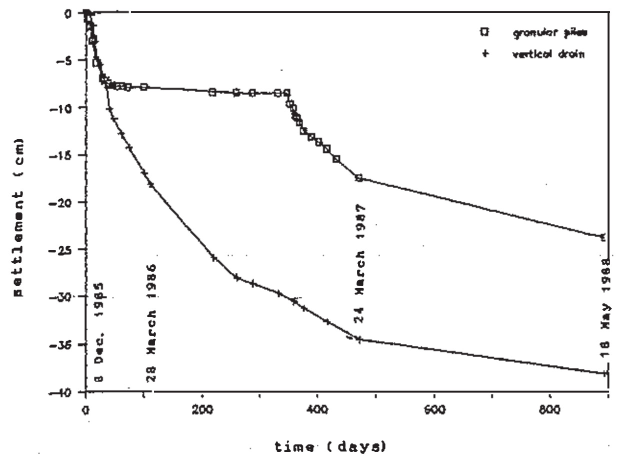

Fig. 7 Maximum observed settlements for both embankments after deleting subsidence effect.

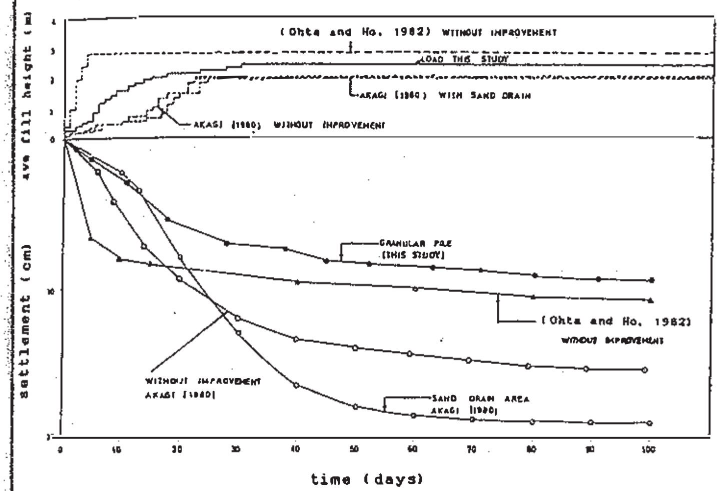

Fig. 8 Comparison of settlement records of embankment on granular piles with previous investigations.

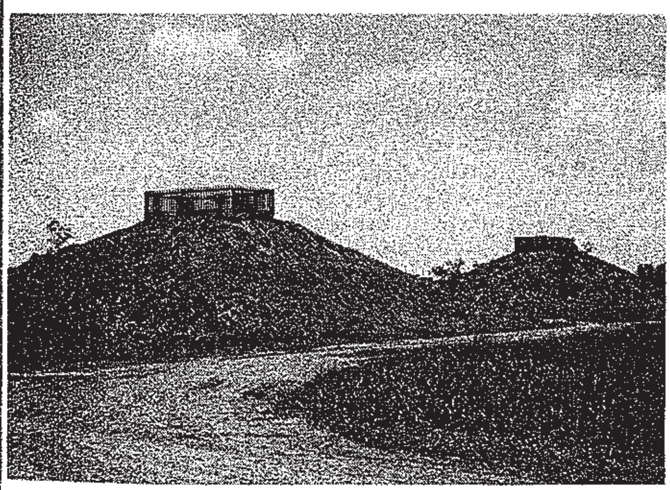

Fig. 9 Test embankments on granular piles and plastic band drains.

about 44% of the maximum settlement observed for vertical drains. After 345 days, the settlement increment of the embankment on granular piles was closed to zero, with a maximum recorded settlement at the center surface settlement gauge of 8.65 cm, which is 29% of the maximum observed for the embankment on vertical drains.

After filling up of the embankment to reach 4.0 m height, it was observed that the settlements recorded, 470 days after the start of construction, was about 51% of the maximum observed settlement for vertical drains. A maximum surface settlement of 23.86 cm was then recorded about 900 days after construction for granular piles as compared to 38.20 cm for vertical drains. This amounted to about 62% settlement of granular piles embankment relative to the embankment on vertical drains.

Granular piles then, seemed to have reduced the settlement as compared to the vertical plastic band drains. This is probably due to the reason that granular piles are stiffer than the surrounding soil, such that their effect seems to reinforce the soil and support the embankment rather than the usual concept of providing drainage to the subsoil to accelerate consolidation.

10. COMPARISON WITH PAST INVESTIGATIONS

The time-settlement curves in this study from both test embankments were compared with the corresponding observations of the past studies of Akagi (1979) and Ohta and Ho (1982). As the settlements recorded in this study were made under different stress levels, it may not be appropriate to compare settlement magnitudes directly. However, relative comparisons can be made. As shown in Fig. 8 for instance, it can be seen that at 71 days after embankment construction, the embankment on granular piles with 2.4 m height settled nearly 40% less than that of the embankment of Akagi (1979) without improvement and about 20% less of the embankment of Ohta and Ho (1982). Thus, the granular piles seemed to function as reinforcement in the clay rather than drains. Figure 9 shows the two test embankments on improved soft Bangkok clay.

11. CONCLUSIONS

1) The settlement observed for the test embankment on granular piles was about 62% of the maximum observed for the embankment on vertical drains. Hence, granular piles seem to reduce settlements by acting as soil reinforcement to support the embankment, rather than providing drainage to accelerate the consolidation process.

2) Relative comparison with past investigations indicates that granular piles decrease the settlement of soft clay foundations by as much as 20 to 40%. This confirms the idea that granular piles function as reinforcement to the clay rather than drains.
3) The stress concentration factor measured in the granular pile foundation ranged from 2 to 5 and was found to decrease with the increasing applied load.

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