

THE GRANULAR PILE: ITS PRESENT STATE AND FUTURE PROSPECTS FOR IMPROVEMENT OF SOFT BANGKOK CLAY

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SYNOPSIS

The current state-of-the-art of granular pile as soft ground improvement technique including the methods of construction as well as the engineering behavior of the composite ground was reviewed. In addition, the applicability of granular piles as an alternative scheme of foundation improvement of approach embankments to bridges and viaducts on the soft and subsiding ground was evaluated and investigated in the light of actual performance of a full scale test embankment and full scale load tests at the Asian Institute of Technology (AIT) campus. Moreover, a case history of full scale test involving sand compaction (granular) piles is also discussed. Based on the observations made in this study, it is recommended that although sufficient knowledge is currently attained for successful design of granular pile projects, more laboratory full scale model tests and fully instrumented test embankments (unreinforced or reinforced) have to be constructed for more understanding and confidence of foundations improved with granular piles especially the magnitude of outward lateral and horizontal movements.

INTRODUCTION

The Central Plain of Thailand is situated on a flat deltaic-marine deposit with north-south dimension of about 300 km and an east-west width of about 200 km. The presence of about 6-7 m thick deposits of soft Bangkok clay and the effects of ground subsidence due to excessive pumping of groundwater create foundation problems to earth structures and infrastructures especially at approach embankments (transition units) to bridges and viaducts. Basically, the problem is due to the occurrence of differential movements between the pile-supported bridges and viaducts and the ground-supported approach embankments. To

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mitigate such natural hazard, granular piles are proposed as appropriate and viable foundation treatment.

Granular piles are composed of compacted sand or gravel inserted into the soft clay foundation by displacement method. The term "granular piles" used in this paper refers to these compacted gravel and/or sand piles. It also refers to those known as stone columns. The ground improved by compacted granular piles is termed as composite ground. When loaded, the pile deforms by bulging into the subsoil strata and distributes the stresses at the upper portion of the soil profile rather than transferring the stresses into the deeper layers, thus causing the soil to support it. As a result, the strength and bearing capacity of the composite ground can be increased and the compressibility reduced. In addition, lesser stress concentration is developed on the granular piles. The component material being granular and with higher permeability, granular piles could also accelerate the consolidation settlements and reduce the post construction settlements.

METHODS OF GRANULAR PILE CONSTRUCTION

Various methods for installation of granular piles have been used all over the world depending on their proven applicability and availability of equipment in the locality. The following common methods will be briefly described with their corresponding references.

Vibro-Compaction Method

The vibro-compaction method is used to improved the density of cohesionless, granular soils using a vibroflot which sinks in the ground under its own weight and with the assistance of water and vibration (Baumann and Bauer, 1974; Engelhardt and Kirsch, 1977). After reaching the predetermined depth, the vibroflot is then withdrawn gradually from the ground with subsequent addition of granular backfill thereby causing compaction. Figure 1 illustrates the steps in vibro-compaction process. The range of grain size distribution of soils suitable for this method is shown in Fig. 2.

Vibro-Replacement Method

The vibro-replacement method is used to improved cohesive soils with more than 18% passing no. 200 U.S. standard sieve. The equipment used is similar as that for vibro-compaction. The vibroflot sinks into the ground under its own

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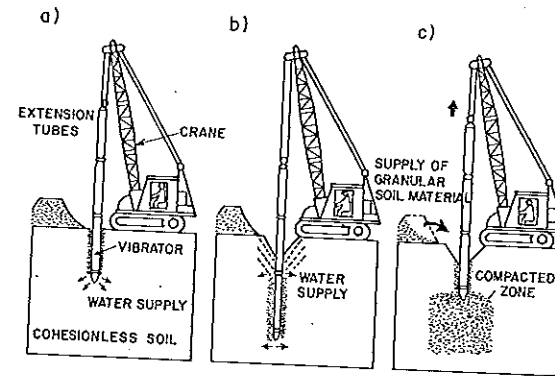


Fig. 1 The Vibro-Compaction Process (after Baumann and Bauer, 1974)

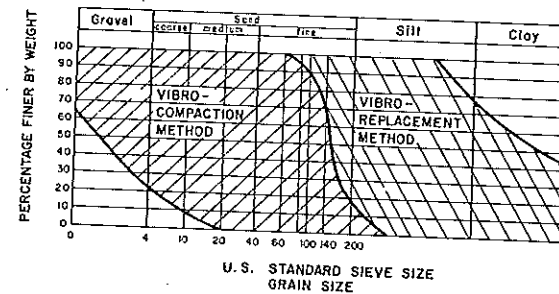


Fig. 2 Range of Soils Suitable for Vibro-Compaction and Vibro-Replacement Methods (after Baumann and Bauer, 1974)

weight assisted by water or air jets as a flushing medium until it reaches the predetermined depth (Baumann and Bauer, 1974; Engelhardt and Kirsch, 1977). The process is illustrated in Fig. 3. The grain size range of the suitable soils for this treatment is also shown in Fig. 2. The method can be carried out either with the wet or dry process. In the wet process, a hole is formed in the ground by jetting a vibroflot down to the desired depth with water. When the vibroflot is withdrawn, it leaves a borehole of greater diameter than the vibrator. The uncased hole is flushed out and filled in stages with 12 mm - 75 mm size imported gravel. The densification is provided by an electrically or hydraulically actuated vibrator

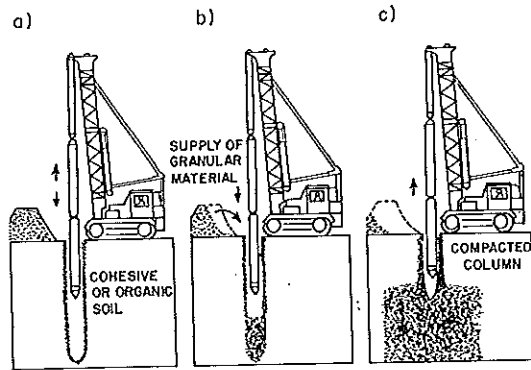


Fig. 3 The Vibro-Replacement Process (after Baumann and Bauer, 1974)

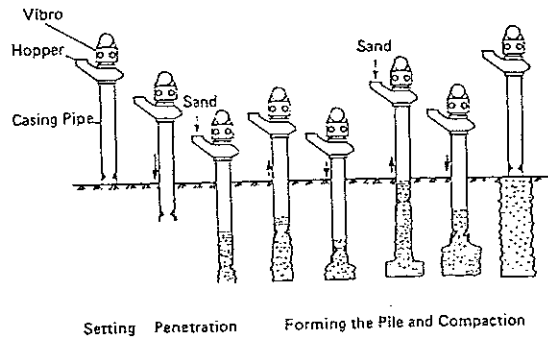


Fig. 4 The Vibro-Compozer Method (after Aboshi and Suematsu, 1985)

near the bottom of the vibroflot. The wet process is generally suited for unstable borehole and a high ground water table. The main difference between the dry process and the wet process is the absence of jetting water during the initial formation of hole in the former. In the dry process, the borehole must be able to stand open upon extraction of the vibroflot, which requires the soil under treatment to have an undrained shear strength of more than 40 kN/m^2 and a relatively low ground water level.

Vibro-Compozer Method

The method is popularized in Japan and is used for stabilizing soft clays in the presence of high ground water level (Aboshi et al., 1979; Aboshi and Suematsu,

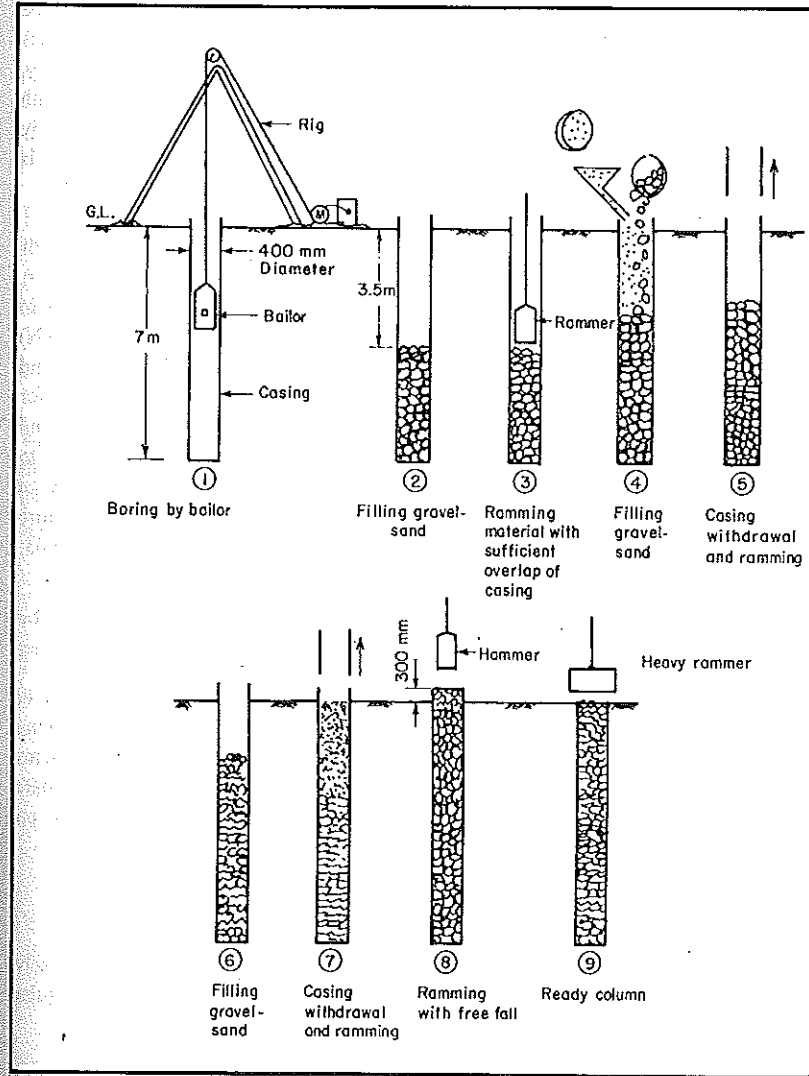


Fig. 5 The Cased-Borehole Method (after Datye and Nagaraju, 1975)

1985; Barksdale, 1981). The installation procedures are illustrated in Fig. 4. The resulting pile is usually termed as sand compaction pile. The sand compaction piles are constructed by driving the casing pipe to the desired depth using a heavy, vertical vibratory hammer located at the top of the pipe. The casing is filled with specified volume of sand and the casing is then repeatedly extracted and partially redriven using the vibratory hammer starting from the bottom. The process is repeated until a fully penetrating compacted granular pile is constructed.

Cased-Borehole Method

In this method, the piles are constructed by ramming granular materials in the prebored holes in stages using a heavy falling weight (usually of 15 to 20 kN) from a height of 1.0 to 1.5 m (Datye and Nagaraju, 1975; Datye, 1978; Datye and Nagaraju, 1981; Bergado et al., 1984). The method is a good substitute for vibrator compaction considering its low cost. However, disturbance and subsequent remolding by the ramming operation may limit its applicability to sensitive soils. The method is useful in developing countries utilizing only an indigenous equipment in contrast to the methods described above which require special equipment and trained personnel (Rao, 1982; Ranjan and Rao, 1983; Ranjan, 1989). The installation process is illustrated in Fig. 5.

ENGINEERING BEHAVIOR OF COMPOSITE GROUND

The performance of composite ground is best investigated in terms of ultimate bearing capacity, settlement, and general stability. In the following sections, basic relationships of the composite ground as well as failure mechanisms of granular piles on homogeneous soft clay are first described and the ultimate bearing capacity, settlement, and stability of the composite ground based on experimental and analytical studies are then presented.

Unit Cell Concept

The tributary area of the soil surrounding each granular pile is closely approximated by an equivalent circular area. For an equilateral triangular pattern of granular piles, the equivalent circle has an effective diameter of:

$$D_e = 1.05S \tag{1}$$

while for a square pattern,

$$D_e = 1.13S \tag{2}$$

where S is the spacing of granular piles. The equilateral triangular pattern gives the most dense packing of granular piles in a given area. The resulting cylinder of composite ground with diameter D_e enclosing the tributary soil and one granular pile is known as the unit cell.

Area Replacement Ratio and Stress Concentration Factor

Figure 6 illustrates the area replacement as well as the stress concentration in the granular pile. The area replacement ratio is defined as the ratio of the granular pile area over the whole area of the equivalent cylindrical unit within the unit cell and expressed as:

$$a_s = \frac{A_s}{A_s + A_c} \tag{3}$$

where A_s is the horizontal area of a granular pile and A_c is the horizontal area of the clayey ground surrounding the pile. The area replacement ratio can also be expressed in terms of the diameter (D) and spacing (S) of the granular pile as follows:

$$a_s = c_1 (D/S)^2 \tag{4}$$

where c_1 is a constant depending upon the pattern of granular piles used; for square pattern $c_1 = \pi/4$ and for equilateral triangular pattern $c_1 = \pi / (2 \sqrt{3})$.

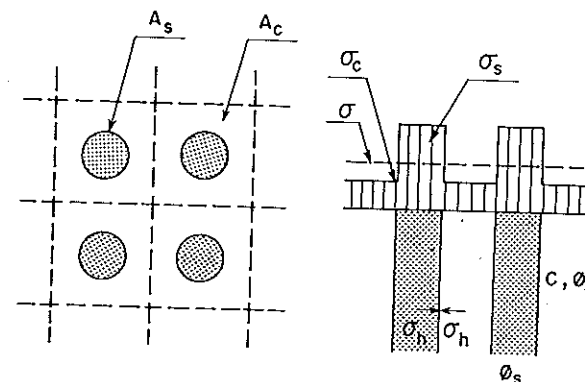


Fig. 6 Diagram of Composite Ground

When the composite ground is loaded, studies (e.g. Greenwood, 1970; Aboshi et al., 1979; Goughnour and Bayuk, 1979; Balaam et al. 1977) indicated that concentration of stress occurs in the granular pile accompanied by the reduction in stress which occurs in the surrounding less stiff clayey soil (Fig. 6). This can be explained by the fact that, when loaded, the vertical settlement of the granular pile and the surrounding soil is approximately the same (Vautrain, 1977) causing the occurrence of stress concentration in the granular pile which is stiffer than the surrounding cohesive or loose cohesionless soil. The distribution of vertical stress within the unit cell can be expressed by a stress concentration factor defined as:

$$n = \sigma_s / \sigma_c \quad (5)$$

where σ_s is the stress in the granular pile and σ_c is the stress in the surrounding cohesive soil. The magnitude of stress concentration also depend on the relative stiffness of the granular pile and the surrounding soil. The variation of stress concentration factor with area replacement ratio compiled by Barksdale and Bachus (1983), Vautrain (1977), Goughnour (1983), and Parsons and Co., Inc. (1980) ranged from 2 to 5. Meanwhile, Aboshi et al. (1979) and Bergado et al. (1987) obtained higher stress concentration factor as much as 9. The higher value obtained by Bergado et al. (1987) was probably due to the high rigidity of the plates used during the load tests. From full scale test embankment observations on soft Bangkok clay at low area replacement ratio of 0.06, the stress concentration factor of 2 was obtained and was found to decrease to 1.45 with the increasing applied load (Bergado et al., 1988). The average stress over the unit cell area corresponding to a given area replacement ratio is expressed as:

$$\sigma = \sigma_s a_s + \sigma_c (1-a_s) \quad (6)$$

The stresses in the pile and the clay using the stress concentration factor are:

$$\sigma_s = n\sigma / [1 + (n-1)a_s] = \mu_s \sigma \quad (7)$$

$$\sigma_c = \sigma / [1 + (n-1)a_s] = \mu_c \sigma \quad (8)$$

where μ_s and μ_c are the ratio of stress in the pile and clay, respectively, to the average stress over the unit cell area.

Failure Mechanisms

In practice, granular piles are usually constructed fully penetrating a soft soil layer overlying a firm stratum. It may be constructed also as floating piles with their tips embedded within the soft clay layer. Granular piles may fail

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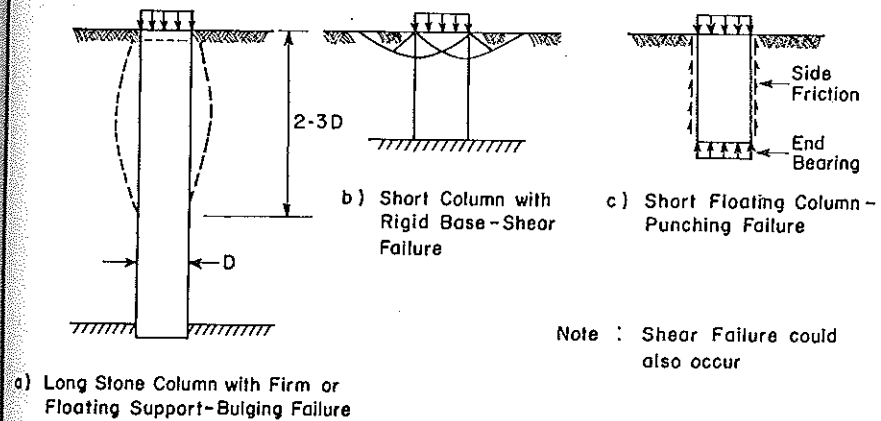


Fig. 7 Failure Mechanisms of a Single Granular Pile in a Homogeneous Soft Layer (after Barksdale and Bachus, 1983)

individually or as a group. The failure mechanisms for a single pile are illustrated in Fig. 7, indicating the possible failures as: a) bulging, b) general shear, and c) sliding. For pile groups, additional failure mechanisms such as lateral spreading and shear failure across the granular pile cross-section may occur.

Ultimate Bearing Capacity of Single, Isolated Granular Pile

For single, isolated granular piles, the most probable failure mechanism is bulging failure. This mechanism develops whether the tip of the pile is floating in the soft soil or fully penetrating and bearing on a firm layer. The lateral confining stress which supports the granular pile is usually taken as the ultimate passive resistance which the surrounding soil can mobilize as the pile bulges outward. Most of the approaches in predicting the ultimate bearing capacity of a single, isolated granular pile has been developed based on the above assumption. Table 1 tabulates the different methods to estimate the ultimate bearing capacity corresponding to bulging, general shear, and sliding modes of failure as presented by Aboshi and Suematsu (1985). A relationship between ultimate bearing capacity and area replacement ratio is shown in Fig. 8. The relationship between internal friction angle of granular material, strength of the surrounding clay, and the ultimate bearing capacity of single granular pile is shown in Fig. 9.

Table 1 Estimation of Ultimate Bearing Capacity (after Aboshi and Suematsu, 1975)

| MODE OF FAILURE | DERIVED FORMULA | REFERENCE |
|-----------------|---|---|
| Bulging | $q_{ult} = (\gamma_c Z K_{pc} + 2C_o \gamma_c K_{pc}') \frac{1 + \sin \phi_s}{1 - \sin \phi_s}$ | Greenwood (1970) |
| | $q_{ult} = (F_c' C_o + F_q' Q_o) \frac{1 + \sin \phi_s}{1 - \sin \phi_s}$ | Vesic (1972) Datye and Nagaraju (1975) |
| | $q_{ult} = (\sigma_{vo} + 4C_o) \frac{1 + \sin \phi_s}{1 - \sin \phi_s}$ | Hughes and Withers (1974) |
| | $q_{ult} = \frac{1 + \sin \phi_s}{1 - \sin \phi_s} (4C_o + \sigma_{vo} + K_o q_s) (W/B)^2 + [1 - (W/B)^2] q_s$ | Madhav et al. (1979) |
| General Shear | $q_{ult} = C_o N_c + (1/2) \gamma_c B N_\gamma + \gamma_c D N_q$ | Madhav and Vitkar (1978) |
| | $q_{ult} = 2A_s (K_{pc} q_o + 2C_o \gamma_c K_{pc}') + (1/K_{sa}) [3d_s K_{pc} \gamma_c \{1 - (3d_s/2L)\}]$ | Wong (1975) |
| | $q_{ult} = (1/2) \gamma_c B \tan^2 \psi + 2C_o \tan^2 \psi + 2(1-a_s) C_o \tan \psi$ $\psi = 45^\circ + \frac{\tan^{-1} (\mu_s a_s \tan \phi_s)}{2}$ | Barksdale and Bachus (1983) |
| Sliding Surface | $\tau = (1-a_s) C_o + (\gamma_c Z + \mu_s \sigma_s) a_s \tan \phi_s \cos^2 \theta$ $\mu_s = \frac{n}{1 + (n-1) a_s}$ | Aboshi et al. (1979) |

Note: Refer to APPENDIX for Notations

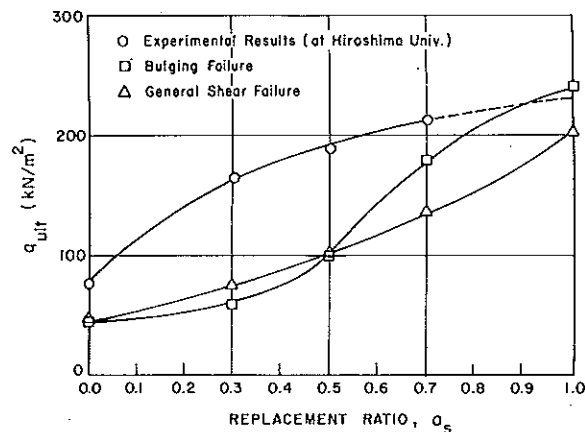


Fig. 8 Relationship Between Ultimate Bearing Capacity and Area Replacement Ratio (after Aboshi and Suematsu, 1985)

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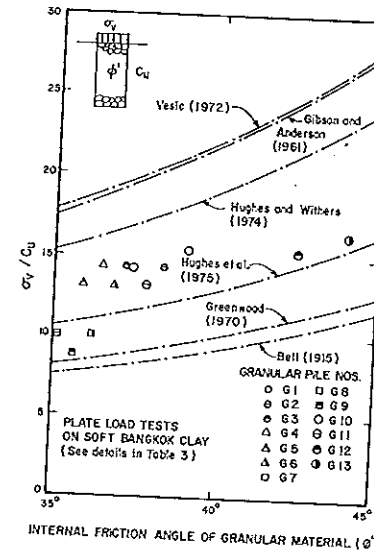


Fig. 9 Relationship Between Internal Friction Angle of Granular Material, Strength of the Surrounding Clay, and Ultimate Bearing Capacity of Single Granular Pile (after Bergado and Lam, 1987)

Ultimate Bearing Capacity of Granular Pile Groups

The common method for estimating the ultimate bearing capacity of granular pile groups assumed that the angle of internal friction in the surrounding cohesive soil and the cohesion in the granular pile are negligible. Furthermore, the full strength of both the granular pile and cohesive soil has been mobilized. The pile group is also assumed to be loaded by rigid foundation. The ultimate bearing capacity of granular pile groups as suggested by Barksdale and Bachus (1983) is determined by approximating the failure surface with two straight rupture lines as shown in Fig. 10. Assuming the ultimate vertical stress, q_{ult} , and the ultimate lateral stress, σ_3 , to be the principal stresses, then the equilibrium of the wedge requires:

$$q_{ult} = \sigma_3 \tan^2 \beta + 2c_{avg} \tan \beta \tag{9}$$

where:

$$\sigma_3 = \frac{\gamma_c B \tan \beta}{2} + 2c \tag{10}$$

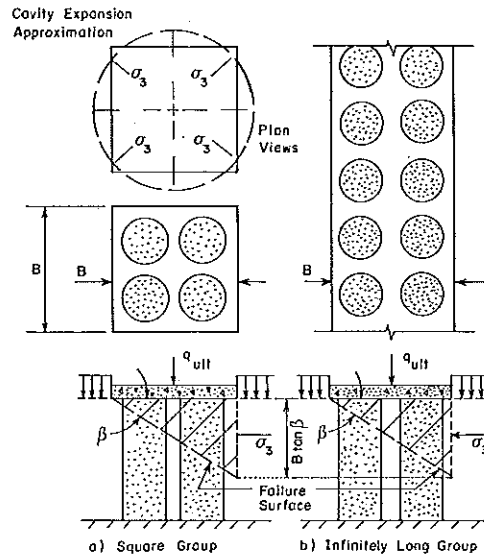


Fig. 10 Granular Pile Group Analysis (after Barksdale and Bachus, 1983)

$$\beta = 45 + \frac{\phi_{avg}}{2} \quad (11)$$

$$\phi_{avg} = \tan^{-1} (\mu_s a_s \tan \phi_s) \quad (12)$$

$$c_{avg} = (1 - a_s) c \quad (13)$$

where γ_c = saturated or wet unit weight of the cohesive soil; B = foundation width; β = failure surface inclination; c = undrained shear strength within the unreinforced cohesive soil; ϕ_s = angle of internal friction of the granular soil; ϕ_{avg} = composite angle of internal friction; c_{avg} = composite cohesion on the shear surface.

The development of the above approach did not consider the possibility of a local bulging failure of the individual pile. Hence, the approach is only applicable for firm and stronger cohesive soils having an undrained strengths greater than 30-40 kN/m². However, it is useful for approximately determining the relative effects on ultimate bearing capacity design variables such as pile diameter, spacing, gain in shear strength due to consolidation and angle of internal friction.

Table 2 Estimation of Settlement of Composite Ground (after Aboshi and Suematsu, 1975)

| METHODS | CONTENTS | REFERENCES |
|-----------------------|---|--|
| Equilibrium Method | $S_t = m_v (\mu_c \sigma) H$ $R = \mu_c = \frac{1}{1 + (n-1)a_s}$ | Aboshi et al. (1979) |
| Priebe Method | $\frac{1}{R} = 1 + a_s \left[\frac{1/2 + f(\mu, a_s)}{(K_A)_s f(\mu, a_s)} - 1 \right]$ $f(\mu, a_s) = \left[\frac{1 - \mu^2}{1 - \mu - 2\mu^2} \right] \left[\frac{(1 - 2\mu)(1 - a_s)}{1 - 2\mu + a_s} \right] (K_A)_s = \tan^2 (45^\circ - \frac{\phi_s}{2})$ | Priebe (1976) |
| Granular Wall Method | $s_s = RH (1 - \mu^2) \left(1 - \frac{\mu^2}{1 - \mu^2} \right) \frac{\sigma}{E}$ $R = f(a_s, \phi_s, \mu, \sigma/E)$ | Van Impe and De Beer (1983) |
| Finite Element Method | $\epsilon_v = (1 - a_s) \frac{C_c}{1 + e_o} \log_{10} \left[\frac{(P'_{o'vc}) + \Delta P}{(P'_{o'vc})} \right]$ $\Delta P = \frac{(\Delta P)_{vc}^*}{1 + 2K_o} [1 + K + K_o (K \text{ if } K > 1)]$ $K = K_o + \frac{1}{\epsilon_v} \left[\sqrt{\frac{1}{1 - \epsilon_v}} - 1 \right] \frac{\sqrt{s_s}}{1 - \sqrt{s_s}}$ $(\Delta P)_{vc}^* = \frac{(\Delta P)_{vc} + (P'_{o'vc}) a_s - K_o (P'_{o'vc}) a_s \tan^2 (45 + \phi_s/2)}{K F a_s \tan^2 (45 + \phi_s/2)}$ $R_p = \frac{C_c}{1 + e_o} \log_{10} \left[\frac{(P'_{o'vc}) + (\Delta P)_{vc}^*}{(P'_{o'vc})} \right]$ | Goughnour (1983) Baumann and Bauer (1974) Hughes et al. (1975) |
| Finite Element Method | $[K_E] \{\Delta \sigma^{(a-1)}\} = \{ \{\Delta F_E\} + [K_c^{(a)}] \{\Delta \sigma^{(a)}\} + \{\Delta F_{DN}^{(m)}\} \}$ | Balaam and Poulos (1983) |

Note: Refer to APPENDIX for Notations

For the case of the soft and very soft cohesive soils, the pile group capacity is predicted using the capacity of a single, isolated pile located within a group and to be multiplied by the number of piles (Barksdale and Bachus, 1983). The ultimate bearing capacity for a single, isolated pile in this case is expressed as:

$$q_{ult} = cN'_c \quad (14)$$

where N'_c = composite bearing capacity factor for the granular pile which ranges from 18 to 22. For the soft Bangkok clay, N'_c ranges from 15 to 18 using an initial pile diameter of 25.4 cm with the gravel compacted by 0.16 ton hammer dropping 0.70 m (Bergado and Lam, 1987).

Settlement of the Composite Ground

Most of the approaches in estimating settlement of the composite ground assumed an infinitely wide loaded area reinforced with granular piles having a constant diameter and spacing. For this loading condition and geometry, the unit cell idealization is assumed to be valid. The model of a unit cell loaded by a rigid plate is analogous to a one-dimensional consolidation test. Thus, the unit cell is confined by a rigid frictionless wall and the vertical strains at any horizontal level are uniform. Different methods for estimating the settlement of the composite ground are summarized in Table 2. The settlement reduction ratio is expressed as:

$$R = S_i / S_o \quad (15)$$

where S_i = settlement of the composite ground and S_o = settlement of the unimproved ground. In the case of the equilibrium method, estimation of the settlement of the composite ground is expressed as:

$$S_i = m_v (\mu_c \sigma) \quad (16)$$

where m_v = modulus of volume compressibility and H = thickness of layer. The settlement reduction ratio is also expressed as a function of the area replacement ratio (a_s), angle of internal friction of the granular materials (ϕ_s), stress concentration factor, and etc. Figure 11 shows the relationships between the

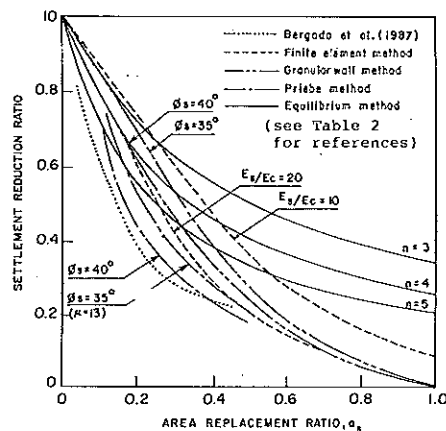


Fig. 11 Comparison of Estimating Settlement Reduction of Improved Ground (after Aboshi and Suematsu, 1985)

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settlement reduction ratio and the aforementioned parameters based on different methods together with the results from the work of Bergado et al. (1987) on soft Bangkok clay.

Slope Stability of the Composite Ground

Granular piles could be used to increase the stability of slopes and embankments constructed over soft cohesive ground. The method of stability analysis on a composite ground performed exactly in the same manner as for a normal slope stability problem except that stress concentration is considered. When circular rotational failure is expected, the simplified method of slices is recommended (Fredlund and Krahn, 1977; Whitman and Bailey, 1967). The method of stability analysis is extended, as an approximation, to evaluate the stability over large areas improved with granular piles and imposed with heavy loads. Stability analysis are usually carried out with the implementation of computer programs. The three general techniques used in stability analysis of the composite ground are described as follows (see Barksdale and Bachus, 1983):

Profile Method

In the profile method, each row of the granular piles is converted into an equivalent, continuous strip with width, W , as shown in Fig. 12. Each strip of granular and cohesive soils is then analyzed using its actual geometry and material properties. For an economical design, the stress concentration developed in the piles must be taken into consideration. The stress concentration in the granular pile results in an increase in resisting shear force. The effect of the stress concentration is being handled by placing thin, fictitious strips of soil above the insitu soil and granular piles at the embankment interface (see Fig. 12). The weight of the fictitious strips of soil placed above the granular piles is relatively large to cause the desired stress concentration when added to the stress caused by the embankment. The weight of the fictitious soil placed above the insitu soil must be negative to give proper reduction in stress when added to that caused by the embankment. The fictitious strips placed above the insitu soil and granular piles would have no shear strength and their weights are respectively expressed as:

$$\gamma_f^c = \frac{(\mu_c - 1) \gamma_1 H'}{T} \quad (17)$$

$$\gamma_f^s = \frac{(\mu_s - 1) \gamma_1 H'}{T} \quad (18)$$

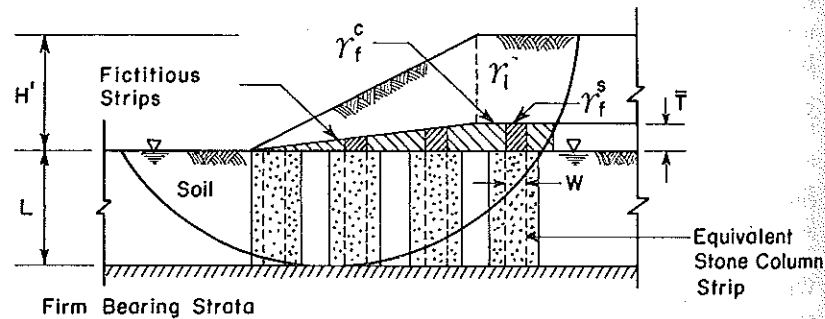


Fig. 12 Granular Pile Strip Idealization and Fictitious Soil Layer for Slope Stability Analysis (after Barksdale and Bachus, 1983)

where μ_s and μ_c are the stress concentration factors of the insitu soil and granular piles, respectively, and the other terms are defined as indicated in Fig. 12. It must be noted that limits should be imposed on the radius and/or grid size of circle centers so that critical circle should not be controlled by the weak, fictitious interface layer.

Average Shear Strength Method

The average shear strength method is widely used in stability analysis for sand compaction piles (Aboshi et al., 1979; Barksdale, 1981). The method considers the weighted average material properties of the materials within the unit cell (Fig. 13). The soil having the fictitious weighted material properties is then used in stability analysis. Since average properties can be readily calculated, this approach is appealing for both hand and computer calculations. However, average properties cannot be generally used in standard computer programs when stress concentration in the granular piles is considered in the analysis.

When stress concentration is considered, hand calculation is preferred. Within the unit cell, the granular pile has only internal friction, ϕ_s , and the surrounding soil is undrained but has cohesion, c , and internal friction, ϕ_c . The state of stresses within the unit cell is also shown in Fig. 13. The effective stresses in the granular pile and the total stress in the surrounding soil are respectively expressed as:

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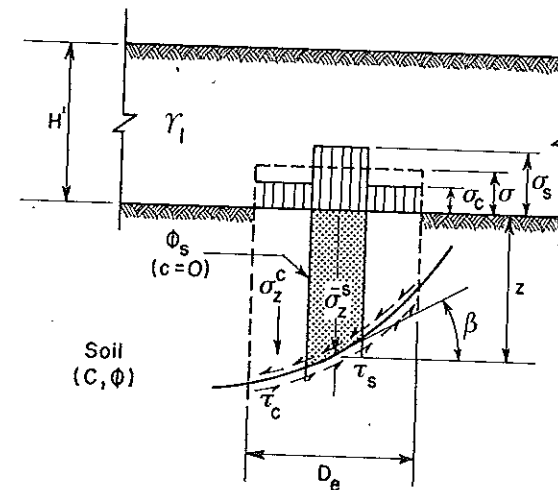


Fig. 13 Average Stress Method of Stability Analysis (after Barksdale and Bachus, 1983)

$$\bar{\sigma}_z^s = \bar{\gamma}_s z + \sigma \mu_s \quad (19)$$

$$\bar{\sigma}_z^c = \bar{\gamma}_c z + \sigma \mu_c \quad (20)$$

where γ_s = bouyant weight of the granular materials; γ_c = saturated unit weight of the surrounding soil; z = depth below the ground surface; σ = stress due to embankment loading, and the other terms are already defined. The shear strength of the granular pile and the surrounding cohesive soil are:

$$\tau_s = (\bar{\sigma}_z^s \cos^2 \beta) \tan \phi_s \quad (21)$$

$$\tau_c = c + (\bar{\sigma}_z^c \cos^2 \beta) \tan \phi_c \quad (22)$$

where β = inclination of the shear surface with respect to the horizontal. The average weighted shear strength within the area tributary to the granular pile is:

$$\tau = (1 - a_s) \tau_c + a_s \tau_s \quad (23)$$

The weighted average unit weight within the composite ground used in calculating the driving moment is:

$$\gamma_{avg} = \gamma_s a_s + \gamma_c a_c \quad (24)$$

where γ_s and γ_c are the saturated unit weight of the granular materials and the insitu soil, respectively. In this approach, the weighted shear strength and unit weight are calculated for each row of granular piles and then used in conventional hand calculation.

When stress concentration is not considered, as in the case of some landslide problems, a standard computer analysis employing average strengths and unit weights, can be performed using a conventional computer program. Neglecting the cohesion in the granular materials and the stress concentration, the shear strength parameters of the use in the average shear strength method are:

$$c_{avg} = c(a_s) \quad (25)$$

$$(\tan \phi)_{avg} = \frac{\bar{\gamma}_s a_s \tan \phi_s + \gamma_c a_c \tan \phi_c}{\bar{\gamma}_{avg}} \quad (26)$$

where $\bar{\gamma}_{avg}$ is given by Eq. (24) using bouyant weight for $\bar{\gamma}_s$ and saturated weight for γ_c . DiMaggio (1978) reported that the use of $(\tan \phi)_{avg}$ based just on the area ratio is not correct as can be demonstrated by considering the case when $\phi_c = 0$. If averages based on the area were used, then:

$$(\tan \phi)_{avg} = a_s \tan \phi_s \quad (27)$$

The above expression would be appropriate to use if $\bar{\gamma}_{avg} = \bar{\gamma}_s$, but incorrect if $\bar{\gamma}_{avg}$ is used to calculate the driving moment.

Lumped Moment Method

The method can be used to determine the safety factor of selected trial circles by either hand calculations or with the aid of a computer. The general approach is described by Chambosse and Dobson (1982). The safety factor of the composite ground is calculated by:

$$SF = (RM + \Delta RM) / (DM + \Delta DM) \quad (28)$$

where RM = resisting moment, DM = driving moment, ΔRM = excess resisting moment due to granular piles, ΔDM = excess driving moment due to granular piles. The DM and RM are first calculated for the condition of unimproved ground. Then ΔRM and ΔDM are added to the previously calculated moments, RM and DM, respectively. The approach is generally suited for hand calculations. The use of computer programs is possible only when adding ΔRM and ΔDM which could be calculated by hand.

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Rate of Primary Consolidation Settlement

Previous studies assumed that granular piles could accelerate the consolidation process in the same way as sand drains. In a cohesive soil reinforced with granular piles, pore water moves towards the pile in a curved path having both vertical and radial components of flow. The average degree of primary consolidation could be handled by considering the vertical and radial consolidation effects separately as expressed by the following equation:

$$U = 1 - (1 - U_v) (1 - U_r) \quad (29)$$

where U = average degree of consolidation of the cohesive layer considering both vertical and radial drainage, U_v = degree of consolidation considering only vertical flow and U_r = degree of consolidation considering radial flow. The degree of consolidation in the vertical direction is calculated by Terzaghi's one-dimensional theory, while that in the radial direction is calculated by Barron's theory. The primary consolidation settlement at any time, t , is expressed as:

$$S_c(t) = U(S_{cf}) \quad (30)$$

where $S_c(t)$ = primary consolidation settlement at any time, t ; S_{cf} = final primary consolidation settlement.

Strength Increase of Clay Due to Consolidation

The rate of construction of embankments on ground improved with granular piles is frequently controlled to allow the shear strength to increase so that the required safety factor against instability is maintained. The undrained shear strength of a normally consolidated clay has been found to increase linearly with effective overburden pressure (Leonards, 1962). Consolidation results to an increase in effective stress due to the dissipation of pore pressure. For a cohesive soil having a linear increase in shear strength with effective stress, the increase in undrained shear strength, $\Delta c(t)$, with time due to consolidation can be expressed as (Barksdale and Bachus, 1983):

$$\Delta c(t) = k_1 [\sigma \mu_c] [U(t)] \quad (31)$$

where $k_1 = c/\sigma$, constant of proportionality defining the linear increase in shear strength with effective stress; $U(t)$ = degree of consolidation of the clay at any time, t , and the other terms are defined previously. The ratio of the measured increase in undrained shear strength to the calculated ones is a function of area replacement ratio and stress concentration factor.

Secondary Settlement

Secondary settlement is important in organic soils and some soft clays especially with the presence of free draining granular piles wherein primary consolidation is thought to occur in a short period of time. The prediction for secondary settlement is based on the work of Mesri (1973) and can be calculated by:

$$S_s = C_{\alpha} H \log_{10} \frac{t_2}{t_1} \quad (32)$$

where S_s = secondary compression of the layer; C_{α} = physical constant evaluated by continuing a one-dimensional consolidation test past the end of primary consolidation for a suitable load increment; H = thickness of compressible layer; t_1 = time at the beginning of secondary compression (the time corresponding to 90% of primary consolidation is sometimes used); t_2 = time at which the value of secondary settlement is desired.

PREVIOUS RESEARCHES ON SOFT BANGKOK CLAY IMPROVED WITH GRANULAR PILES

Full Scale Load Tests on Granular Piles

Initial full scale load tests on granular piles were performed to study the feasibility of improving the soft Bangkok clay (see Bergado *et al.*, 1984). The results indicated that the granular piles increased the bearing capacity more than 3 to 4 times that of the untreated ground. Further, the adjacent piles acted independently provided that the pile spacing is 3 times the pile diameter or greater as shown in Fig. 14. An investigation of the behaviour of granular piles with different densities and different proportions of gravel and sand on soft Bangkok clay was carried out by Bergado and Lam (1987). Figure 15 shows that for the same granular materials, the ultimate bearing capacity increases with number of blows per layer because of the increase in the densities and friction angle. Using different proportions of gravel and sand as shown in Table 3, the resulting load-settlement curves are also compared in Fig. 15 and indicated a higher ultimate pile capacity for pure gravel. The average deformed shape of the granular piles is typically bulging type. It was observed that the maximum bulge occurred near the top of the pile and ranged from 10 cm to 30 cm below the ground surface. With an initial pile diameter of 30 cm, the measurements of bulge are in close agreement with the observations of Hughes *et al.* (1975) wherein the maximum bulge

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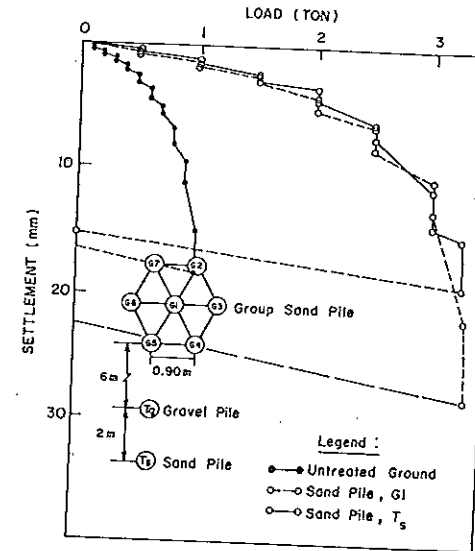


Fig. 14 Load-Settlement Relationship of Granular Piles from Full Scale Load Tests (after Bergado *et al.*, 1984)

occurred near the ground surface at a depth approximately equal to one-half to one pile diameter. The results of full scale load tests on granular piles with different sizes of plates were reported by Bergado *et al.* (1987). As expected, the settlement decreased as the size of the plate increased for the same amount of total load. It was also indicated that as the ratio D_p/D increased, the settlement of the treated ground approaches the settlement of untreated ground as shown in Fig. 16. This implies that beyond $D_p/D = 4.0$, the settlement of treated ground is almost the same as the settlement of untreated ground. It should be noted that D_p is directly related to the spacing of granular piles as expressed in Eqs. 1 and 2, depending on the pattern used.

Site Location and Soil Profile of Test Embankment on Granular Piles

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 4.0 m high test embankment constructed on Mebra

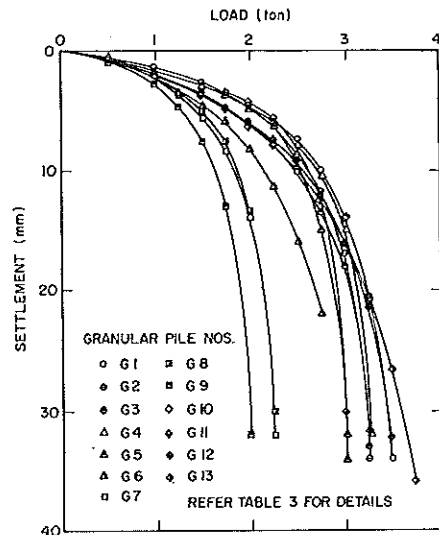


Fig. 15 Comparison of Load-Settlement Curves Between Piles with Different Number of Blows per each Compacted Layer during Installation (after Bergado and Lam, 1987)

vertical drain-improved foundation by Singh (1986). The performances of these two embankments have been already reported by Bergado et al. (1988, 1990a). An evaluation by back-analysis of the geotechnical parameters of the foundation soil improved by granular piles and vertical drains was carried out by Enriquez (1989).

The site is located at the Asian Institute of Technology (AIT) campus, about 42 km north of Bangkok at the Central (Chao Phraya) Plain of Thailand. The soil profile at the site, together with the index and strength properties of the soil, is given in Fig. 17. The profile consists of 2 to 3 m of weathered marine clay underlain by 6 to 8 m thick of soft marine clay and followed by a stratum 5 to 6 m thick of stiff-clay layer. The groundwater table fluctuated with the season and varied from 0.5 to 2.0 m below the ground level.

Details of the Test Embankment

The test embankment has a first stage height of 2.4 m and subsequently raised to a second stage height of 4.0 m after 345 days. The plan and cross-section of

Table 3 Properties of Granular Piles for Full Scale Load Tests (after Bergado and Lam, 1987)

| Group | 1 | | | 2 | | | 3 | | | 4 | | 5 | |
|---------------------------------|-----------------------|------|------|-----------------------|------|------|-----------------------|------|------|-----------------------|------|-----------------------|------|
| No. of Pile | G 1 | G 2 | G 3 | G 4 | G 5 | G 6 | G 7 | G 8 | G 9 | G 10 | G 11 | G 12 | G 13 |
| Proportion of Sand in Volume | 1.0 | | | 1.0 | | | 1.0 | | | 0.3 | | 0.0 | |
| Proportion of Gravel in Volume | 0.0 | | | 0.0 | | | 0.0 | | | 1.0 | | 1.0 | |
| Blows per Compacted Layer | 20 | | | 15 | | | 10 | | | 15 | | 15 | |
| In - situ Average Density (t/m) | 1.73 | 1.71 | 1.66 | 1.64 | 1.51 | 1.67 | 1.47 | 1.53 | 1.50 | 1.91 | 1.96 | 1.76 | 1.79 |
| Average | 1.70 t/m ³ | | | 1.61 t/m ³ | | | 1.50 t/m ³ | | | 1.94 t/m ³ | | 1.74 t/m ³ | |
| Friction Angle (Degree) | 39.1 | 38.4 | 37.2 | 37.0 | 36.0 | 37.6 | 35.1 | 36.2 | 35.6 | 37.4 | 37.9 | 42.5 | 44.7 |
| Average | 38.2 | | | 36.9 ° | | | 35.6 ° | | | 37.7 ° | | 43.3 ° | |
| Ultimate Load (tons) | 3.50 | 3.25 | 3.25 | 3.25 | 3.00 | 3.00 | 2.25 | 2.25 | 2.00 | 3.25 | 3.00 | 3.50 | 3.75 |
| Average | 3.33 tons | | | 3.08 tons | | | 2.17 tons | | | 3.13 tons | | 3.63 tons | |

the embankment including the locations of the monitoring instruments, field tests, and field sampling for laboratory tests are shown in Fig. 18. The monitoring program included surface and subsurface settlement measurements, pore pressures, vertical earth pressures, and lateral movement. The instrumentations installed consisted of 3 surface settlement plates, 4 subsurface settlement gauges at 1.5 m, 3.0 m, 6.0 m and 8.0 m depths, 8 closed hydraulic piezometers, 2 earth pressure cells, and SIS Geotechnica C412 type inclinometer casing with readings provided by the SIS Geotechnica sensor, and three lateral movement stakes. The embankment was compacted in layers by a light vibrating plate tamper and was found to have an average density of 18.0 kN/m³.

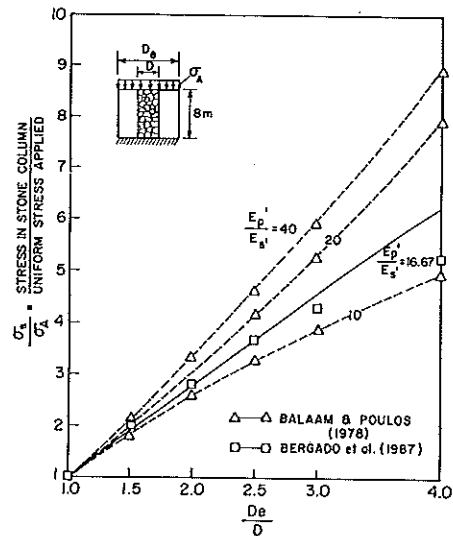


Fig. 16 Variation of Vertical Stress in Granular Pile with D_p/D (after Bergado et al., 1987)

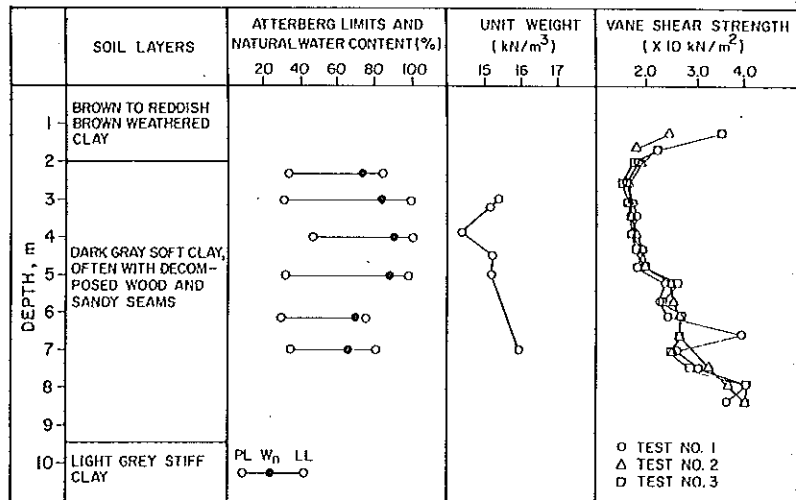


Fig. 17 Soil Profile and Properties at AIT Campus

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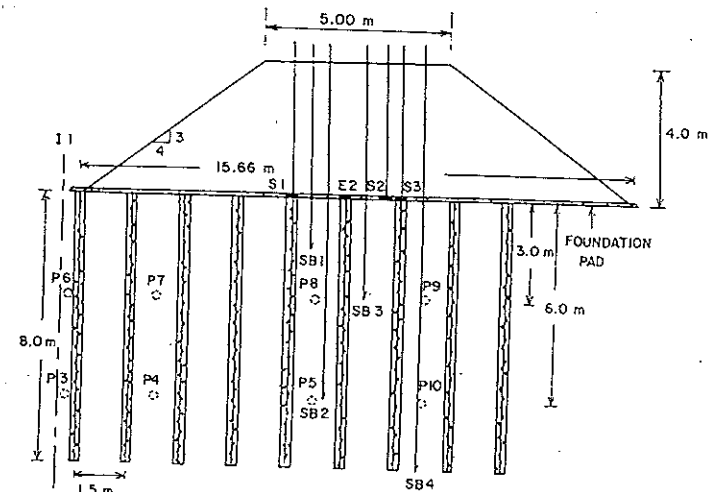
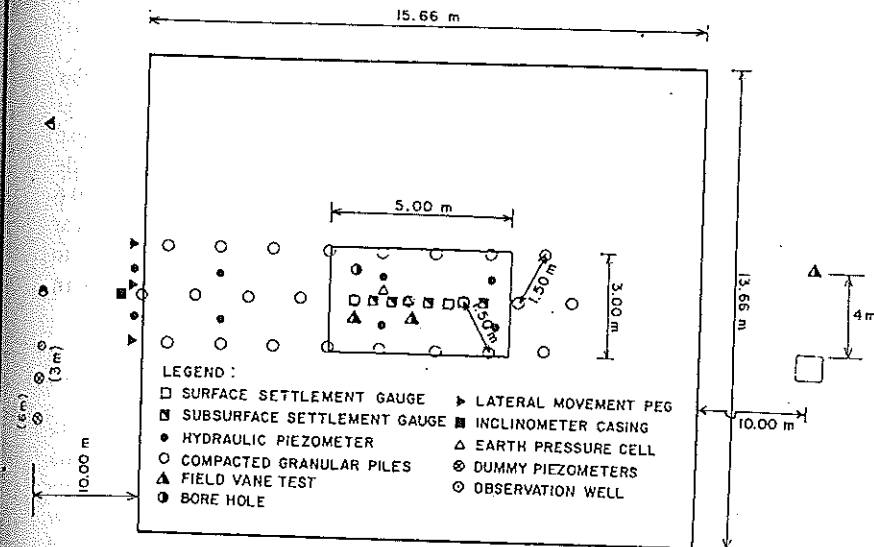


Fig. 18 Plan and Embankment Cross-Section Including Locations of Instrumentations, Field Tests, and Field Sampling

Construction of Compacted Granular Piles

The Cased Borehole Method was employed in constructing the granular piles (Bergado *et al.*, 1984; Bergado and Lam, 1987). The compaction was done by means of dropping the 1.6 kN hammer at 0.6 m falling height to the steel disc which was placed on the surface of the granular material. Each layer of the pile was compacted with 15 blows per layer, with a compacted thickness of about 0.6 m. The friction angles of the compacted granular piles obtained from direct shear tests varied from 39 to 45 degrees with compacted densities ranging from 17 to 18.1 kN/m³. The compacted granular piles were arranged in triangular pattern

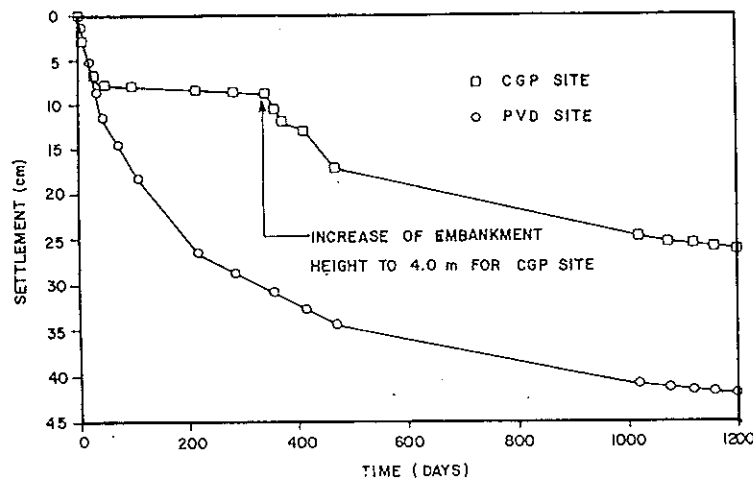


Fig. 19 Comparison of Maximum Surface Settlements at Granular Piles and Vertical Drains Test Embankments (after Enriquez, 1989)

with a spacing of 1.5 m. The piles were 30 cm in diameter and have a length of 8.0 m, fully penetrating the soft clay layer. The granular materials consists of whitish-gray, poorly graded crushed limestones with a maximum size of 20 mm. A drainage blanket of 0.25 m thick consisting of clean sand was laid on top of the compacted granular piles.

Performance of Granular Piles under Embankment loading

Based on full scale load tests on granular piles, Bergado *et al.* (1984, 1987) and Bergado and Lam (1987) earlier reported that the bearing capacity of the soft

Bangkok clay using granular piles increased by up to 4 times, the total settlements reduced by at least 30% and the slope stability safety factor increased by at least 25%. The comparative study of the performance of test embankments on compacted granular piles (CGP) and on prefabricated vertical drains (PVD) indicated that the embankment on granular piles settled about 40% less than the embankment on vertical drains, as shown in Fig. 19. The study confirms the idea that granular piles function as a reinforcement to the clay rather than drains (Bergado *et al.*, 1988), attributed perhaps to the larger zone of disturbance (smeared zone) surrounding the pile caused by the method of installation.

SOME PROSPECTS OF GRANULAR PILES IN COMBINATION WITH OTHER SOIL IMPROVEMENT TECHNIQUES

At Ebetsu in Hokkaido, Japan, two full scale test embankments were constructed with different soil stabilization methods in addition to the test embankment without treatment (Aboshi and Suematsu, 1985). One of the test embankments was stabilized with sand drains and steel sheet reinforcements while the other was stabilized with sand compaction piles. At an embankment height of 3.5 m, the test embankment without treatment collapse with substantial deformations on the subsoil and cracks in the embankment fill. While the embankments treated with sand drains and steel reinforcements and with sand compaction piles were completed up to 8 m high without failures. The test embankment stabilized by sand compaction piles yielded a stress concentration factor of 3 which is in close agreement with the observations of Bergado *et al.* (1988) on soft Bangkok clay improved with granular piles having a stress concentration factor ranging from 2 to 5. Based from the results of these test embankments, it is envisioned that the use of granular piles and steel grids reinforcements could be a good combination for ground stabilization. The grid reinforcements help minimize the lateral spreading of embankment and provide steeper side slopes or even vertical sides. The granular piles provide the reduction of settlements as well as the increase in strength and bearing capacity of the ground foundation. An example for such applications is on transition units to bridges and viaducts as shown in Fig. 20. As an alternative of using expensive ideal materials for embankment fill such as sand, savings in the construction costs can be realized by using cheaper, locally available cohesive-frictional soil with more than 18% having particle size diameter lower than 0.74 mm. Extensive research has been done on steel grids reinforcements with poor quality backfills consisting of weathered clay, lateritic soils and clayey sand (see Bergado *et al.*, 1990b). However, the combination scheme of granular piles and steel grids reinforcements to improve

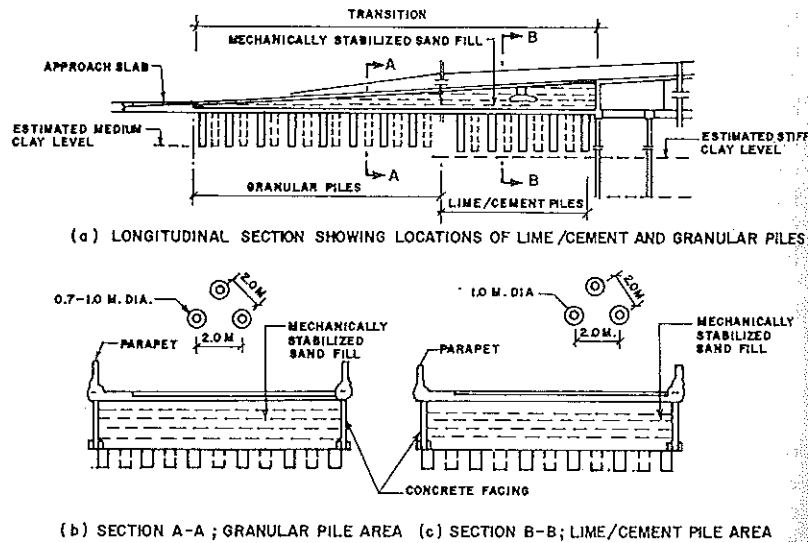


Fig. 20 Granular Piles with Mechanically Stabilized Embankment

the ground and embankment fill, respectively, must be studied through full scale field prototype so that their effectiveness on soft and subsiding Bangkok clay would be proven and the reduction of lateral spreading can be measured.

SUMMARY AND CONCLUSION

The current state-of-the-art on granular pile scheme has been discussed but there are still some loopholes which need to be studied for better understanding of this ground improvement method. Among these are the influences of the method of construction, characteristics of pile materials, better estimation of stress concentration factors, stress distribution with depth and time, improvement factors as well as the effect of overlying surcharge consisting of mechanically stabilized (reinforced) embankment. It was indicated that there is a need for additional research to improve the design methods and develop a complete understanding of the mechanics of granular pile behavior. More full scale tests and model tests with extensive instrumentation in combination of finite element model studies should be carried out to shed light to the uncertainties and improve confidence especially when the scheme is applied to the soft and subsiding ground.

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Already, a test embankment on granular piles has been constructed at AIT to evaluate and investigate the applicability of such scheme on the soft and subsiding Bangkok clay. The performance of this test embankment has been described in the preceding section. The improvement factors on bearing capacity, settlement and gain in strength have been determined and indicated substantial values. Based on the results of these studies as well as in Japan, it was suggested to use granular piles in the subsoil in combination with mechanically stabilized earth (MSE) embankments using grid reinforcements as one alternative for ground improvement at the approach embankments to bridges and viaducts in the Second Stage Bangkok Expressway. The implementation of this ground improvement scheme was postponed mainly due to the lack of available data regarding its actual performance. Therefore, it is deemed necessary to construct a full scale field prototype so that the effectiveness of granular piles, possibly combined with other soil improvement techniques, as an alternative scheme for ground improvement on approach embankments to bridges and viaducts would be proven.

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APPENDIX: Notations for Tables 1 and 2

- A_s : cross sectional area of sand pile
- B : width of loaded area
- C_c : compression index of clay
- C_o : Original undrained shear strength of clay
- D_f : depth of foundation
- E : modulus of elasticity
- F_c', F_q' : cavity expansion factors
- H : thickness of layer
- K : earth pressure coefficient applying to the load increments
- K_{as} : active earth pressure coefficient of sand pile
- $[K_c^{(m)}] \{ \Delta\sigma^{(m)} \}$: vector of corrections due to yielding of the pile and/or clay, these corrections are treated as an additional external load
- $[K_E]$: elastic stiffness matrix
- K_o : coefficient of earth pressure at rest

- K_{pc} : soil coefficient of passive earth pressure
- L : length of sand pile
- N_c, N_q, N_q' : dimensionless factors which depend on properties of soil and pile material and area replacement ratio
- $(P_o)_{vc}$: initial vertical stress in the clay
- R : settlement reduction factor
- S_t : settlement of composite foundation
- W : width of equivalent granular pile strip
- Z : depth from surface of composite foundation
- a_s : area replacement ratio
- d_s : pile diameter
- e_o : initial void ratio
- m : iteration number
- m_v : modulus of volume compressibility
- n : stress concentration factor
- q_o : overburden pressure
- q_s : bearing capacity of soft soil expressed as $(2/3) C_o N_c$
- q_{ult} : ultimate bearing capacity
- γ_s, γ_c : unit weight of sand and clay, respectively
- $\{\Delta F_{DN}\}$: vector of incremental nodal forces at the dual nodes along the pile clay interface
- $\{\Delta F_E\}$: vector of incremental nodal forces due to applied tractions (usually applied along the top of the sand pile)
- $(\Delta P)_{vc}^*$: effective vertical stress increase in the clay averaged over the horizontal projected area of clay
- $(\Delta P)_v^*$: effective vertical stress increase averaged over horizontal projected area of unit cell
- $\{\Delta\sigma^{(m+1)}\}$: vector incremental deflections
- ϵ_v : vertical strain (same for sand and clay)
- θ : vertical angle of sliding surface at each sand pile
- μ : Poisson's ratio
- μ_c : reduction in stress coefficient of clay
- σ : vertical stress
- σ_{ro} : initial radial stress along the granular pile
- σ_z : overburden pressure at depth z
- τ : shear resistance of composite foundation
- ϕ_s : angle of internal friction
- ψ : angle between the assumed failure surface and foundation