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Load Bearing Capacity of Skirted Granular Piles

by

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Introduction

Founding on very soft/loose subsoil deposits can be effectively achieved by improving the subsoil characteristics. Various techniques for ground improvement have been developed and adopted by various investigators. Using granulaı piles/stone columns a simple method of construction of granular piles has been developed (Ranjan and Rao, 1983). In-situ tests on prototype granular piles (single and groups), both with and without a rigid skirt round the piles have been carried out at four different sites and the ultimate capacity of the piles/group have been worked out. A rational analytical method, using the analogy of expansion of a cylinderical cavity in an infinite soil mass has been developed to estimate the ultimate capacity of single granular pile. The analysis has been extended to pile groups with and without skirt. The load bearing capacities obtained from full scale insitu tests have been compared with the predicted values obtained from the analytical method. Based on the study, conclusions have been made.

Earlier work

During the past decade the technique of reinforcing weak sub-soil by granular pile/stone columns have attracted considerable attention. Estimates of the bearing capacity of a single granular pile in a soft cohesive subsoil stratum have been made using analytical, empirical and experimental investigation methods. These are briefly summarized in the following section.

Analytical Approaches

The expressions for ultimate capacity of granular piles have been arrived at by utilising passive pressure of the soft clay deposit surrounding the pile (Greenwood 1970, Hughes and Withers 1974). These approaches consider

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two-dimensional plastic failure case though actually it is a three dimensional case. The results are thus on the conservative side. The energy concept (Datye and Nagaraju, 1981) has also been utilised to estimate the ultimate capacity. Cavity expansion approach for the two extremes of consistency of the clay (soft and stiff clays) for both drained and undrained conditions has also been used to compute ultimate capacity. The granular piles are considered as shear pins. Also, simplified assumptions regarding distribution of load between the granular pile and the surrounding soil, depth of failure of granular pile and rigidity index of the soil deposit have been made. Shear failure modes for the computation of bearing capacity of strip footing placed on granular trench in cohesive soft soil have also been considered (Madhav *et al.*, 1978).

Empirical Approaches

Computation of ultimate capacity, based on empirical methods, attempt to relate the granular pile capacity with the undrained shear strength, c_u of the clay deposit (Thornburn and McVicar 1968, Thompson 1975) and are reported to compare well with predicted allowable load (Hughes and Withers 1974). In developing these empirical correlations it is assumed that the total design load is carried by the granular pile alone. The predicted pile capacities based on this assumption are highly conservative. The assumption may be true for strip footings resting on granular piles but may not be justified for a raft or an embankment which transmit load to a large area. Charts relating allowable load per pile and undrained shear strength of the clay surrounding the pile for specific settlement have also been developed (Smoltezyk *et al.*, 1979).

Semi-empirical approaches

Utilizing the pressuremeter theory (Bishop *et al.*, 1945, Gibson and Anderson 1961) a semi-empirical design approach has been proposed by Hughes and Withers (1974). The ultimate capacity of a single pile, q_u is given by

$$q_u = K_p\,(\sigma_{ho} + k\,c_u) \tag{1}$$

where K_p = coefficient of passive earth pressure for the pile material,

c_u = undrained shear strength of the clay,

σ_{ho} = total initial lateral ground stress and

k = constant (equation 2).

$$k = [1 + Ln\left\{\frac{E_s}{2c_u(1+\mu)}\right\}] = 1 + Ln(I_r) \tag{2}$$

where

E_s = elastic soil modulus.

μ = poisson ratio

I_r = rigidity index (Vesic, 1972).

Equation 1 is developed on the basis that all the design load is sustained by the pile only which is contrary to actual field conditions. Also the reliability of ultimate capacity obtained from equation 1 is dependent upon the correct estimation of the parameters K_p, σ_{h_0} and the constant k.

The value of K_p is dependent on the value of ϕ' for the granular pile material which varies from 3.69, 4 to 5.89 for ϕ' values of 35°, 38° and 45° respectively as found from pile load tests (Hughes *et al.*, 1975, Englehardt and Golding 1975, Mori 1979, Broms 1979 and Thorburn 1983). The total initial lateral ground stress σ_{h_0} may be obtained from the limit pressure-strain relationship from insitu pressure meter test or be taken as equal to $2c_u$ (Hughes *et al.*, 1975) though the former is recommended to be preferable. Mori (1979) has used effective over-burden pressure ($0.5\ \gamma h$) for σ_{h_0}. The value of h is taken as 5 m below ground level and is equivalent to effective overburden pressureat a depth of four times the granular pile diameter. The value of the constant k varies from 3 to 5 as reported by different investigators (Hughes *et al.*, 1975), Mokashi *et al.*, 1976, and Broms 1979).

From equation 2 it may be noted nthat the value of k determines the compressibility of the clay around the installed granular pile and is dependent on rigidity index I_r. The value of I_r varies from 10 for soft clay to 300 for stiff clays. The corresponding values of k are between 3.33 to 6.7 . In view of these observations a value of k of (Hughes and Withers, 1974) seems to be on conservative side. A better approximation is to have k value of 5 as proposed by Broms (1979) and Mori (1979).

Experimental approaches

A few experimental investigations in the laboratory have been reported (Hughes *et al.*, 1975, Mokashi *et al.*, 1976, Long and George 1967). Radiographic techniques have also been used (Hughes and Withers, 1974) besides few large scale field investigations (Greenwood 1970, Hughes *et al.*, 1975, Englehardt and Golding 1975). Hughes *et al.*, (1975) reported load test data of a single granular pile loaded to its ultimate capacity. The ultimate pile capacity has been predicted using equation 1. The model or field tests reported in the literature focus attention on the behaviour of single granular pile installed in soft clay deposit only. The group effect and the distribution of the applied load between the pile and the surrounding soil has not been considered.

Concept of skirted granular pile

The depth of bulging in a granular pile, subjected to sustained vertical load at the top of the pile is limited to about five pile diameters (Rao, 1982). When granular piles are grouped under a raft or embankment the last two or three rows of piles towards the edge are found to have reduced load bearing capacity than the inner rows of piles due to abrupt disappearance of the design load (Greenwood, 1970). Further, since the granular piles under load bulge, there is need to strengthen the zone, suggestions in this direction have been made to replace the bulged portion of the pile by concrete or injection of cement grout in the upper portion (Engelhardt and Kirch 1977, Floss 1979). The passive restraint on the peripheral granular piles can also be increased by providing surcharge around the foundation (Mokashi *et al.*, 1976). These solutions though appear technically viable but are uneconomical and many a time difficult to execute. The use of vertical barriers for preventing lateral soil movement was effectively utilized for construction of subway in Japan (Kotada, 1978). Broms *et. al.* (1981) has also propagated the concept of providing vertical barriers termed "tubular elements" for increasing load carrying capacity of shallow foundations. A barrier made up of concrete (Fig. 1), prefabricated brick panels, piles or timber piles arranged so as to form a rigid wall termed "skirt" around the foundation is capable of supporting fairly high loads and provide significant reduction in overall settlement (Ranjan and Rao 1983, 1985 and 1986). The concept of "skirting of foundations" is useful (Goughnour, 1988) as it offers possibility of controlling bulging near the critical top portion of granular pile. Also, the skirt results in artificially deepening the shallow foundation. A detailed systematic system on various factors controlling behaviour of skirt has been reported by Ranjan and Rao (1990).

Insitu Test program—Subsoil conditions

Full scale field tests were carried out at four different sites viz Site 1 to Site IV (Rao, 1982). Detailed sub-soil explorations of the various sites consisted of exploratory borings using the standard penetration test, the

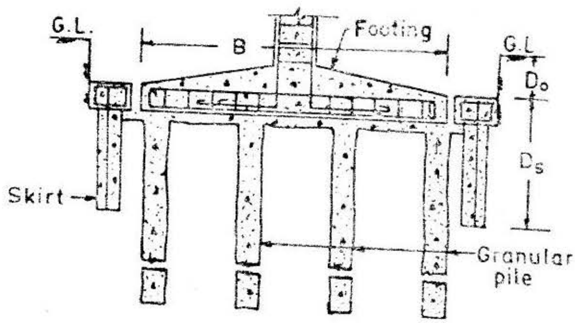

FIGURE 1 Skirted granular pile foundation

dynamic cone penetration test, the static cone penetration test, full scale footing tests and tests on disturbed/undisturbed soil samples. Of the four sites, the first two were underlain for the most part by cohesionless silty sand. The other two sites viz. Site III and IV were predominantly clayey silt and silty soft clay deposits. The bore logs of these sites with classification of soils as per unified soil classification system and N-values is shown in Fig. 2.

Casting of granular piles

Figure 3 shows the sequence of construction of granular piles. Granular piles were cast in pre bore holes using 20-30 mm stone aggregates (at site I and II) and 20-27 mm size stone aggregates (at site III and IV) with 20-25 percent of locally available sand having an uniformity coefficient of 2-3. The stone aggregates were placed in layers of 300-500 mm followed by the sand layer of 50 mm—100 mm. A 1250 N force hammer, having diameter smaller than the half for the pile and operated by a power winch with a fall of 0.75 m was used to compact the sand/stone aggregate layer. During the compaction (impact) the voids in the stone aggregates were filled by the sand followed by lateral and downward movement of the charged material until the full compaction was achieved. Thus the lateral displacement of the material helped in compaction of the surrounding soil. The uniformity in compaction of the granular piles all through the length was checked by set measurements. It was carried out at least 3 to 4 times during the installation of a granular pile depending upon the length of the pile.

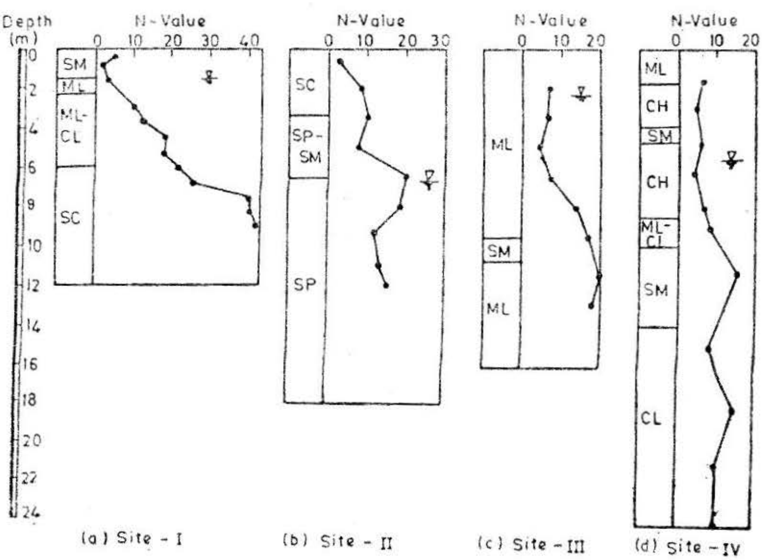

FIGURE 2 Subsoil strata and SPT values for different Sites

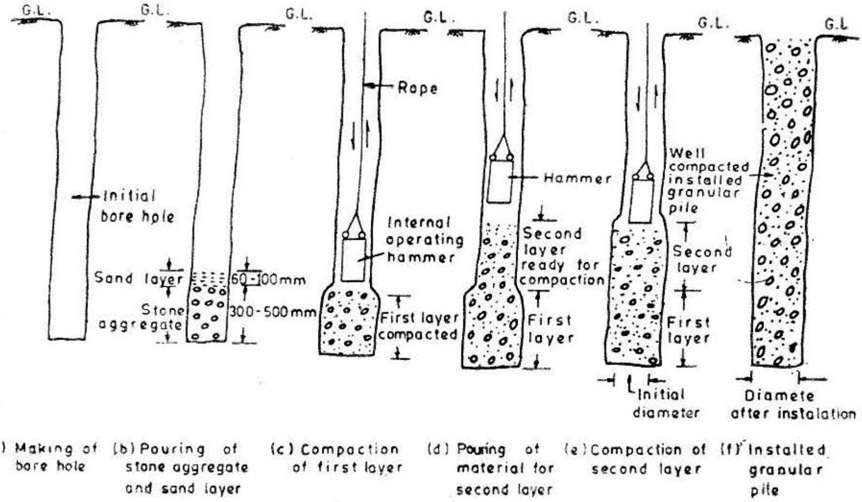

FIGURE 3 Sequence of construction of granular pile

The set measurement was carried out after a particular layer has been compacted and it consisted of allowing the hammer to fall freely about 10 times, until the compression or set become equal to less than 25 mm. For casting of piles at site I and II, since the pile depth was not large (250 mm dia 3.5 m deep) the hammer was operated manually. However, at site III and IV where the piles were 550 to 600 mm dia, and 9.5 m and 14 m deep a power winch was used with hammers weighting 3000 and 5000 N force. The holes were stabilised using 5 percent Na-bentonite slurry.

Construction of skirt

Rigid skirts of reinforced cement concrete, mild steel pipes, prefabricated pipe units and timber pile skirting were provided. The detailed procedure of construction of various types of skirt is reported elsewhere (Ranjan and Rao, 1983). At site I granular piles whether single or in groups of 2, 3 and 4 and having 250 mm dia, 3.5 m length were individually skirted by mild steel pipes (250 mm dia, 1.0 m length). However, in case of 350 mm dia piles, 8 m long, and in groups of 2, 3 and 4 piles were skirted by a cast insitu reinforced cement concrete wall having 150 mm thickness and 0.80 m depth. Also, for some of the 4 piles groups of 250 mm dia, prefabricated pipe units and timber pile skirting was used.

Insitu load test Procedure and test results

Plain granular piles, either single or in groups, skirted individually or collectively were tested under vertical load. Also, for comparison plain footing with plan dimension equal to that of the pile cap of granular piles were also tested. The load was applied in suitable increments through hydraulic jacks taking reaction against a loaded platform. The load was recorded through a proving ring and the deformation through suitably mounted dial gauges. Typical results of the load displacement for insitu tests carried out at site I under sustained vertical loads are shown in Figs. 4 and 5.

Analysis

The ultimate capacity of a skirted granular pile foundation is considered as the sum of the ultjmate capacity of granular pile group below the plane YY (Fig. 6) and the ultimate capacity of the skirted granular piles above the plane YY.

Ultimate capacity of granular pile group

The expansion of a cylindrical cavity in a homogeneous isotropic and infinite soil mass has been utilized to estimate ultimate capacity of a single granular pile.

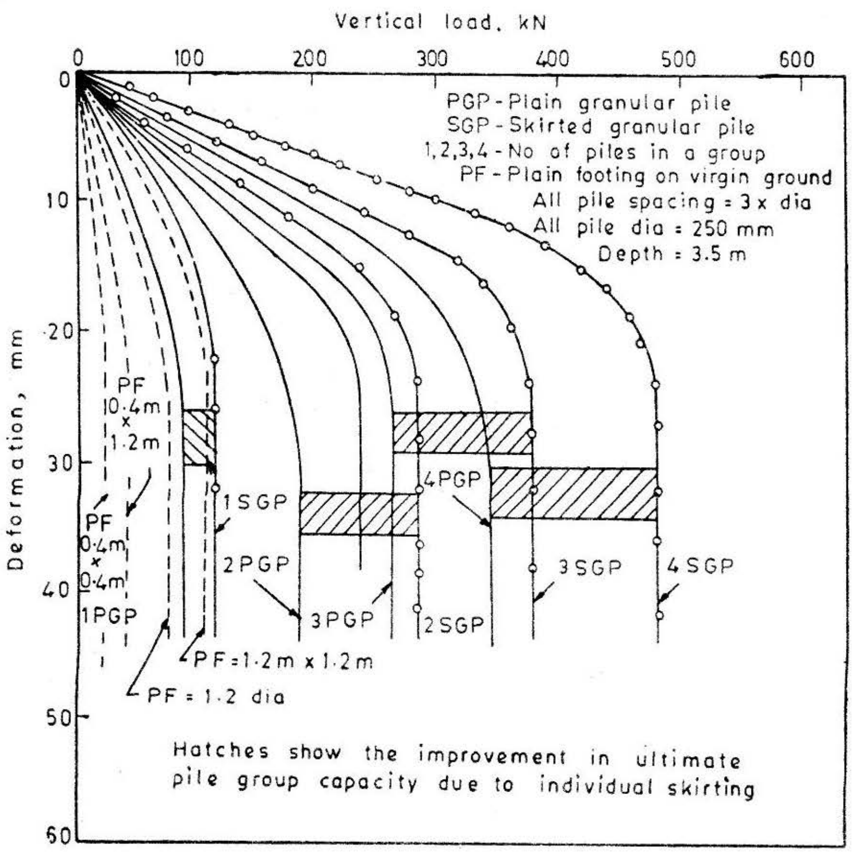

FIGURE 4 Load deformation behaviour of single and groups of 2, 3 and 4 pile groups, plain and individually skirted piles at Site-I

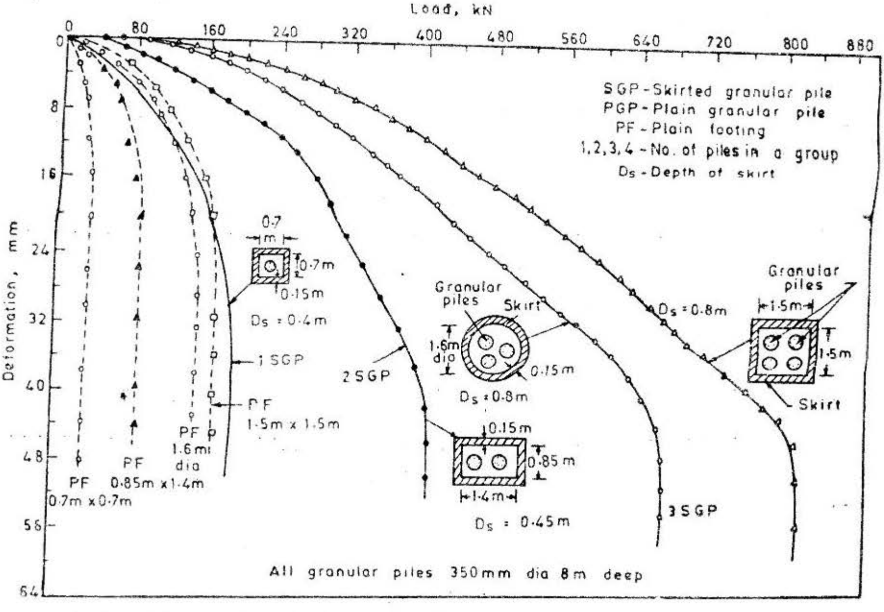

FIGURE 5 Load deformation behaviour of single and groups of collectively skirted granular pile at Site -I

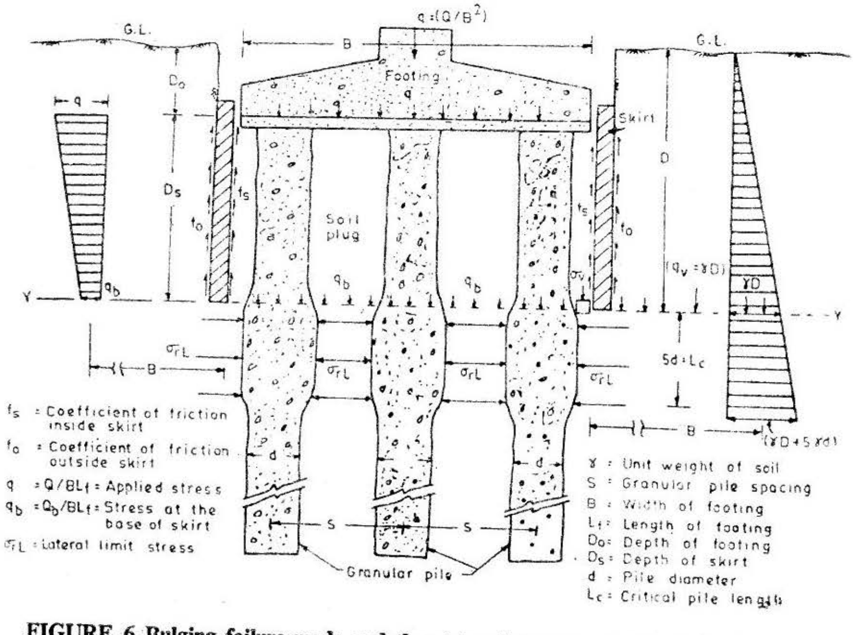

FIGURE 6 Bulging failure mode and the state of stresses at ultimate load in skirted granular pile group

The granular pile is assumed to derive its capacity from the restraint provided by the surrounding soil deposit against the lateral building of the granular pile. When a single granular pile is subjected to sustained vertical loading on the pile top it is assumed that the granular pile fails in bulging at the instant when the lateral induced stress, σ_r in the pile body by the applied vertical load, q of the pile top, attains the ultimate values σ_{rL} and q_{ult} respectively. Further, it is assumed that the depth of bulging hence, forth termed as the critical pile length Lc, (limited to 4-5 pile diameter, as indicated in Figs. 7 and 8) or depth of bulging failure mode is analogous to the expansion of a cylindrical cavity of diameter and height Lc. Due to gradual increase in applied load, q and consequently σ_r in the pile body the cylindrical zone around the bulged pile (Fig. 7) will pass into the state of plastic equilibrium and beyond the zone of plastic equilibrium the soil is considered to remain in elastic equilibrium condition. The applied load q is assumed to be shared between the granular pile q_p and the surrounding soil q_s in proportion to their respectjve elastic moduli E_p and E_s.

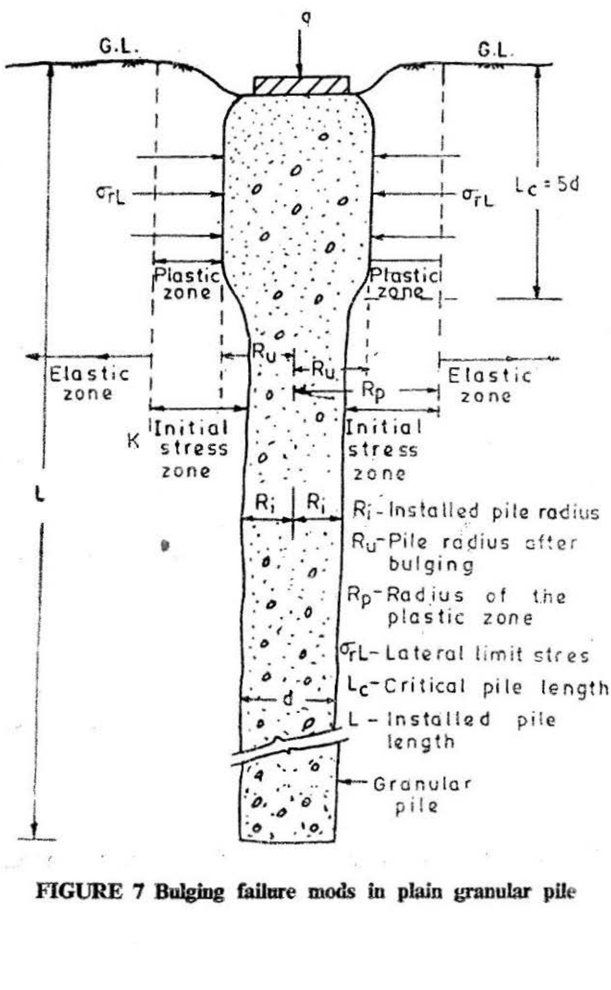

FIGURE 7 Bulging failure mods in plain granular pile

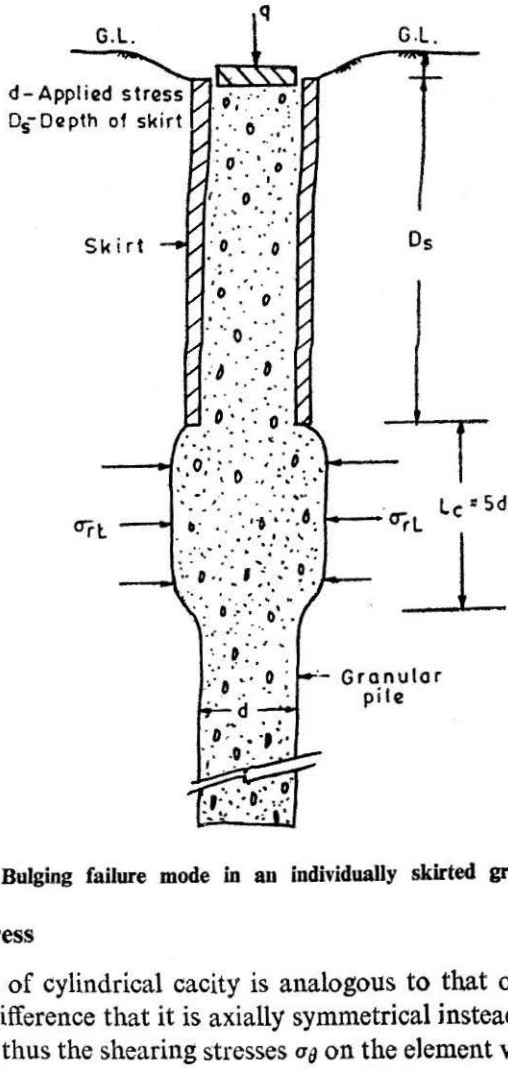

FIGURE 8 Bulging failure mode in an individually skirted granular pile

Lateral limit stress

The problem of cylindrical cacity is analogous to that of the spherical cavity with the difference that it is axially symmetrical instead of spherically symmetrical and thus the shearing stresses σ_θ on the element vanish. Therefore, for cylindrical cavity, the equation of equilibrium for axial symmetry is given by

$$\frac{\partial \sigma_r}{\partial_r} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \tag{3}$$

Satisfying the condition of rupture and the limit equilibrium the solution of equation 3, can be put in the form (Rao 1982).

$$\sigma_{rL} = cF_c' + \sigma_m F_q' \tag{4}$$

where F_c' and F_q' are dimensionless cylindrical cavity expansion factors (Fig. 9).

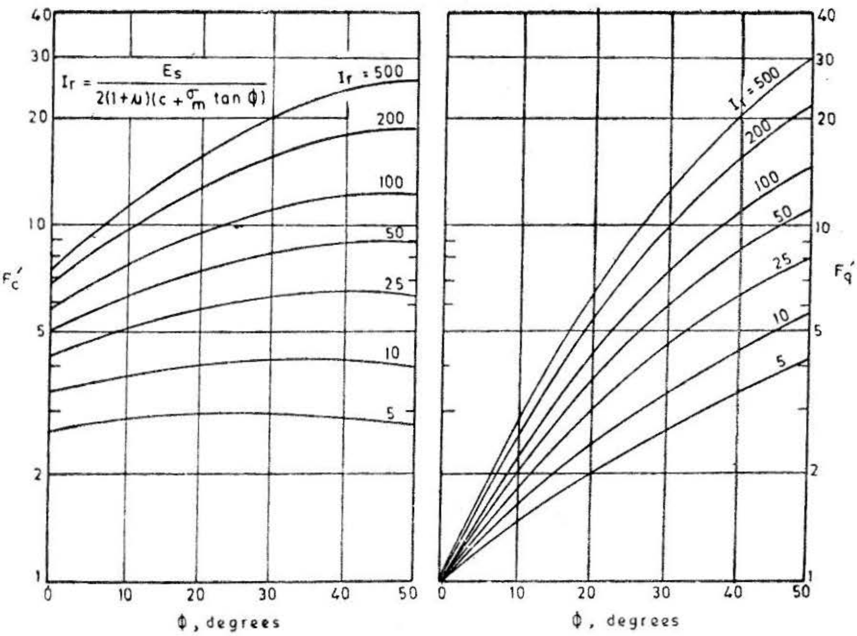

FIGURE 9 Cylindrical cavity expanson factors (After Vesic 1972)

The values of these dimensionless factors F_c' and F_q' are dependent upon angle of internal friction, ϕ and iigidity index, I_r (Eq. 5).

$$I_r = \frac{E_s}{2(1+\mu)(c_u + \sigma_m \tan \phi)} = \frac{G}{S} \tag{5}$$

σ_m being mean normal stress, G the shear modulus and S is the shear strength of soil.

For cohesive soil with $\phi = 0, \mu = 0.5$ and $k_o = 1$, $F_q' = 1$, and

$$F_c' = 1 + Ln\, I_r \tag{6}$$

The lateral limit stress σ_{rL} (Eq. 4) reduces to

$$\sigma_{rL} = c(1 + Ln + I_r) + \sigma_m \tag{7}$$

It may be noted that F_c' is the same as k given by equation 2 proposed by Hughes and Withers (1974) for clays.

Correction for the elasrtic soil Modulii

The measured elastic modulus for the pile material, E_p and for the surrounding cohesionless soil, E_s increase with increase of confining stress (DeBeer, 1967). For the case where initial ground stress is isotropic, the modulus of deformation is found to vary as square root of initial ground stress (Lambe and Whiuman, 1969, Rao 1982). To account for the actual state of stress on modulus of deformation it is assumed that the soil modulus varies with square root of the mean normal stress σ_m. Thus the measured soil modulus, E_s is corrected (Eq. 8) as

$$E_s' = E_s \left[\frac{\sigma_m}{\sigma_1} \right]^{0.5} \tag{8}$$

Thus the rigidity index is given by Eq. 9.

$$I_r = \frac{E_s'}{2(1 + \mu)(c + \sigma_m \tan \phi)} \tag{9}$$

where E_s and E_s' are the measured and corrected soil modulii, σ_m is the mean normal stress at a depth z equal to critical pile length Lc and σ_1 is the normal stress taken as 100 kN/m^2.

Load shared between the pile and the soil

In actual practice the applied load, q is shared between granular pile and the surrounding soil. Because of this, the experimental structures and model tests in which only pile top is loaded and surrounding soil is maintained in vertical state of stress do not represent the true field condition. Under such a circumstance a different state of shear between the pile and surrounding insitu soil exist (Floss, 1979). Attempts have been made in the recent past to assess the magnitude of load distribution between pile and soil (Miggio 1978, Broms 1979, Bauman and Bauer, 1974, Datye and Nagaraju 1975). However, these have been inconclusive (Rao, 1982).

In the present analysis, the total applied load Q is assumed to be shared by the granular pile, Q_p having across sectional area after installation as A_p, and the surrounding soil, Q_s under the footing having an area A_s. If A is the total area of the footing base then the applied lead Q on the area A may be expressed as

$$Q = Q_p + Q_s \tag{10}$$

Within elastic limit the unit stresses, q_p and q_s shared by the pile and the soil respectively are sssumed to be proportional to their respective modulii E_p and E_s and are given by

$$\frac{q_p}{q_s} = \frac{E_p}{E_s} \tag{11}$$

Where (q_p/q_s) is the load concentration factor and (E_p/E_s) is the soil pile stiffness ratio. If α is the relative pile area defined as (A_p/A) and q is the total applied stress, then from equation 10 and 11 in terms of unit stress (Rao 1982). Based on large number of insitu tests, Rao and Ranjan (1985) observed that the plastic yield point (ultimate load) of cohesionless soil deposit reinforced with granular piles, reaches at a deformation of 10%

of pile diameter. At the corresponding deformation of the soil estimation of q_s value based on Eq. 11. is likely to yield conservative value.

It can be shown that

$$q_p = q \left[\frac{E_p}{aE_p + (1-a)E_s} \right] = q\ (E_p/E_{eq}) \tag{12}$$

and

$$q_s = q \left[\frac{E_s}{aE_p+(1-a)E_s} \right] = q\ (E_s/E_{eq}) \tag{13}$$

where $\quad E_{qs} = aE_p + (1-a)E_s \quad$ (14)

In terms of soil-pile stiffness ratio or modular ratio (E_p = m E_s) and relative pile area a the load distribution between pile and soil equation 12 and 13 may also be expressed (Rao, 1982).

$$q_p = q \left[\frac{m}{1 + (m-1)a} \right] \tag{15}$$

$$q_s = q \left[\frac{1}{1+ (m-1)a} \right] \tag{16}$$

The load distribution between the pile and the soil can therefore be computed from equation 12 and 13 having known the pile and the soil modulii E_p and E_s and the equivalent modulus E_{eq} of the composite soil mass. It is further indicated from equation 15 and 16 that the distribution of the applied load between the pile and the surrounding soil is found to depend upon the soil-pile stiffness ratio or the load concentration factor m and the relative pile area of the soil-pile system.

Ultimate load capacity of Soil-Pile system

The lateral limit stress σ_{rL} can be estimated from equation 4. For cohesionless soil (c = 0), $\sigma_{rL} = \sigma_m F_q'$. Thus, knowing the value of σ_m' I_r and F_q' (Fig. 9), the ultimate load capacity is then given by equation 17.

$$q_{ult} = K(\sigma_m F_q') = K\ \sigma_{rL} \tag{17}$$

where K is a coefficient of lateraearth pressure in the granular pile material which is assigned a value equal to 6 (Rao, 1982).

The mean normal stress σ_m increases to σ_m' due to the applied load q of which qs is shared by the soil surrounding the granular pile, thus

$$\sigma_m' = \tfrac{1}{3}\ (1+2K_o)q_s \tag{18}$$

Thus the increase in granular pile capacity q'_{ult} is given by

$$q'_{ult} = K\ \sigma'_{rL} = K\ (\sigma_m' F_q') \tag{19}$$

Hence the ultimate load capacity of a single granular pile having A_p as the area of the installed pile is given by

$$Q_{ult_1} = (q_{ult} + q'_{ult})A_p \tag{20}$$

$$Q_{safe} = \frac{1}{\text{Factor of safety}} (Q_{ult_1}) \tag{21}$$

For cohesive soil ($\phi = 0$, $\mu = 0.5$) as given by equation 6 cavity expansion factor F'c is equal to ($1+\text{Ln } I_r$) hence equation 7 is rewritten as

$$\sigma_{rL} = [\sigma_v + c\,\{1 + Ln(I_r)\}] \tag{22}$$

or

$$\sigma_{rL} = [\gamma_{sub}.\, L_c + c\,\{1+Ln\,(I_r)\}] \tag{23}$$

The value of I_r varies between 10 to 300 for soft to stiff saturated clays (Vesic, 1972). The corresponding values of ($1+\text{Ln } I_r$) is found to vary between 3.72 to 7. Thus a value of 5 for ($1+\text{Ln } I_r$) for soft to medium stiff clays can be suitably adopted. Hence, equation 23 is written as

$$\sigma_{rL} = (\gamma_{sub}\, L_c + 5c) \tag{24}$$

It may be recalled that initial lateral ground stress ($\sigma_{ho} = \gamma_{sub}\ L_c$) taken as 2 c_u by Hughes *et al.* (1975). Also Mori (1979) assigned a value equal to 0.5 γH (where H = 5 m). Thus ultimate load is

$$q_{ult} = K\,\sigma_{rL} = K(0.5\,\gamma_{sub}\, L_c + 5c) \tag{25}$$

and increase in ultimate load due to contribution of applied load qs

$$q'_{ult} = K(q_s + 5c) \tag{26}$$

Hence

$$Q_{ult_1} = q_{ult} + q'_{ult}) \tag{27}$$

$$Q_{safe} = \frac{Q_{ult_1}}{FS=2-3} \tag{28}$$

Ultimate capacity of skirted soil plug reinforced with granular piles

In the analysis of ultimate capacity of skirted soil plug reinforced with granular pile above the plane YY (Fig. 6), it is assumed that the applied load, Q is transmitted through the footing to the reinforced soil plug which is resisted by the reinforced subsoil below the plane YY and the resistance offered due to friction between the soil plug and skirt interface.

When the soil plug is subjected to a uniform intensity of strees q, the footing cast on the soil plug undergoes a little deformation in the initial stages of loading while the skirt around the soil plug remains stationary.

With further increase of load due to friction between the soil plug and skirt interface the soil plug, skirt and the footing move as one unit. The effect of q in computation of inside friction is thus ignored. The resistance offered due to outside friction on the skirt wall may also be ignored in view of small depth as compared to the width of footing. Since depth of the skirt D_s is equal to either half of the width of footing B or 5 times the instal led pile diameter whichever is less (Rao, 1982) Figs. 1 and 6.

$$q = q_b + \frac{\frac{1}{2}\gamma k_o D_s \tan\phi \, 2(B+L_f)D_s}{BL_f} \tag{29}$$

As the shearing plane between the soil plug and skirt interface is likely to be away from the inside interface of the skirt hence, ϕ is taken equal to friction angle instead of δ. In equation 29 the term ($\frac{1}{2}\gamma$ $k_o D_s$) is the average hoirzontal stress σ_h, on the interface of the skirt.

In case of rigid cylinder filled with sand and loaded with a ram it has been demonstrated (Gupta *et al.*, 1972) that the horizontal stress distribution in terms of applied stress expressed in percent, varied from 38 percent at the base of the loading ram which reduces to 32 percent and 25 percent respectively at depth, equal to B/2 and B respectively (B being the width of the loading ram. Further, about 95 percent of the applied stress is trans ferred to a depth equal to B/2 of the loading ram. This value reduces to 70 percent and 20 percent of the applied stress at depth equal to B and 2B of the loading ram). If the loading of the sand column confined by a rigid cylinder and loaded by the ram is considered analogous to the loading of the reinforced soil plug confined by a rigid skirt and loaded through the reinforced cement concrete footing the horizontal stress, σ_h on the interface of the skirt wall may conservatively be taken as 40 percent of the applied stress. Therefore the term ($\frac{1}{2}\gamma D_s k_o$) in equation 29 is replaced by 0.4 q which is the average horizontal pressure on the skirt wall interface. Therefore equation 29 is

$$q = q_b + \frac{0.4q \tan\phi \, 2(B+L_f) D_s}{B L_f} \tag{30}$$

or

$$q = \frac{q_b B L_f}{B L_f - 0.8 D_s \tan\phi(B+L_f)} \tag{31}$$

It may be noted that the stress q_b on the plane YY reaches its ultimate value $(q_{ult})_1$ when the stress q becomes critical $(q_{ult})_2$. Where $(_{ult})_1$ is the ultimate load bearing capacity of the subsoil reinforced with granular piles below the plane YY Fig. (6) and $(q_{ult})_2$ is the ultimate load capacity of the skirted footing above the plane YY. Thus

$$(q_{ult})_2 = \frac{(q_{ult})_1 B L_f}{BL_f - 0.8 D_s \tan(B+L_f)} \tag{32}$$

$$q_{safe} = \frac{(q_{ult})_2}{F.S. = 3} \tag{35}$$

Comparison of observed/predicted ultimate loads

The ultimate loads obtained from insitu load tests of plain and skirted granular piles single and in groups have been compared with the computed ultimate loads from the soil properties.

Ultimate loads

The ultimate load carrying capacity of plain and skirted granular piles, single and in groups obtained from load settlement curves (defined as the load at which the load settlement curves becomes steep straight tangent) have been tabulated (Col. 6, Table 1). The corresponding ultimate load computed analytically have also been shown in Col. 7, Table 1 and the ratio of experimental/analytical loads worked out in Col. 8. This ratio is noted to vary in a narrow range of 0.92 to 1.18 indicating a good degree of correspondence.

Failure mode of granular piles

The granular piles are noted to follow bulging failure mode when loaded to their ultimate capacity. To confirm this a 2 plain pile group, having 250 mm dia and 3.5 m length was observed after load testing at site I by carefully removing the soil from the sides and the front. The diameter of the bulged piles at depth 0.30, 0.50 and 1.0 m below the top were noted as 300, 340 and 320 mm respectively. Similar observations were made on individually skirted 2-pile group. Hughes and Withers (1974), Mokashi *et al.* (1976) in model tests and Hughes, Withers and Green Wood (1975) and Thorburn (1983) on actual piles cast in the field and loaded upto their ultimate capacity also reported similar observations.

Critical pile length

Critical pile length, L_c which is the depth of bulging is assumed to be equal to 4-5 pile diameter in the analysis. In case of 250 mm diameter plain granular piles after the insitu load test were examined by carefully removing the surrounding soil. The increase in pile diameter i.e. bulging of the pile was noted upto 1.02 m incase of collectively skirted piles whereas in case of individually skirted piles it was found to be 1.17 m. This indiates that the depth of bulging is limited to 4-5 pile dia. Theoretically, for granular piles installed in soft clay the critical length/dia ratio is shown to be 4.0 (Hughes and Withers, 1974). Williams (1969) indicated this ratio to vary between 1.9 to 2.33. However, the critical length of pile in model tests were found to be 2.85 the pile dia (Mokashi *et al.* 1976) and beyond this depth the pile is not stressed. Thus, the assumption of criticalpile length as 4-5 studies the pile diameter is in order.

TABLE 1

Comparison of observed and computed loads

Site	Insitu	Pile dia. (mm)	Pile length (m)	P/IS/CS*	Ult. load experi-mental (KN)	Ult. load analyti-cal (kN)	Ratio (Expt. load) Ana.load
I	Single pile	250	3.5	P	85.0	80.9	1.05
	Group of 2	250	3.5	P	190.0	160.0	1.18
	Group of 3	250	3.5	P	265.0	240.0	1.10
	Group of 4	250	3.5	P	350.0	320.0	1.09
	Single pile	350	8.0	P	140.0	155.3	0.90
	Group of 2	350	8.0	P	280.0	310.6	0.90
	Single	350	8.0	CS	200.0	220.0	0.90
	Group of 2	350	8.0	CS	420.0	427.2	0.90
	Group of 3	350	8.0	CS	680.0	646.4	1.05
	Group of 4	350	8.0	CS	830.0	875.0	0.94
	Single	250	3.5	IS	120.0	117.0	1.02
	Group of 2	250	3.5	IS	240.0	234.0	1.02
	Group of 3	250	3.5	IS	380.0	351.0	1.08
	Group of 4	250	3.5	IS	480.0	468.0	1.02
II	2 Group	350	3.5	P	370.0	348.8	1.06
	4 Group	350	3.5	CS	700.0**	980.0	—
III	Single	600	9.0	P	330.0	360.0	0.91
	2 Group	600	9.0	P	665.0	720.0	0.92
IV	2 Group[1]	600	9.5	P	827.0	820.0	1.01
	2 Group[2]	660	14.0	P	827.0	810.0	1.02

*P = plain granular pile, IS = *individual skirted granular pile*,
CS = collectively skirted granular pile

**Pile could not be loaded beyond 700 kN due to inadequate kentledge.

1. Furnace oil tank site
2. Crude oil tank site

Group efficiency

The group efficiency factors from tests on plain or skirted single and pile groups (with piles tested at spacing of three piles diameter only) are shown in Table 2. The group efficiency factor is found to vary between 0.82 to 1.13. Thus group efficiency factors equal to one for piles at a spacing of three pile diameters may be adopted for design purposes in the case of granular piles for both plain or skirted, compressive loads.

TABLE 2

Group efficiency factors for skirted granular Piles for tests at site I (with piles at three Diameter spacing)

Single pile or pile group	Pile dia. (mm)	Pile length (m)	Observed ultimate load (kN)	Ultimate load of single pile in a group (kN)	Group efficiency	*P/CS/IS	Material of skirt
Single Pile	350	8.0	200.00	—	1.00	CS	Reinforced cement concrete
Group of 2	350	8.0	420.00	210.0	1.05	CS	-do-
Group of 3	350	8.0	680.00	226.0	1.13	CS	-do-
Group of 4	350	8.0	830.00	207.5	1.03	CS	-do-
Group of 4	250	3.5	380.00	95.0	1.00	CS	Timber pile
Group of 5	250	3.5	390.0	78.0	0.82	CS	Brick panel (Prefabricated)
Group of 4	250	2.5	380.0	95.0	1.00	CS	Pipe units
Single pile	250	3.5	120.0	—	1.00	IS	Mild steel pile
Group of 2	250	3.5	240.0	120.0	1.00	IS	-do-
Group of 3	250	3.5	380.0	126.6	1.05	IS	-do-
Group of 4	250	3.5	480.0	120.0	1.00	IS	-do-
Single	350	8.0	140.0	—	1.00	P	
Group of 2	350	8.0	280.0	140.0	1.00	P	
Single	250	3.5	85.0	—	1.00	P	
Group of 2	250	3.5	190.0	95.0	0.89	P	
Group of 3	250	3.5	265.0	88.0	0.92	P	
Group of 4	250	3.5	350.0	87.0	0.92	P	

*P = plain granular pile, IS = individually skirted granular pile,
CS = collectively skirted granular pile.

Ground Improvement

On cohesionless sub-soil deposits reinforced with plain and skirted granular piles under compressive load at site 1 the ultimate loads corresponding to yield point have been tabulated (Table 3), and compared with

TABLE 3

Improvement in ultimate load capacity of the virgin soil due to granular pile reinforcement at site I

Insitu Tests	Ultimate load (kN)			Percent improvement in ultimate load capacity due to reinforcement by		
	Plain footing equivalent to pile cap on virgin soil Q_s	Plain granular Q_p	Skirted piles Q_{sk}	Plain piles over virgin soil $(Q_p-Q_s)/Q_s$	Skirted pile over plain pile $(Q_{sk}-Q_p)/Q_p$	Skirted pile over virgin soil $(Q_{sk}-Q_s)/Q_s$
	Granular piles 250 mm dia.; 3.5 m deep indiviually skirted at three diameter spacing					
Single pile	18.0	15.0	120.0	427.7	26.3	566.7
Group of 2	49.0	190.0	260.0	287.7	26.3	389.3
Group of 3	85.0	265.0	380.0	214.7	43.4	347.0
Group of 4	115.0	350.0	480.0	204.0	37.1	317.0
	Granular piles 350 mm dia., 8 m deep collectively skirted at three diameters spacing					
Single pile	36.0	140.0	200.0	288.8	34.3	455.5
Group of 2	92.0	280.0	420.0	204.3	50.0	356.5
Group of 3	160.0	—	680.0	—	—	325.0
Group of 4	180.0	—	830.0	—	—	361.1
	Granular piles, 250 mm dia, 3.5 m deep collectively skirted using brick panel skirting at three diameter spacing					
Group of 5	100.00	—	390.0	—	—	290.0
	Granular piles 250 mm dia., 3 m deep collectively skirted					
Group of 2	140.0	370.0	—	164.2	—	—
Group of 4	376.0	—	Greater than 1000 kN	—	—	Greater than 167.0

ultimate loads of the virgin sub-soil deposits from footing tests having the same dimension as pile caps. The study of Table 3 indicates that the ultimate load capacity for the virgin ground reinforced with single granular piles or with piles of groups of 2, 3 and 4 plain piles increases from 164 percent to 427 percent over the ultimate load of virgin sub-soil without any pile reinforcement. The improvement is further augmented when the piles are skirted. This increase is noted to vary between 290 percent to 566 percent which is significant.

In the case of clayey silt at site III the increase is found to be 209 percent over the bearing capacity computed from undraned shear strength of the deposit. However, in the case of soft clay deposits at site IV, the improvement is found to be 363 percent. The data thus indicates that there is a significant improvement in ultimate load capacity of weak sub-soil deposits when reinforced with plain/skirted granular piles.

Types of skirting

Different types of skirts have been used in the present study namely (*i*) reinforced cement concrete with mild steel bars; (*ii*) reinforced cement concrete with G.I. or M.S. sheet reinforcement: (*iii*) prefabricated brick pannels; (*iv*) prefabricated pipe unit skirting; (*v*) precast cement concrete piles, steel/hume pipe piles or timber piles. The choice of a particular type of skirt is dependent upon its utility based on design requirement, availability of the local materials, equipment for construction and local experience. The skirted may be designed sufficiently rigid to provide confinement to bthe soil plug.

Test results of various types of skirting at site I were noted to show about identical stress deformation curves indicating similar behaviour. Typical results are given in Table 2. Thus, any convenient type of skirting could be used which may be sufficiently rigid and may be able to provide required confinement.

Conclusions

Safety against shear failure and settlement within limits are two essential requirements for satisfactory behaviour of any foundation. The present study is limited to ultimate capacity of granular piles only, the settlement aspects are discussed elsewhere (Rao and Ranjan, 1985). Based on the present analytical and full scale experimental investigations related to the skirted granular piles subjected to sustained vertical loads in weak sub-soil deposits, the following conclusions are drawn :

1. The theoretical predictions of ultimate load capacities of plain single and group of granular piles and skirted granular pile foundation, based

on bulging failure phenomenon and lateral limit analysis, have demonstrated a good agreement with observed insitu full scale tests in weak soil deposit.

2. Weak sub-soil stratum reinforced with granular pile group demonstrated a significant increase in bearing capacity over the un-reinforced virgin sub-soil stratum. The increase is further augmented when the pile groups are provided with skirt.

3. The group efficiency factors for piles plain, individually or collectively skirted at three pile diameters, is found close to unity.

4. Full scale tests carried out on granular pile groups at different sites with skirts of different materials indicated that skirt of any material e.g. brick panels, closely driven pipes, prescast pre-stressed concrete piles and timber piles may be provided which may be reasonably rigid, However, in case closely spaced piles are used to form skirt the heads of all these piles need to be connected properly at the foundation level. Prefabricated pipe unit skirting has the advantage that these are reretrievable and hence can be reused.

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