Behavioural Prediction and Performance of Structures on Improved Ground and Search for New Technologies*

by *B.G. Rao***

INTRODUCTION

The best buildable lands have already gone thus one has to make the most of what is left. It is also expected that when the industry disperses into interior, the population too shifts there, and naturally the foundation technologies also move into those remote areas. These are generally found to be filled up sites, including low lying water logged waste lands, creek lands with deep deposits of soft saturated marine clays having very low strength. The problem, is further aggravated when design loads are high and the site is situated in seismic zones. The traditional foundation techniques in such situations are found to cost more than the super structure and in many situations, can not be build at all.

Designer's options, most often had been to remove and replace the unsuitable soils or very often tempted to take recourse to deep piling, pier or well foundation to transfer loads to lower most stable strata (Fig.1). There again lack of knowledge about the ground conditions, induce them to make no distinctions between the implications of methods of installations of pile or that of advancing of pier or well. Ignorant about the rate of developments of negative drag which is almost a certanity when deep foundations are placed in soft compressible strata, over conservation in design gain favour.

Many a time when designing a pile foundation, even the contribution of pile cap is ignored adding further to the factor of safety, probably to counter the factor of ignorance. In such situations, there are three options left to the designer ;

*Abandon or reject the site and search for better one.

*Reduce the design load by reducing the number of storeys or use light weight material.

*To adopt efficient, speedy and cost effective methods of ground treatment.

The first two options are not acceptable in the context of present technological growth since the new techniques are available to force the soil to behave according to the project requirements rather than having to

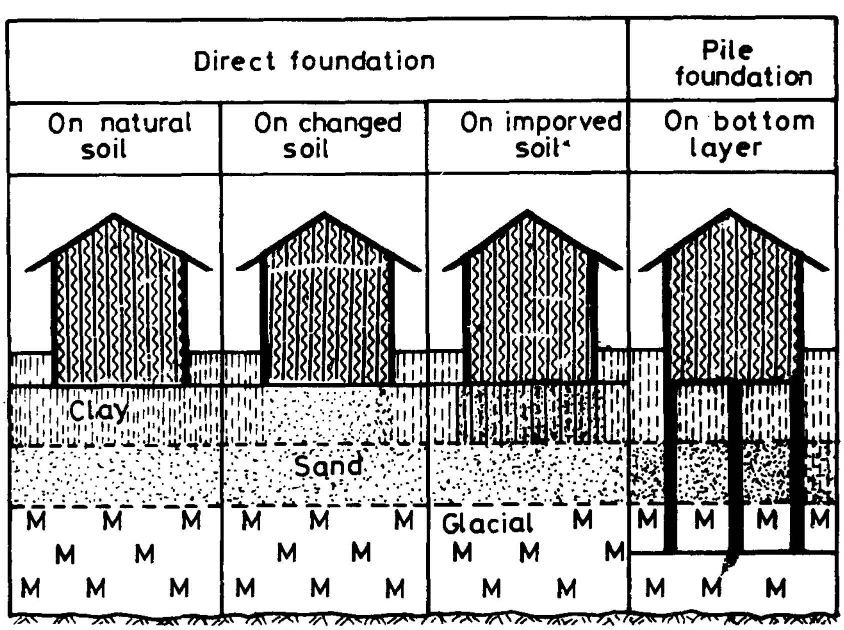

Fig.1 GROUPS OF FOUNDATION METHODS (Harikainen 1983)

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change the project to meet the soils limitations.

GROUND IMPROVEMENT TECHNIQUES -A WIDE CHOICE

Techniques of ground improvements are now sufficiently well developed to transform a weak soil into strata of desired strength and compressibility (Fig.2). The assessment of ground conditions provide direct information on the degree of ground improvement. Many of the ground improvement techniques such as the use of preload actuated sand drains, sand wicks, band drains granular piles, lime piles, also serve as means of expediting excess pore water pressures and accelerating settlement rates in addition to the reinforcement effect they provide. Since considerable percentage of settlement takes place during the stage of ground improvement and construction itself, such foundations are often trouble free from the long range maintenance point of view. There are equal or greater number of options available in other category also and choice of the method of ground treatment often dictated by value judgment of the designer is based on techno-economic considerations.

Foundations on improved or treated soils are designed basically according to the principles of foundation design as on natural soils since the improved soil is still soil. The basic difference is in working out the design parameters which are more difficult for improved soils than for natural soils. This is because of the fact that improved soil layers are in many cases very non-homogeneous and these may have some properties that natural soils do not have, (Hartikainen, 1981; Green & Padefield 1983l; Johnson et al 1983). For example reinforced ground can take tension and much more shear and compression than natural soil. The main purpose of ground treatment measures are :

- Increasing the bearing capacity
- Reducing the overall total/differential settlements
- Decreasing the permeabilities
- Increase the soil resistance to horizontal shear
- Control the movement of cellular structures
- Improve the stability of a slope cut in weak soils
- Changing the dynamic response
- Reducing the risk of liquefaction

A number of options such as deep compaction (Smoltczyk 1983) preconsolidation/loading (Jamilokowski et al 1983) soil reinforcement (Schlosser, et. al. 1983) soil stabilization (Broms & Anttikoski, 1983) and soil grouting (Jessberger, 1983), are available as the methods of ground treatment. Further details have been discussed by several investigators e.g. Rao (1982), Ranjan (1988) (Fig.2). There are a number of successful case records of cost-effective foundations resting on ground treated with drains, (Mohan, et. al. 1958 & Harris, 1981, Casagrande & Poulos, 1961; Nonveiller, 1976; sand wicks; Dastidar, et. al. 1969), rope drains (Mohan, et. al. 1977), band drains, geo drains (Broms & Burke & Sanucha, 1981) and alidrains and colbond etc. Such techniques do not need much elaboration at this time. The partial or full replacement of weak soil stratum has also been an age old approach, though with rather diminishing trend of application in view of the current accent towards structures with more stringent performance requirement.

The choice of a particular method of improvement depends on many factors (Mitchel & Katti, 1981) (a) soil type and its properties (b) volume of the soil to be treated (c) availability of material, (d) equipment and skills, (e) local experience and preferences, (f) cost economics and (g) time frame. The purpose of treatment will establish the level of degree of improvement. It may be noted that the parameters (d), (e) & (f) can not be ignored in relation to vibro compaction, particularly in developing countries, since it involves huge foreign exchange. Further the risk involved when the method in certain soils may reach suitability limit. It is better not to go too close to suitability limit. Figs.3, 4 & 5. However, if it is unavoidable, and when the suitability limit is exceeded, one must weigth the possibility of overcoming such a situation in terms of intensifying the treatment, increasing expenditure on energy and time for consolidation, reducing the acceptance criteria by proposing a modified foundation or structure.

GROUND REINFORCEMENT TECHNIQUES & FAILURE MECHANISM

Conceptually, all the different design approaches could be classified once the mechanism of failure could be understood, and linked with the techniques of ground treatments,Fig.6. The various options of ground

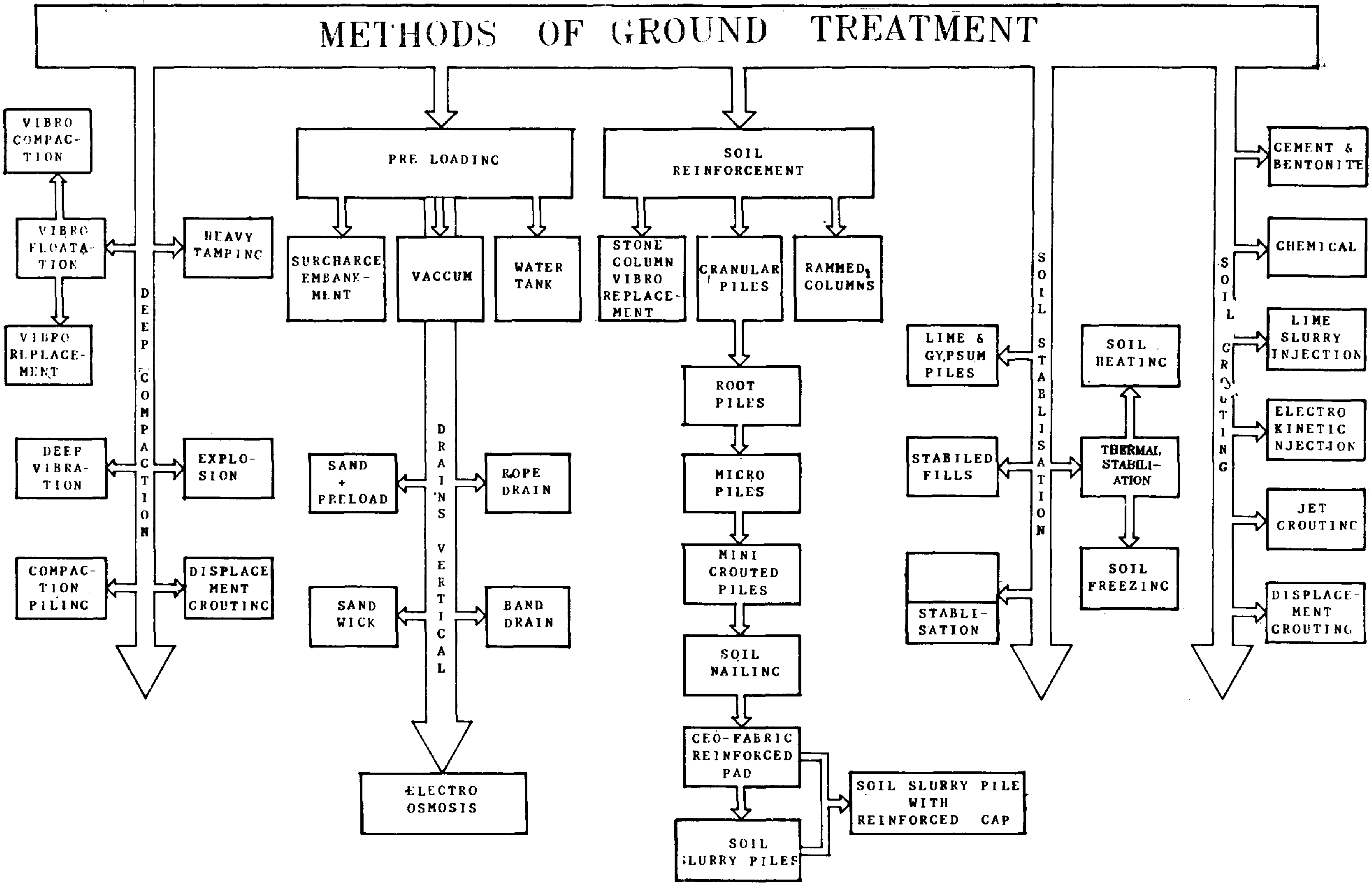


FIG. 2 GROUND TREATMENT METHODS

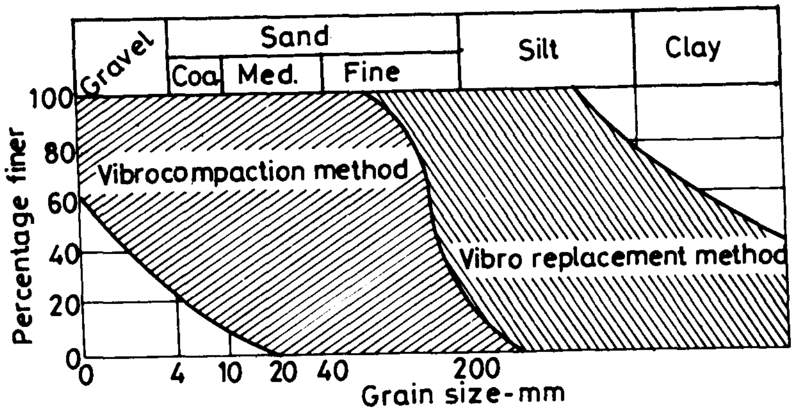


Fig.3 RANGE OF SOILS SUITABLE FOR STABILIZATION
(Baumann & Bauer 1974)

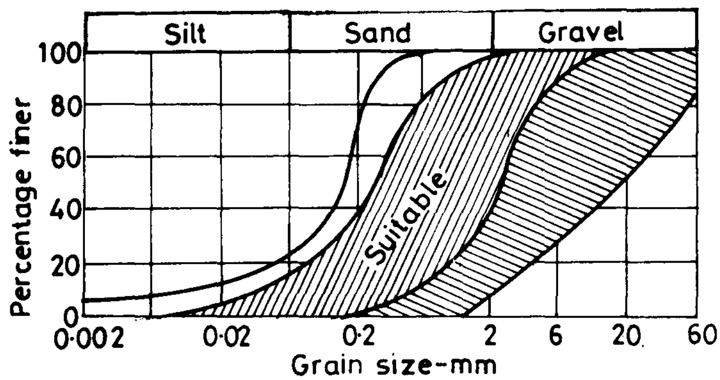


Fig.4 GRAIN SIZE AREA SUITABLE FOR VIBROFLOTAION (Brown 1979)

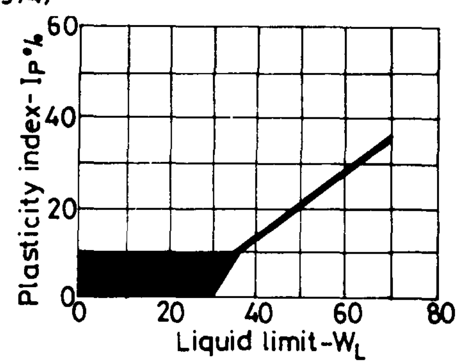
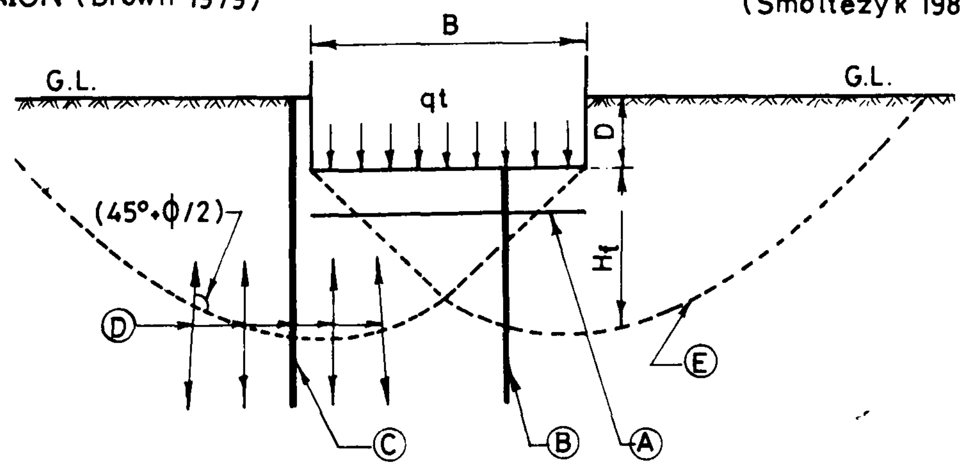


Fig. 5 APPLICABILITY OF HEAVY TAMPING IN THE CASE OF CO-HESIVE SOIL
(Smoltezyk 1983)



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A. Horizontally below the footing as tensile reinforcement length = footing width.(B) depth = 0·8 (B)
B. Vertically below the footing as compressive reinforcement
C. Vertically around the footing as tensile reinforcement depth must be below the potential slip line
D. Direction of minimum principal strain
E. Potential slip line

Fig.6 MECHANISM OF GROUND TREATMENT
(After Tatsuoka & Miki 1982)

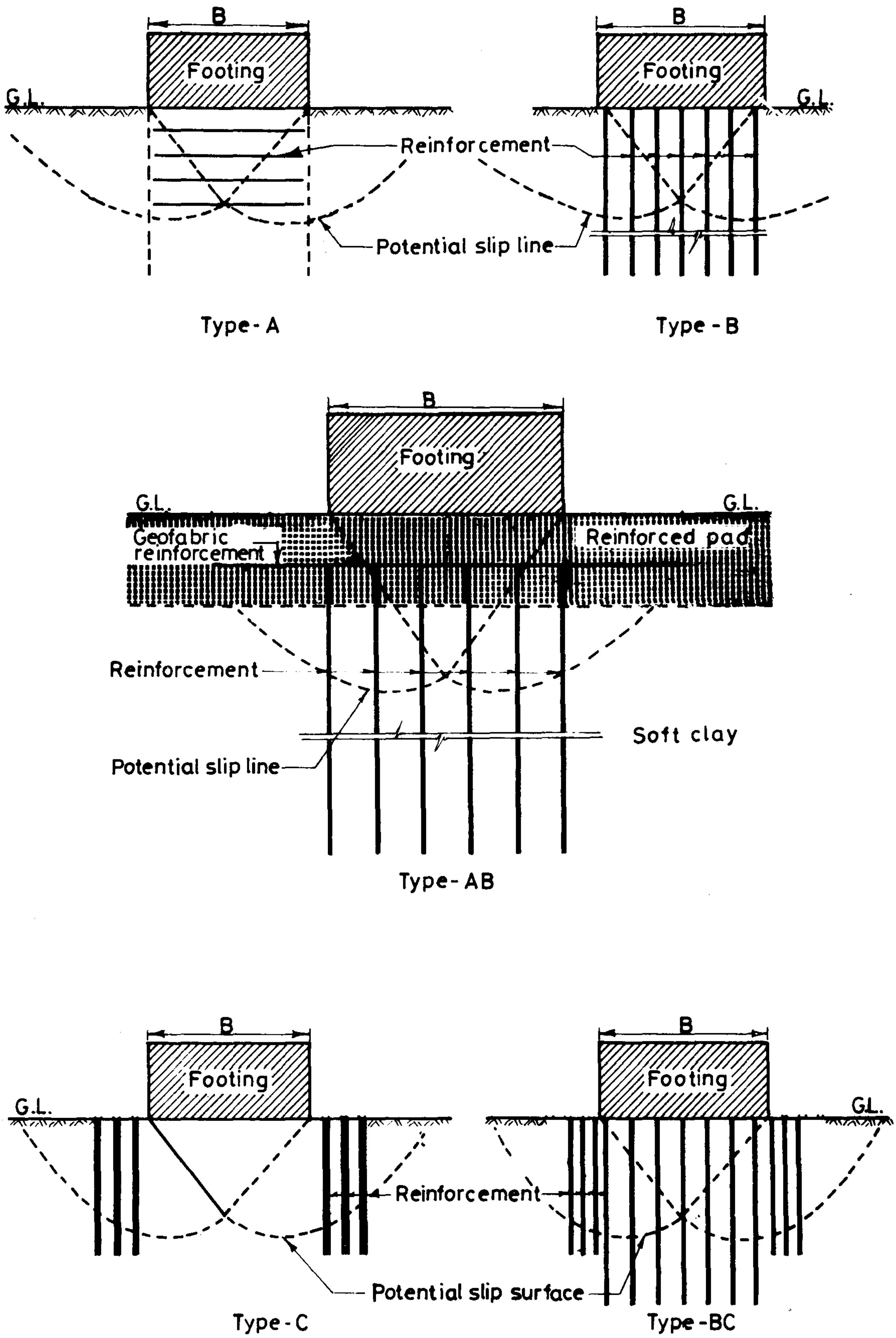


Fig.7 OPTIONS FOR GROUND REINFOCEMENT METHODS

reinforcements have been shown in Fig.7. If the direction of reinforcing bars, are kept parallel to the slip surface, neither compressive nor tensile forces shall induce in the bar, hence ground will not be considered as reinforced, and no improvement in the strength and reduction in compressibility shall be observed. On the other hand the ground will be considered as reinforced when the reinforcing bars are placed parallel to the direction of minimum principal strain (ε_3) and the bars are having rough surface. This will result in the development of tensile stresses in the bar(Fig.6).

Since the direction of ε_3 is not consistent in ground, there may be many methods of placing reinforcement. Type (A) and (C) are two typical methods of reinforcement among many more effective methods. Thus tensile forces induced in the reinforcing bars increase the confining pressure for the adjacent soil mass, hence the minimum principal stress (σ_3) in the adjacent soil mass will increase which result in increase of maximum principal stress (σ_1) and hence increase in bearing capacity. If the bars are placed parallel to the direction of maximum principal strain ($\in_{3f}$) or maximum compression strain, it will represent type (B) method and similar to the ordinary pile group. The various options of ground reinforcements (Fig.2) are presented in Fig.7 and are outlined below:

GROUND STRENGTHENING IMMEDIATELY BELOW THE FOUNDATIONS/STRUCTURE

It is valid for both shallow and deeper depths. Techniques of partial or full replacement or the use of geogrids reinforced soil pad would fall in the category of treatment to shallow depths (Fig. 7 Type A and Fig.8) The minimum length of tensile reinforcement is 0.8 times the footing width and its depth equal to width. Use of sand drains, granular piles, minigrouted piles & bakau or precast spliced piles (cap resting condition), self setting soil slurry piles,jet grouted columns of cement mortar or soil cement are applicable for deeper treatments (Fig.7 Type B)., and placed vertically below the structure as compressive reinforcement. The ground treatment immediately underneath the footing; Type A, Fig.7 and that for deep depth; Type B, Fig.7 could be combined together as shown in Fig. 7 Type AB, Such an option is likely to ensure better performance than Type A or Type B individually. Such an option would include geofabric reinforced pad overlaid on group of granular or self setting soil piles.

Stiffening, strengthening or reinforcing of the ground vertically ground the foundation (Fig.7, Type C) as tensile reinforcement keeping the depth well below slip line is often resorted to confining deeper depths, and prevent buckling of vertical reinforcements. The restraining effect could be provided by providing different types of skirtings, e.g. (a) RCC skirt using mild steel bars, (b) RCC skirt using GI sheet reinforcement, (c) prefabricated brick panel skirting, (d) interlocked pipe pile skirting, (e) steel or hume pipe, timber, precast RCC piles or contiguous cast insitu minigrouted piles. (Rao et al 1979, Rao & Sharma, 1980; Rao, 1982; Ranjan, 1988; Rao & Ranjan, 1990). Many a times, a series of contiguous granular piles/stone columns are also deployed to create similar effect (Rao & Ranjan, 1988) The designer has the option to pick up any one of the solutions which meets the requirement the best. Ground treatment both underneath the foundation as also around it could also be used together (Fig.7 Type B.C.) This would for example include all cases of skirted granular or mini grouted pile foundations.

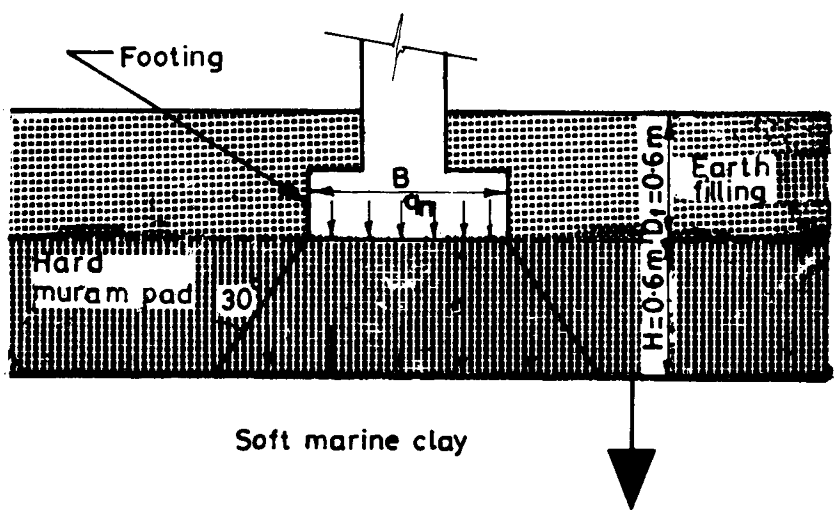


Fig.8 FOUNDATION ON HARD MURRAM PAD UNDER LAIN BY SOFT MARINE CLAY

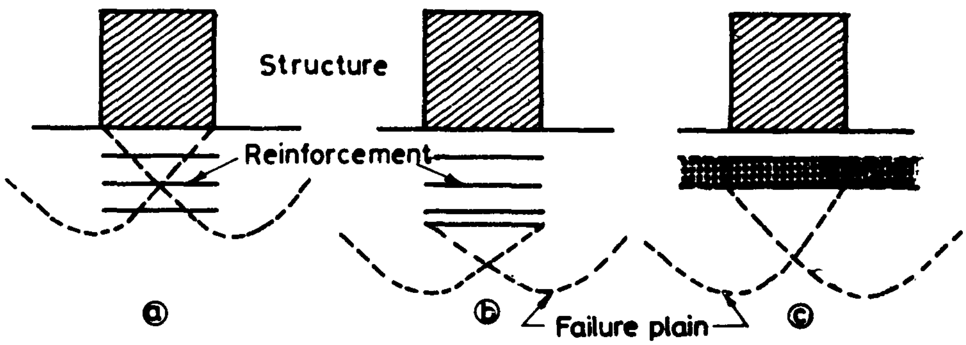
GROUND TREATMENT RESTRICTED TO SOIL IMMEDIATELY BELOW THE FOUNDATION

The most promising solutions in the category would be (a) provision of a compacted stabilised soil pao (Fig.8), (b) trench packed with granular soil (Madhav and Vitkar, 1978) or (c) reinforcing with geosynthetics, Fig. 9 (Fukuda et al, 1987).

The easy accessibility of material for providing compacted soil pad when compared to geo-synthetics naturally provide preference to the former approach. Care should be taken that the material of which the hard pad is made, is not very brittle and impermeable because brittleness leads to progressive failure and impermeability to delayed settlements. However, in cases where large volume of construction is involved and transportation of earth in huge quantities may lead to ecological degradation and time delays, the geo-synthetic approach (Fig.9) deserve preference.

The solutions of the kind suggested above are normally not acceptable when large depths of compressible strata are involved. In such cases designers prefer treatment with granular piles. Various considerations which go into the design of granular piles and into selecting of an appropriate method of construction have been discussed elsewhere (Rao and Bhandari, 1979; Rao and Sharma, 1980; Rao, 1982; Rao and Ranjan, 1983; 1985, Bhandari, 1987; Ranjan, 1988).

Although the ground improvement effected by appropriately spaced vertical granular piles immediately below the foundation may provide adequate load carrying capacity, their performance could be further improved to a significant extent by ensuring that they are made to transfer their loads to deeper depths. If such a load transfer is not achieved the granular piles in the deeper portions will serve merely as drainage media. it has been amply demonstrated that provision of a skirt around the granular piles, restrain them from bulging (Rao and Bhandari, 1980; Rao 1982) (Rao and Ranjan, 1990) thereby facilitating transfer of load to deeper depths. Such bulging has been reported to result upto about five times the installed pile diameter (Rao & Bhandari, 1979; 1980; Rao, 1982, Ranjan and Rao, 1983; Rao and Ranjan 1985; 1988).



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a. Spa cely layered system (After Tatsuoka)
b. Densely layered system (After Tatsuoka)
c. Mattress foundation

Fig.9 GROUND TREATMENT FOR SHALLOW FOUNDATION
(Use of geogrids or partial replacement)

STRENGTHENING THE GROUND AROUND THE FOUNDATION

Decades ago, designers had conceived the idea of controlling the lateral ground displacement around the foundation by introducing vertical barriers. During the construction of a subway in Japan in 1927, it is reported that arrow plates were placed around the foundation to improve resistance and prevent lateral ground displacement. Katoda (1987) reports that such confinement effects, introduced in Nigata city Administration Building and Nagai Electrical Building in 1957 and 1960 respectively, were made strong by binding the loose sandy subsoil with arrow plates used in pit excavation. This prevented their collapse during the 1964 Nigata earthquake whereas, due to the same earthquake, many other buildings collapsed or tilted. Nigata earthquake of 1964, recorded the typical phenomenon known as "appearance of fluidity" (liquefaction). This phenomenon leads to the shaking of foundations.

Examples of application of skirted foundation can be cited, however the number of such examples is not very large. The use of skirted foundation in Japan for the construction of a very tall building is shown in Fig.10.

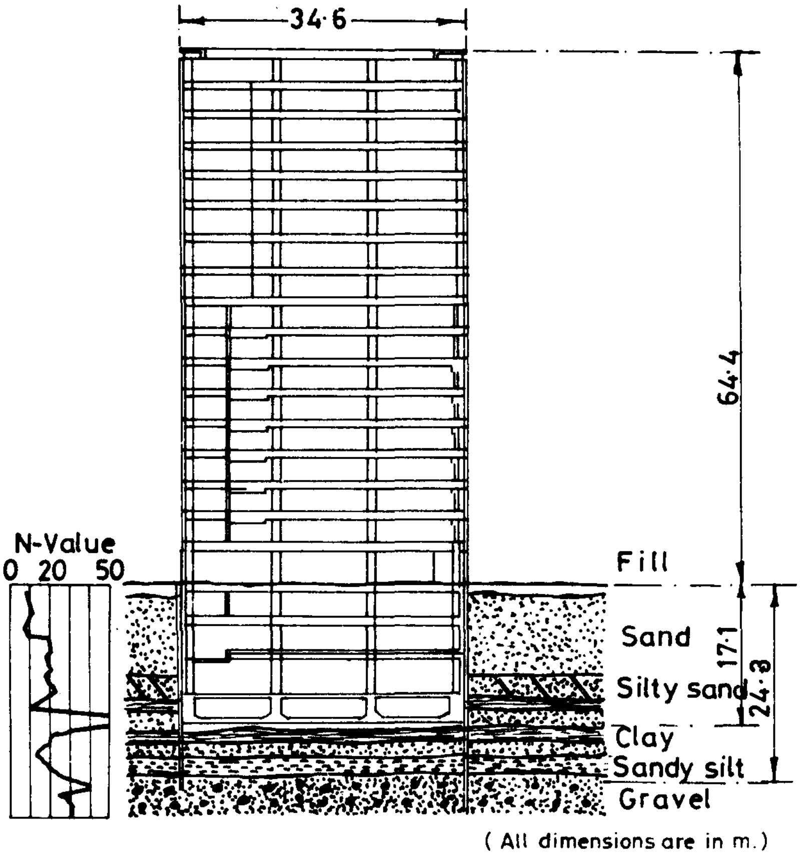


Fig.10 NISHI NIHON BUILDING (Takenaka Koumuten Co.JAPAN)

The sub-soil below the foundation consisted of sand and clay layers of moderate density and the design stress for this building was 31.5 tonnes/m^2. In this example the earth wall around was left during pit excavation protected by RCC wall.

The central Building Research Institute (A.R.1970, 71 and 72) reported studies on skirted footing foundations and demonstrated that their load capacities could be increased manifold with corresponding reduction in settlement. The results of researches (Narhari & Rao 1979) threw up the idea, that, perhaps, in some cases, the need for piling could be eliminated all together by confining traditional foundation with skirt penetrating close to half the width of the foundation.

A considerable amount of further work in the area now stand documented (Rao and Bhandari, 1980; Rao and Sharma, 1980; Rao & Ranjan, 1990) and question is not whether skirt provides the benefit but that how the selection of a particular skirt type is to be made between several options keeping in view the ground conditions and the design expectations.

SIGNIFICANCE AND UTILITY OF SKIRT

The provision of a rigid skirt around a foundation is the commonest approach currently being followed which also turns out to be cost effective in many situations (Rao & Bhandari, 1980; Ugo Picagli 1969). There are a number of case records which establish that, following the approach the stability of super structure could be considerably improved in short and long term. It may also find application in (i) situations where by necessity one has to use comparatively high load for a given settlement such as column footings of framed structures, and (ii) in situations where one has to restrict settlement for a given load such as Radar Antennae, High Tension Transmission line towers et. (Narhari and Rao, 1979). The utility of the skirted foundation can further be extended to foundation for small buildings, industrial floors on clay, sand or silt deposits (Broms et al 1981) besides storage tanks, grain soils and industrial chimneys in sand and gravel or fills. For an efficient and cost effective foundation in weak subsoil deposits several investigators e.g. Rao and Bhandari, 1979; Rao, 1982; Ranjan and Rao, 1985, 1987; 1991 have combined skirt with granular piles individually or collectively. Skirted foundations provide an efficient and safe foundation in earthquake areas since it augment confinement to lateral flow of soil beneath the footing and improved resistance to lateral flow (Rao and Sharma 1980) and have demonstrated benefit of skirting in increasing factor of safety against base shear failure by two to three when the skirted soil plug is reinforced with granular piles (Rao, 1982; Ranjan, 1988).

Polymer grid reinforcement have emerged as powerful means of reinforcing soils to improve bearing capacity of foundations (Fig. 9) and is quite effective in reducing the settlements (Fukuda, Taki and Sutoh, 1987). Concern of cost effectiveness demand that alternatives are simulataneously studied and a comparison of techno-economic suitability is sought. Reinforcement in a sense is also achieved by provision of skirting around foundations thereby providing cost effective to polymer grid reinforcement.

ULTIMATE CAPACITY OF GRANULAR PILES

The analogy of expansion of cylinderical cavity (Vesic, 1972) and bulging failure phenomenon of granular pile in homogenous, isotropic and infinite soil mass has been used to estimate the ultimate bearing capacity of a single pile. The analysis has also been extended to granular pile groups. The basic assumptions and the details of analysis have been provided earlier (Rao, 1982; Ranjan and Rao, 1985; 1987, Ranjan, 1988; Ranjan & Rao 1990, 1991).

The ultimate bearing capacity of a single granular pile installed in a weak sub soil deposit (Rao 1982, Ranjan and Rao, 1987 and 1991) is given by Eq.1

$$q_{ult} = [q_{ult1} + q_{ult2}] \quad \text{....(1)}$$

For cohesionless soil (C = O)

$$q_{ult} = K(\sigma_m + \sigma_m') F_q' \quad \text{....(2)}$$

Hence Eq.1. in case of Cohesion less soils can be written as Eq 3.

$$\text{or } q_{ult} = 1/3\, K (1 + 2K_o) (\sigma_v + q_s)F_q' \quad \text{....(3)}$$

where σ_m is the effective mean normal stress σ_m' is the increased effective mean normal stress, K is a constant which is assigned a value equal to 6. Further, F'_q is the Vesic's dimensionless cylinderical cavity expansion factor found from the chart (Vesic, 1972). Also K_o is the coefficient of earth presure at rest, σ_v is the effective overburden pressure, and q_s is the load shared by ambient weak soil.

For cohesive soils : (ϕ = O, μ = 0.5, K_O = 1)

$$q_{ult1} = K(0.5\, \gamma_{sub}, L_c + 5\, C_u) \quad \text{...(4)}$$

$$q_{ult2} = K(q_s + 5\, C_u) \quad \text{.......(5)}$$

where q_s is the load shared by the ambient ground L_c is the critial pile length equal to 5 times the installed pile diameter d, γ_{sub} is the effective unit weight of the soil and C_u is the undrained shear strength of the clay.

Hence Eq. 1 in case of cohesive soils is taken as sum of Eqs. 4 & 5.

$$q_{ult} = K(10\, C_u + q_s + 2.5\, \gamma_{sub}\, d.) \quad \text{......(6)}$$

$$Q_{ult} = A_p\, (q_{ult1} + q_{ult2}) \quad \text{.....(7)}$$

$$\text{hence } Q_{safe} = \frac{Q_{ult}}{(F.S. = 2 \approx 3)} \quad \text{.....(8)}$$

where Q_{safe} is the safe load and A_p is the area of cross section of granular pile.

SETTLEMENT ANALYSIS

Based on the concept of equivalent coefficient of volume compressibility of the composite mass of the soil and pile material, in case of both the, cohesionless and cohesive soil deposit, a simple method of settlement prediction is also available (Rao, 1982; Rao & Ranjan, 1985, 1988). The method utilizes the properties of granular pile material, and the ambient soil, pile size, spacing and its depth as well as the soil-pile stiffness ratio n have been incorporated. The study has been extended to skirted pile group also.

PLAIN GRANULAR PILES

The total settlement "S" of the improved ground reinforced with partially penetrating granular piles under the footing/raft can be estimated from Eq. 9.

$$S = \Delta L + \Delta H \quad \text{...(9)}$$

were ΔL is the settlement in reinforced layers thickness, L, divided into n layers; and the applied stress is distributed by 2 : 1 method. If qi, m_{veqi} and hi are the applied stress, equivalent coefficient of volume compressibility, and thickness of ith layers in the reinforced layers respectively then the settlement (ΔL) is given by Eq. 10.

$$\Delta L = \sum_{i=1}^{n} q_i' \, m_{veqi} \cdot h_i$$

and the settlement in the unreinforced compressible layers below the pile tips is given by Eq. 11.

$$\Delta H = \sum_{i=1}^{n} q_i' \, m_{vi} \cdot h_i \qquad ..(11)$$

When the granular piles are allowed to penetrate hard stratum the value of ΔH is taken as zero.

Further the settlement in untreated virgin soil strata Δ L' is given by Eq. 12. (Rao, 1982; Rao & Ranjan, 1985, 1988).

$$\Delta L' = \sum_{i=1}^{n} q_i \, m_{vi} \cdot h_i \qquad ..(12)$$

Therefore, the settlement reduction ratio β, according to Rao and Ranjan (1988) is given by Eq.13.

$$\beta = \frac{\Delta L}{\Delta L'} = \frac{m_{veqi}}{m_{vi}} \qquad ...(13)$$

or $\beta = 1/[1 + (m - 1)\,\alpha]$ ...(14)

Where m is the stiffness ratio and α is the Replacement ratio (A_p/A); A being the area of the foundation.

DESIGN PREDICTION

As stated earlier, the foundations on treated sub soil are designed according to the design principles as on natural soils. The main difference is in working out the design parameters which are more difficult for improved soil than for natural soils. it is wellknown that for a satisfacory performance of foundation primarily two criterias, namely the safety against shear failure of the subsoil and settlements, both total and differential are to be kept within permissible limits. Keeping above inview, the design analysis for single and groups of granular piles in cohesionless deposit was developed for the prediction of ultimate bearing capacity and the total and differential settlements of foundation placed in composite soil deposits. The analysis has further been extended to individually and collectively skirted pile groups in both cohesionless and cohesive soils also (Rao, 1982, Ranjan, Rao & Gupta 1985; 1988, 1990 and Ranjan, 1988; Ranjan and Rao, 1991).

The proposed design methodology is unique and superior to empirical approaches (Greenwood, 1970, hughes & Withers, 1974; Hughes, Withers & Greenwood, 1975), since it recognizes the contribution of load shared by the ambient weak ground in the analysis and result in increasing the load carrying capacity of granular piles which is an important feature, not considered in earlier approaches due to analytical difficulties (Thorburn, 1975). This is due to the fact that the response of the soft cohesive soil strengthened by dense granular piles was understood qualitatively till 1975, and the complexity of the soil pile interaction problem did not permit a simple solution (Thorburn, 1975). Further the method for predicting the settlement by Equivalent coefficient of volume compressibilities approach (Rao, 1982; Rao & Ranjan, 1985, 1988) is simple and uses soil parameters which are obtained from the field tests in any sub soil investigation (Goughnour, 1988), in addition, it does not call for computer programming and computer time which is not within the reach of every practising geotechnical engineer particularly at site.

Although the upper bound of settlement could be estimated (Thorburn, 1975) based on the recommendations of Hughes and Wither's (1974) which is related to the compatibility of vertical strains between the dense granular piles and soft cohesive soils. Such an approach would give an indication of vertical ground deformation. This is true when the stress/strain relationship of the pile material and ambient soil are known. It has further been recommended that the settlement of densely packed granular piles penetrating into hard strata, at working load may be expected between 5-9 mm (Thorburn, 1975). However for partially penetrating piles based on field experiences, Thorburn recommended this settlement (5-9 mm) be added to the settlement of the unreinforced ground below pile toe.

The settlement prediction method proposed by Rao (1982), Rao and Ranjan (1985, 1988), is simple in application and fully recognizes the stress/strain relationship of the pile material and the ambient weak soil, in the analysis, because of their respective modulii, E_p and E_s, an important feature found missing in Hughes & Withers (1974) approach. Thus in the settlement prediction approach (Rao & Ranjan, 1985; Rao, 1988), the equivalent coefficient of volume compressibility is ȧn important design parameter wherein the area occupied by the total number of piles and the ambient soil under the footing, besides compressibility of pile material and the ambient soil have been given full weightage.

Further it is interesting to note that the methodology adapted by Rao (1982) and Rao and Ranjan (1985) in development of the settlement analysis for the skirted granular piles, the lateral displacement of the soil or the dispersion of the design load was assumed as 2V : 1H method. Also the lateral displacement of sub soil under the footing was assumed to start from the edge of the footing and in the case of skirted foundations, from the tip of the skirt. The assumption is fully verified from the model test results carried out by Katoda (1987) and presented in Figs. 11 and 12, showing clearly the contours of lateral displacements and isochromatics of photo elastic methods. Further the circumferenctial stress distribution along the depth of skirt have been presented in Fig. 13 showing circumferential stress higher at the tip of the skirt confirming thereby that the distribution of stresses from the plate to the sub soil start from the lower end of the skirt.

While appreciating the analytical model proposed by Rao, 1982; Rao and Ranjan (1985) for computing the settlement in composite ground and its usefulness because of its versatility to accommodate changing sub-soil conditions with depth and based on the parameters which can easily be estimated (Goughnour, 1988). Doubts have however been raised about its limitation in soft cohesive soil deposits where the pile material and ambient soil are assumed to behave as linearly elastic material. In the ultimate analysis what matters is its stress/ deformation behaviour.

STRESS DEFORMATION BEHAVIOUR OF COMPOSITE GROUND

The significance of the above statement is highlighted in Fig. 14. The stress/displacement characteristics of weak cohesionless sub-soil stratum treated with skirted granular piles, with skirt and untreated ground have been presented through curves 3, 2 and 1. Curve 1, represents the result of load test on an untreated cohesionless deposit If the footing is confined by the provision of a skirt, trends to add zone of elastoplastic behaviour between elastic and fully plastic zone, and the stress deformation behaviour follows curve 2. If the ground is treated by granular piles in addition to skirt around the footing, the behaviour of the composite ground improves to curve 3 showing an elastic behaviour upto elastic yield point E followed by a more pronounced elasto-plastic behaviour EP upto the pastic yield point P. until the ultimate load P is reached (Rao, 1982). It is clearly found that virgin ground depicts nonlinear behaviour (Rao and Ranjan, 1988) which can broadly be characterised as elasto-plastic behaviour following the initial elastic range at one hand and terminating into the plastic range on the other, the fact which fully justifies the assumption of elastic behaviour of composite mass in the settlement analysis. Further strengthening by granular piles makes the elasto plastic range more pronounced as in-curve 3. Similar behaviour have been found at two other sites where the sub-soil consisted of soft saturated clays (CH) and clayey silt (CL) deposits (Figs.15 and 16). The above noted behaviour is at variance with that put forward by Goughnour and Bayuk (1979) which deserves to be corroborated by more feedback studies (Rao and Ranjan, 1988). This observation bring out the fact that where bold designs are aimed, chances of failure do not seem as likely in a composite ground as could be feared on virgin situation. It has further been observed that the plastic yield point

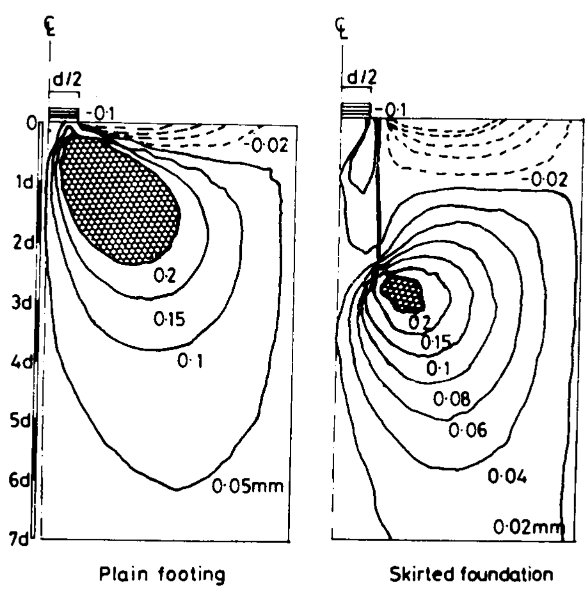


Fig.11 LATERAL DISPLACIMENT OF CONTOUR

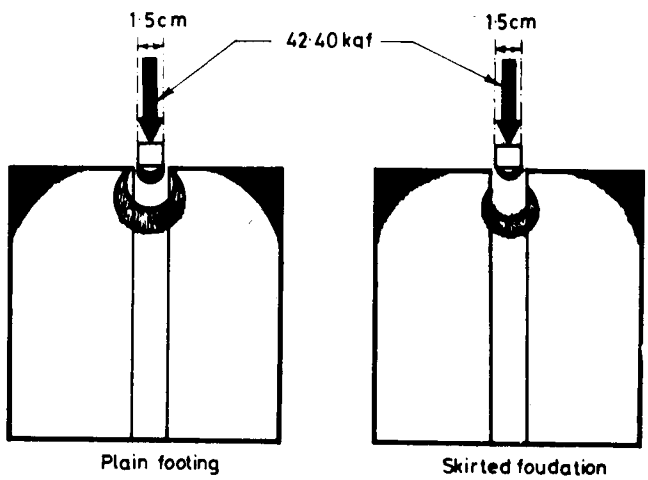


Fig.12 ISOCHROMATICS OF PHOTO ELASTIC METHOD

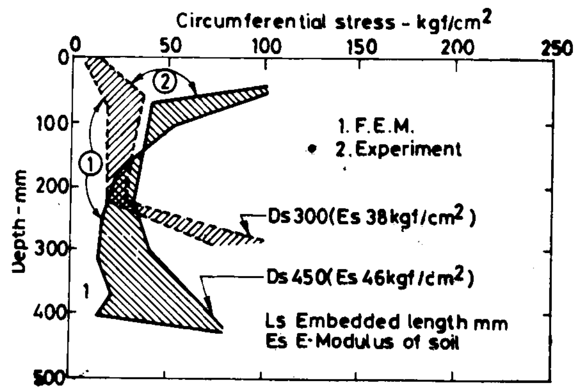


Fig.13 CIRCUMFERENTIAL STRESS ON SKIRT

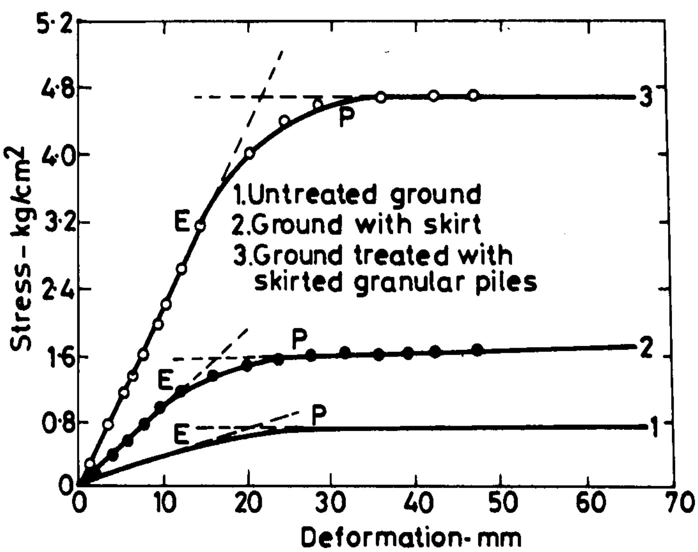


Fig.14 STRESS DEFORMATION BEHAVIOUR OF GROUND TREATED WITH GRANULAR PILES
(Rao 1982, Rao & Ranjan 1985)

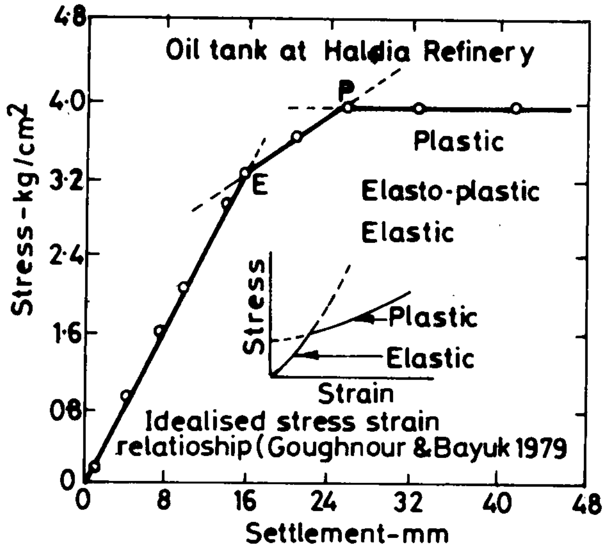


Fig.15 STRESS DEFORMATION BEHAVIOUR OF COMPOSITE GROUND(SOFT CLAY DEPOSIT TREATED WITH GRANULAR PILES)

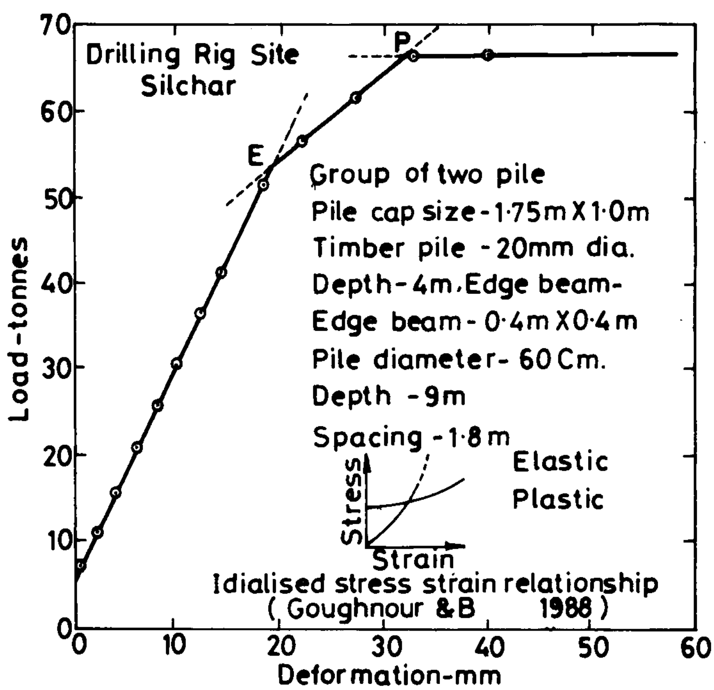


Fig.16 LOAD DEFORMATION BEHAVIOUR OF TWO GRANULAR PILE GROUP WITH TIMBER PILE SKIRTING (Rao & Ranjan 1988)

of the cohesionless soil reinforced with both plain/skirted granular piles, whether single or in groups, mobilizes at a deformation equal to 10 per cent of the pile diameter. The elastic yield stress is found to occur at a deformation equal to 5 per cent of the pile diameter. These deformation limits for plastic and elastic yield points are found to be 5 per cent and 2.5 per cent for the saturated clayey silt and soft clay deposits tested (Rao, 1980-1982, Rao and Ranjan, 1985).

IMPROVEMENT OR SETTLEMENT RATIO

Within the elastic limit the stress concentration ration n and the effective modular ratio m are taken as equal as illustrated by Rao and Ranjan (1985), and (1988). These have been presented below Eq.15.

$$q_p/q_s = E_p/E_s \qquad ...(15)$$

and $q_p/q_s = n = E_p/E_s = m$ ..(16)

The applied load, q is shared by the pile, q_p and the ambient weak soil, q_s and given by Eqs. 17 and 18.

$$q_p = q \frac{E_p}{\alpha E_p + (1-\alpha)E_s} \qquad ..(17)$$

$$q_s = q \frac{E_s}{\alpha E_p + (1-\alpha)E_s} \qquad ..(18)$$

In which the expression $[\alpha E_p + (1-\alpha)E_s]$ is regarded as equivalent modulus E_{eq} and the inverse of it is taken as equivalent coefficient of volume compressibility $(m_v)_{eq}$ and is expressed by Eq. 19.

$$(m_v)_{eq} = [\alpha E_p + (1-\alpha)E_s] \qquad ...(19)$$

Therefore according to Rao & Ranjan (1988) the settlement reduction ratio β is given by Eq.20.

$$\beta = \frac{\Delta L}{\Delta L'} = \frac{(m_v)_{eqi}}{(m_v)_i} \qquad ...(20)$$

Eq. 20 can be rewritten in the form of Eq.21.

$$\beta = E_s/[\alpha E_p + (1-\alpha)E_s] \qquad ...(21)$$

$$\beta = 1/[1 + (m-1)\alpha \qquad ...(22)$$

The settlement ratio according to Meyerhof (1984) is given by Eq. 23:

$$1/\beta = 1/[1 + (m-1)\alpha \qquad ...(23)$$

Also the settlement improvement ratio R proposed by Priebe (1976) and discussed by Greenwood and Kirsch (1984) is expressed by Eq.24

$$R = [1 + (E_p/E_s - 1)A_p/A] \qquad ...(24)$$

and the equivalent coefficient of volume compressibility m_{veq} is expressed by Eq. 19. (Rao, 1982, Schlosser and Juran, 1979) which is the same as proposed by Priebe (1976) hence

$$(m_v)_{eqi} = 1 + (m-1)\alpha \qquad ...(25)$$

The equivalent coefficient of volume compressibility is found to depend on effective modular ratio (m) and replacement factor α, (Rao and Ranjan, 1988).

STRESS CONCENTRATION RATIO

Among the most important parameters in the desing of granular piles are stress concentration ratio n= q_p/q_s which is a fundamental parameter which depends on several factors including replacement factor α (Schlosser et al 1983) and effective modular ratio m = E_p/E_s. Further based on the assumption of uniform settlement Aboshi et al (1979) have related these factors with settlement reduction ration β and given by Eq. 26.

$$\beta = \left[\frac{1}{1 + (n - 1)\alpha}\right] \quad .....(26)$$

Rao and Ranjan (1988) have replaced stress concentration ratio n with the stiffness factor or effective modular ratio, m, since accurate determination of n is not easy and does not have unquique value. Also under desing load, n is found to have diminition trend (Vautrain, 1977). It is therefore rational to use stiffness factor m in Eq.25 which can easily be assessed. It has further been indicated that the replacement factor, (α) is the prime determinant of the shape of the load/settlement curve whilst stiffness ratio, m control magnitude of the settlement (Greenwood and Kirsch, 1984). The stress concentration ratio, n appear to reflect relative stiffness ratio, m of the granular pile and the ambient clay. Thus the validity of assumption by Rao & Ranjan (1985) that within the elastic range n and m are equal, is justified. This is further substantiated by full scale tests in different soil conditions.

The settlement reduction ratio β(Aboshi, 1979, Schlosser, et al 1983; Rao & Ranjan, 1988) and inverse of β..i.e. settlement ratio (Meyerhof, 1984) or settlement improvement ratio R, (Priebe, 1976, Greenwood & Kirsch 1984) or equivalent coefficent of volume compressibility ($m_{v\ eq}$) (Rao & Ranjan, 1988) are totally dependent on effective modular ratio m and replacement factor α.

The statement is further substantiated by an interesting and unique relationship between settlement ratio and area ratio proposed by Meyerhof (1984) for varying stiffness ratios (m=5-200). The chart includes different ground improvement techniques such as granular piles,lime piles and driven reinforced concrete piles (Fig.17). Recent data for granular piles, minigrouted piles and self setting soil slurry piles from India, have also been superimposed over it. The study of Fig. 17 clearly demonstrated superiority of the chart for design predictions.

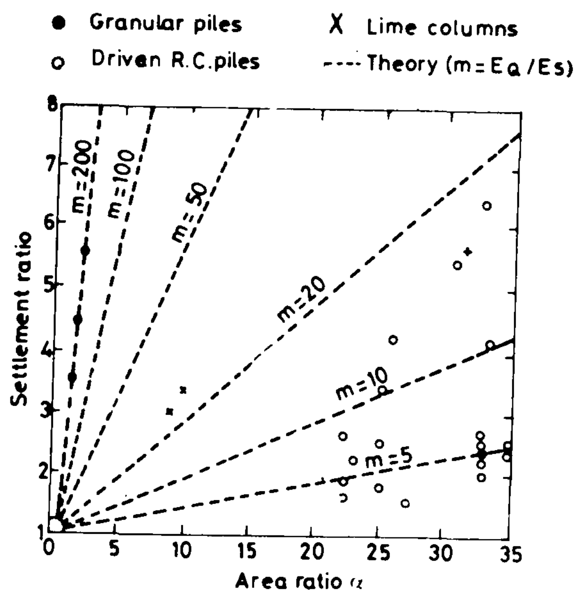


Fig.17 RELATIONSHIP BETWEEN SETTLEMENT RATIO AREA RATIO & EFFECTIVE MODULAR RATIO (Meyerhof 1984)

MOTIVATION AND COMMENTS

In the context of present technological growth and efforts towards search and research have resulted into varieties of ground treatment methods during past 2-3 decades, covered in detail during 11th IGS Lecture (Ranjan, 1988) and also response of ground treated with granular piles was presented.

As it is evident from the theme of the lecture that the scope is vast in the sense that it involves varieties of structures and methods of treatments besides different sub-soil conditions including filled up and waste lands. Therefore, any attempt to treat such a wide scope of the theme in a single paper will not be fair and justified. Further, it may also be agreed that the main purpose of research or search is to disseminate the proven results of research for the use of a common man. Of course, by doing so, the importance of publishing research papers in high quality journal or in the books can not be de valued. However, the statement is more true particularly for developing countries like ours since we can not afford to be satisfied and feel happy about it by simply decorating our shelves and almirah, with these journals or books. Therefore, our efforts should be diverted towards engineering applicaton of these techniques developed through research as an alternatives to sophisticated methods and machines by new foundation techniques which are efficient, speedy and cost effective besides generating employment opportunities. Such decisions should be based primarily on technical soundness of the foundation technique weighted in terms of short and long term stability and trouble free performance in service life.

On the other hand, it is always easy to deal with new constructions where several possibilities exist and the designer enjoys considerable freedom to choose the most ideal, cost effective solution based on value judgement and technoeconomic considerations. The major problem, however, arises when one has to attempt to deal with problems of distressed structure so as not only to improve their life but also to build taller.

It is time, therefore, that due recognizance is given to field application of new technologies as an outcome of research, and its application to live problems and fully utilize in solving foundation problems associated with distressed structures.

In view of the preceeding discussions the researches made by the author during past decade and a half did not only aim at restricting itself in developing the concept for the new technologies, predict their performance thgrough analytical modelling and developing design procedure and feel satisfied after publishing papers, but concentrated attempts have been made to verify the validity of design assumptions through insitu load tests on full size prototype foundations in the field besides perfecting the construction methodologies by actual application to live problems and finally monitoring field performance through feed back studies. This naturally enriched both, the author and the user with the added confidence.

While introducing the failure mechanism under a foundation overlaid on weak sub soil and linking it with the various types of reinforcing methods developing the desing methodology, verifying through full scale insitu tests and monitoring performance of new structures, attempt has been made to introduce few selected typical case studies with a view to create more awareness and confidence among practising geotechnical engineers. it is to this persuit that this lecture is dedicated.

CASE RECORDS

INTRODUCTION

Case studies on performance of new foundation techniques under live structures are very few. Also reported, records of failures are almost found to be nil. The tendency of not publishing such records may be attributed to the lack of confidence among designers and practising engineers due to obvious reasons known to them.

Each case record is a separate identity in itself and each problem has got to be thought and dealt individually, since these are of different types and having altogether different basic philosophies. For example there are situations where

(a) topographical conditions demand raising of natural soil level in creek land.

(b) reinforcing of weak sub soil deposits to a large depth, demand and to support high design loads with almost minimal settlement.

(c) disress due to lowering of water table and unequal movement of foundation exceeding limits of tolerance.

(d) nature of very weak soil deposit demanding special design and construction techniques.

(e) behaviour under static and dynamic loading.

(f) type of structures, inadequate design, use of faulty materials in construction, and finally.

(g) lack of quality assurance.

Therefore, each problem has got to be thought separately and individually since a single solution can not be applicable for all the cases.

In view of the above, few selected case records of analysis, design, construction, performance of foundations under live structures have been identified by the author, out of many successfully used. The various case studies covering varieties of structures such as low and high rise framed buildings, oil drilling rig, under ground power house besides small to very large furanace and crude oil storage steel tanks, remedial strengthening of distressed tank foundation and under pinning of the foundation of a shopping-residential complex have been selected and discussed in the later part of the paper. These cover vide range of difficult sub soil conditions such as loose to medium dense cohesionless deposits with high water table, clayey silt with low plasticity to soft marine clay deposits to large depth with high compressibilities.

Therefore, the case records of performance of the following foundation techniques have been picked up for presentation in this paper.

(a) Pad foundations on soft saturated marine clays.

(b) Large and/rigid/RCC raft overlaid on reinforced weak clay deposit under high intensity of stress, 23m below natural soil level.

(c) alround timber pile skirting with ring beam in soft clayey silt deposits.

(d) behaviour of column footings supported by granular piles in soft clayey silt deposit.

(e) large storage crude oil tank suported on flexible raft overlying deep layers of soft saturated clays treated with 4000 granular piles in creek land.

(f) remedial underpinning of distressed steel tank foundation on cohesionless deposit.

(g) underpinning of foundations of distressed shopping-residential comnlev

MURUM PAD UNDERLAIN BY SOFT MARINE CLAY DEPOSIT

INTRODUCTION

The assignment related to "Assessing the suitability of open RCC footing for RCC columns and framed structure on soft marine clay deposit where topographical conditions called for large earth filling". The problem consisted of construction of RCC open foundations for ground plus three storied buildings on 0.6m thick compacted murum platform laid directly over soft marine clay strata with estimated safe bearing capacity of 6.6 t/m². Competent strata for resting of the bearing piles was not available upto large depth of 20m - 25m.

An interesting facet of the problem was that whether the murum pad compacted under controlled condition by 10 tonne rollers laid on marine clay deposit could possibly be used safely for the structural design with increased bearing capacity of say 18 t/m². Or the murum pad could only assist in spreading the loads from independent column footings of the RCC frame?

Based on the above structural system, planning design and construction of buildings of ground plus three upper floors at a location in Bombay sub-urban area adjoining to the low lying creek lands, requiring about one-meter earth filling had been completed.

CBR test check revealed CBR values, between 4 to 6 (Table 1) and average bulk unit weight of 1.9 t/m² (Table 2).

WORK PLAN

The work plan consisted of (1) Review of available data (2) Study of the strength deformation characteristics of the marine clay samples from the site, (3) Predicting long term total and differential settlement based on current state-of-the-art and (4) suggesting cost effective remedial measures if necessary.

LOCATION

This entire site of housing scheme is situated near a Creek. It is bounded in the west by a railway embankment and on the east, open plot and surrounded by Navghar Village in the south with some builtup structures. The north side is surrounded by the Creek. Local enquiry revealed that the whole site used to be flooded with tidal water, Figs. 18 & 19.

DEVELOPMENT OF THE PLOTS & QUALITY CONTROL

As a first step an earthen embankment was built in the middle of June, 1986 all along the northern side and on the eastern side to arrest the tidal water inflow. The height of embankment was kept 1.2 m above the highest tidal level (HTL). There was no tidal water inflow subsequently. The creek land was required to be raisedby about a meter by earth filling. This was achieved by murum filling in layers of 23 cm, spreading, levelling

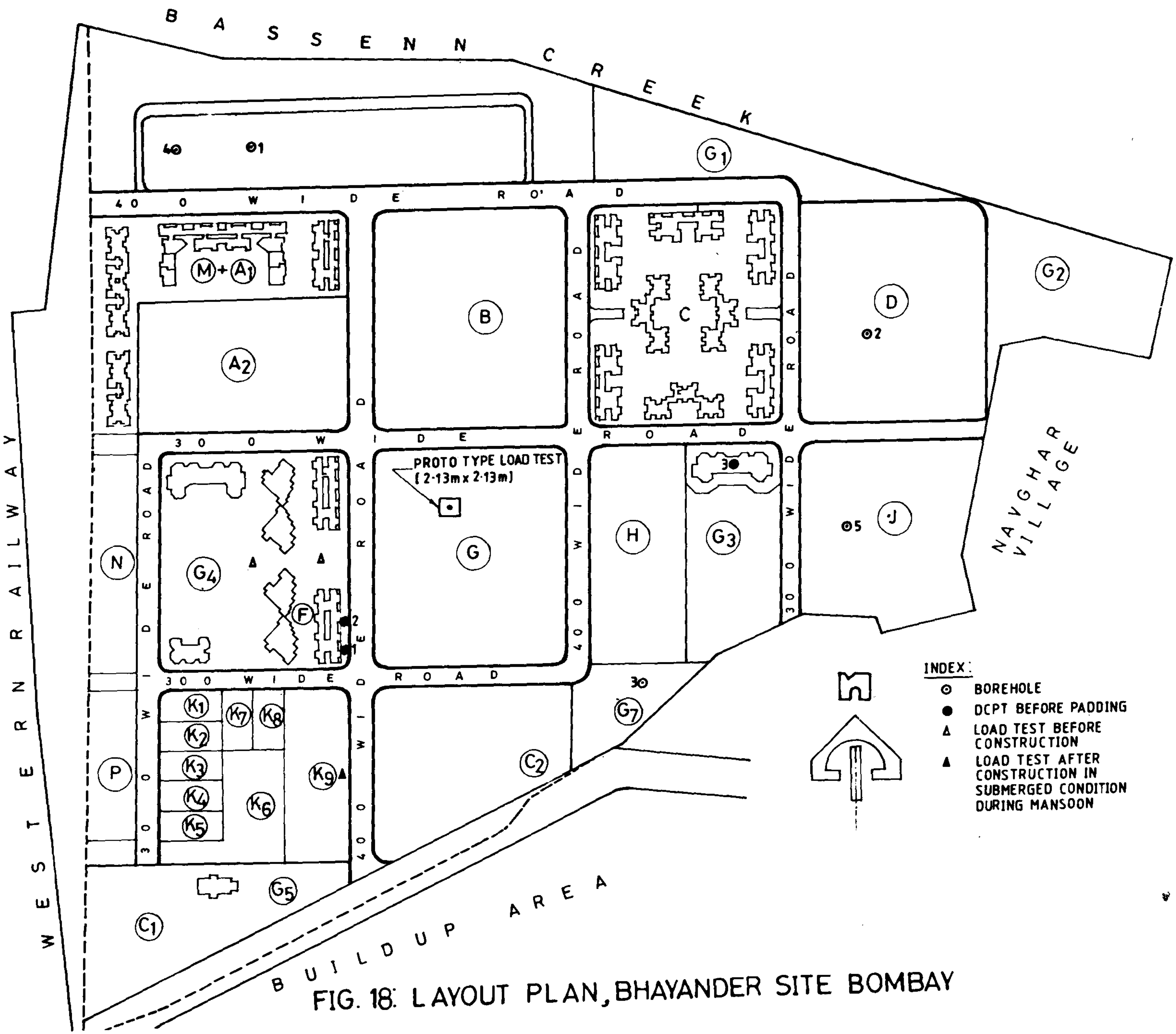


FIG. 18: LAYOUT PLAN, BHAYANDER SITE BOMBAY

TABLE 1

FIELD CHECK FOR CBR VALUE

Locations	CBR Value				
	1	2	3	4	5
1 Layer	4.16	5.44	5.6	4.8	5.44
	(4.0)	(5.0)	(6)	(5)	(5)
II Layer	6.08	4.96	4.8	5.9	5.76
	(6.0)	(5)	(5)	(6)	(6)
III Layer	4.80	5.44	5.28	5.60	5.60
	(5)	(5)	(6)	(6)	(6)
IV Layer	5.44	5.44	6.08	5.76	6.24
	(6)	(5)	(6)	(6)	(6.0)

Values in the bracket are CBR in soaked condition

TABLE 2

FIELD CONTROL FOR DENSITY AND WATER CONTENT

Tests	Field density (t/m^3)				
Sample No.	1	2	3	4	5
III Layer	1.96	1.91	2.00	1.99	2.02
	(5.0)	(4.0)	(6.0)	(6.0)	(6.0)
IV Layer	2.05	2.01	2.09	2.03	-
	(6.0)	(6.0)	(7.0)	(6.0)	-

Values in the bracket shown corresponding water content

FIG 19. FLOODING OF THE SITE WITH TIDAL WATER

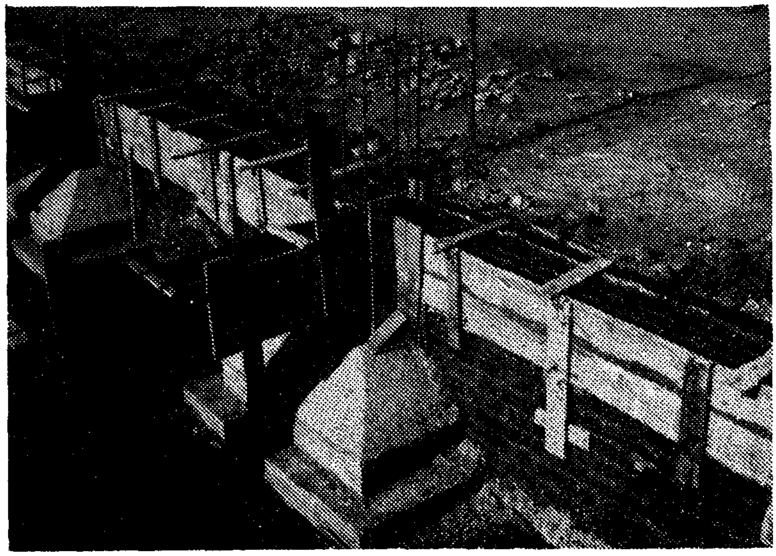


FIG 20. CASTING OF THE PLINTH BEAM

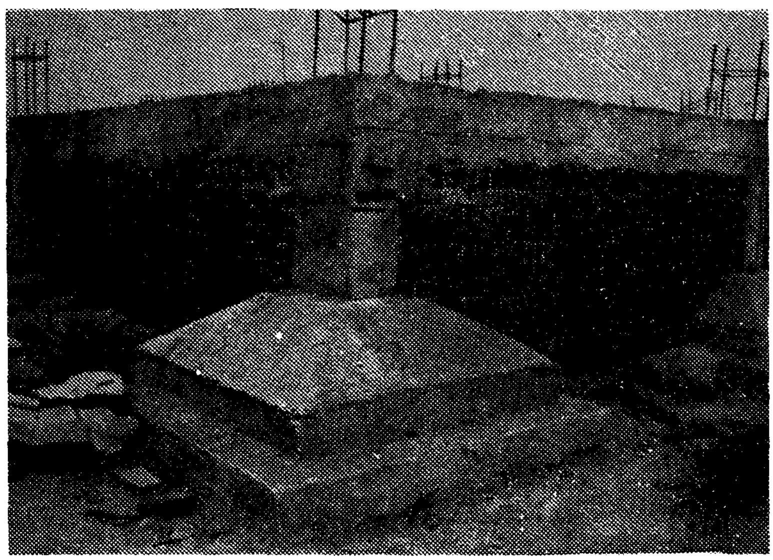


FIG 21. PLINTH BEAM AFTER REMOVAL OF SIDE SHUTTERING

FIG 22. EARTH FILLING UPTO THE PLINTH LEVEL

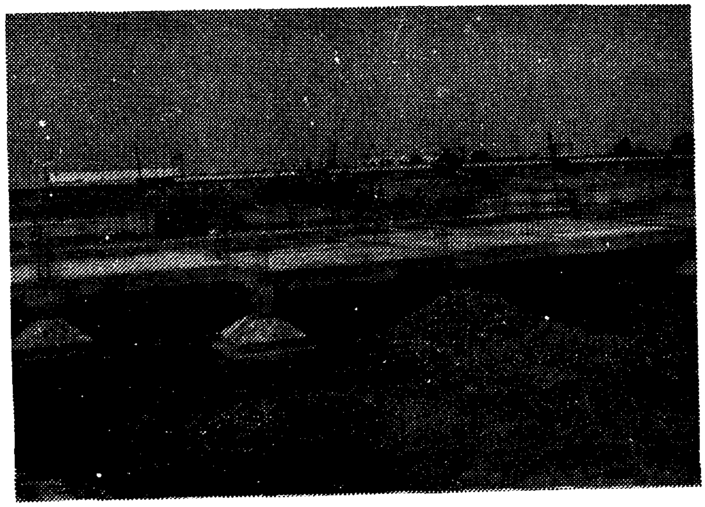
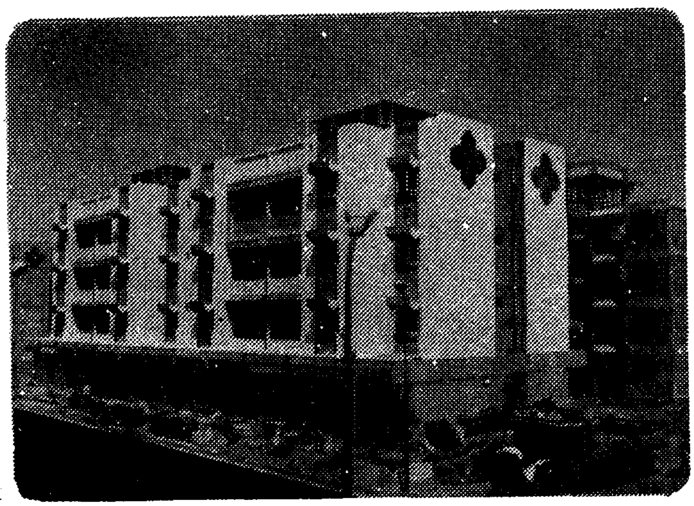


FIG. 23 Casting of the ground floor



FG 24. A COMPLETED BLOCK

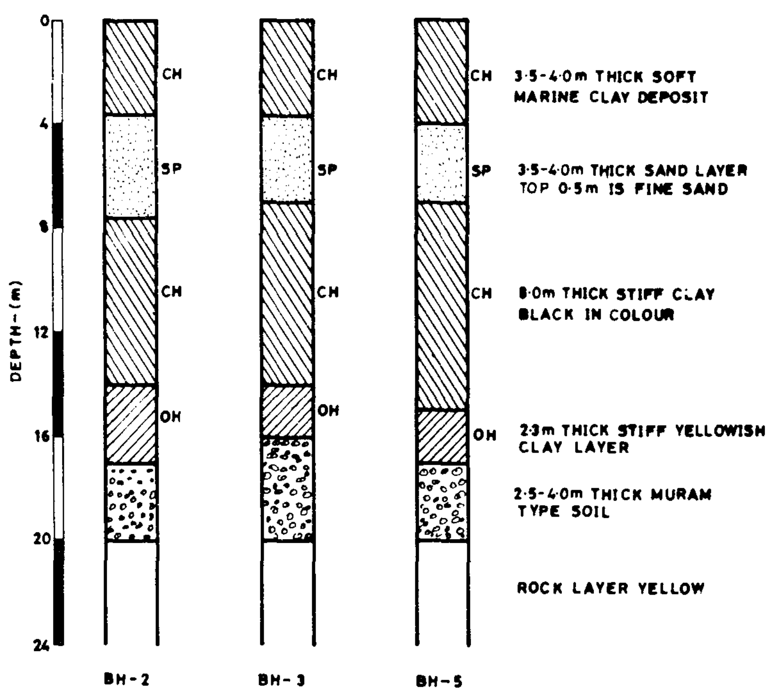


FIG. 25 SUB-SOIL STRATIFICATION BHAYANDAR SITE

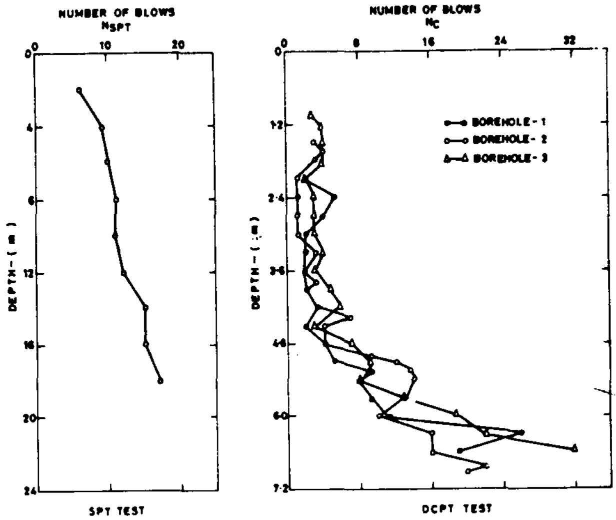


FIG.26 : SPT & DCPT TEST DATA

and compacting each layer to 15 cm by using 10 tonne rollers. Such four layers were laid, making the total thickness of compacted murum pad equal to 0.6 m. The quality control for the compaction of the murum pad was ascertained with the help of CBR test at five selected locations to ensure that the design value of CBR equal to 4 was satisfied. The optimum dry unit weight of the murum wat 1.6 t/m^3 at a water content of 12-15 per cent.

CASTING OF COLUMNNS FOOTING AND PLINTH BEAM

RCC Footings for the different design loads were cast directly on the murum pad (Fig. 20.). This was followed by casting of the plinth beam (Figs. 20 & 21) and filling of earth upto plinth level (Fig. 22).

The ground floor was then cast (Fig. 23). Fully completed blocks of flats have been shown in Fig. 24.

SUB-SOIL CHARACTERISTICS

Based on the study of B.H.2, 3 and 5 (Fig. 25) and the fleld and laboratory identification tests it is observed that the general profile of the subsoil strata consists of 3.5 to 4 m thick soft marine clay deposit (CH) having N_s equal to 6-8 number blows, underlain by a 3.5-4 m thick medium dense sand layer (SP)with N_s average equal to 10 number of blows upto a depth of 7 to 7.5 m. About 0.5 m thick layer of fine sand was found at the top of this layer. Further extension of the bore hole upto 15 m indicated that the standard penetration value Ns, from 11 number of blows at 8 m depth increases to 12 number of blows at 12 m and then further increasing to 15 number of blows at 15 m depth. This 8m thick black colour stiff clay layer is classified as CH as per IS: 1498-1970. This is followed by a 2-3 m thick layer of yellowish stiff clay layer having N_s equal to 15 number of blows per 30 cm penetration Further extension of the bore hole revealed a 2.5-4 m thick murum layer with N_s equal 17 number blows and beyond 20 m of depth is found yellowish rock layer was encountered.

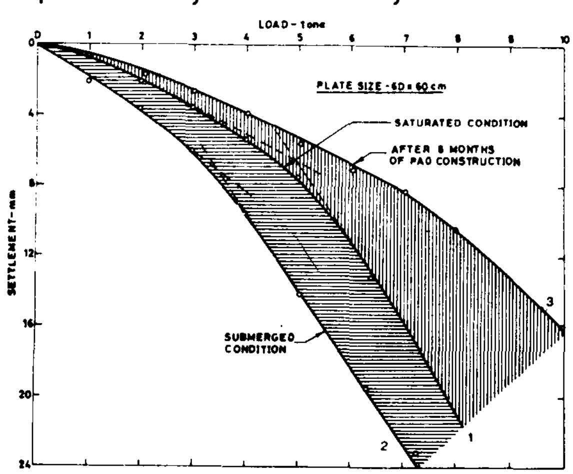


FIG.27 LOAD SETTLEMENT CURVES ON WEAK CLAY LAYER

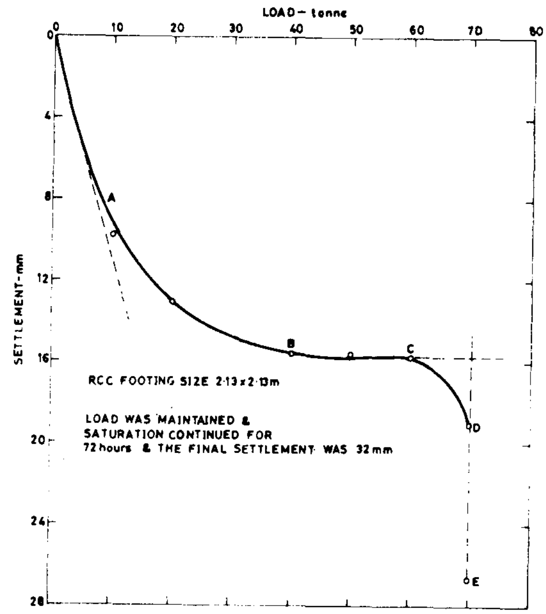


FIG 28 LOAD SETTLEMENT CURVE 213x213m CONCRETE FOOTING

Besides standard penetration tests, dynamic cone penetration tests were also carried out. The test results at three locations have been shown in Fig. 26. These test result confirm that the strata from virgin ground level and upto 4 m depth as saturated clay deposit followed by stiff layer.

INSITU LOAD TESTS

Two plate load tests on 60 cm x 60 cm plate 25 mm thick were carried out under natural and submerged conditions respectively (Fig. 27). Study of the Curves (Fig. 27) indicate that no failure load are observed from curves (1) and (2). However, considering curve (2) as the worst case under submergence, and using double tangent method, the slope of the load settlement curve changes at an intensity of load equal to 9.7 t/m² and the safe bearing capacity (F.S=2.5) is found to be as 3. t/m². On the other hand the the third load test was carried out after eight months after the laying of the murum pad under fully submerged conditions (Curve 3).

Following the same procedure, the safe bearing capacity (F.S=2.5) is found to be 6 t/m². This is in accordance with IS : 6403-1971.

(a) Safe bearing capcity before casting the murum pad

$q_{safe} = 2.5 + \gamma Df = 3.5 + 1.9 \times 1.2 = 3.5 + 2.28$

$q_{safe} = 5.78$ t/m²

(b) Safe bearing capcity after casting the murum pad

$q_{safe} = 6.0 + 2.28 = 8.28$ t/m²

Thus the increase in the safe bearing capacity of the sub soil due to casting of murum pad itself is found to be about 45 percent over the original bearing capacity.

INSITU LOAD TEST ON FULL SCALE FOOTING

A concrete footing (2.13 m x 2.13 m) was cast on the murum pad, 15 cm thick, and was loaded to 70 tonnes in increments of 10 tonnes each. Next increment was applied only after 24 hours. Thus seven increments were applied in 7 days and after reaching a load equal to 79 t, an artificial saturation was done by flooding the area with water. This was continued for 3 more days and under the load of 70 tonnes, a total settlement of 32 mm was obtained. The load/settlement relationship is shown in Fig. 28.

Corresponding to a design load of 18 t/m², the total load on a 60 cm x 60 cm plate, will be equal to 6.48 tonnes. Thus the corresponding settlement for this load from Fig 27 is found to be equal to 19.5 mm.

If S_p = 19.5 mm, B_p = 0.60 m S_f = ? B_f = 2.13 m

In accordance with IS : 1888-1981 for clayey soil

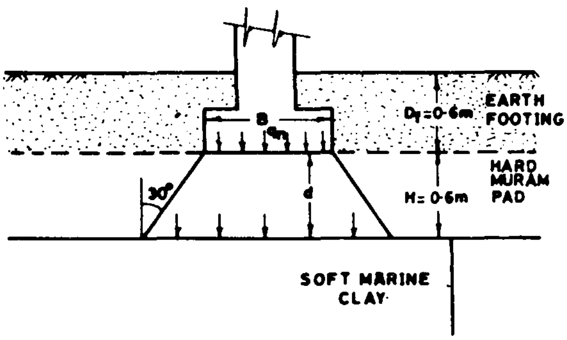


FIG. 29 FOUNDATION ON HARD MURUM PAD UNDER LAIN BY SOFT MARINE CLAY (AFTER TOMLINSON 1986)

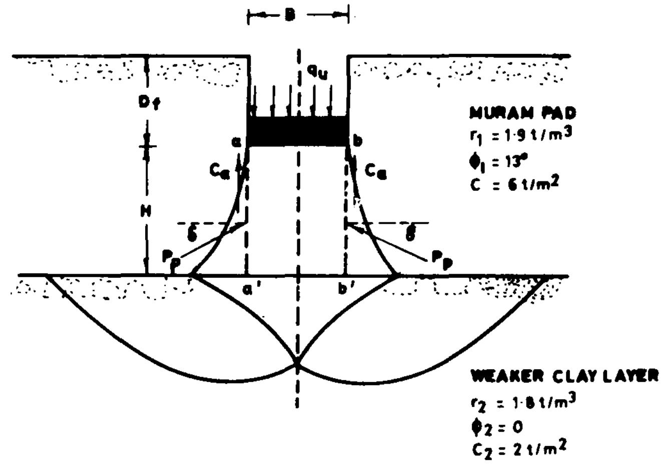


FIG.30 SHALLOW ROUGH COLUMN FOUNDATION ON LAYERED SOIL

$S_f/S_p = B_f/B_p$ ...(27)

or S_f = 19.5 (2.13/0.6) = 69.2 mm.

Say 70 mm

Thus it is seen that the settement of a (2. 13 m x 2.13 m) full size footing under a design intensity of 18 t/m², is likely to be of the order of 70 mm during the insitu load test. Hence for a (1.8 m x 1.8 m) RCC column footing the corresponding settlement will work out to be 50 mm only.

SIGNIFICANCE OF THE LOAD SETTLEMENT CURVE

Further study of the load settlement cruve (Fig. 28) indicates tha when the first increment of 10 tonne was placed the full load was transferred to the compressible weak soft clay layer through the hard murum pad as indicated by the portion OA. On further increase in the load, the applied load was gradually transferred to the medium dense sand layer followed by the stiff clay layer. Portion AB of the cruve is the transition period and beyond the point B, the stiff clay layer fully participated in sharing the applie load, thus indicating practically no settlement from B to C (40 to 60 tonne load). After reaching a load of 70 tonnes, the footing base (murum pad) was flooded which caused an additional settlement of 6.8 mm making the total settlement of 32 mm. Of course, this total settlement does not carry any significance on the over all behaviour of the footing.

INTENSITY OF DESIGN LOAD ON MARINE CLAY SURFACE

(1) *Plain Concrete Footing* (2.13 m x 2.13 mm)

Intersity of load on Murum pad due to concrete footing (2.13 m x 2.13m) = 70/(2.13 x 2.13) = 15.43 t/m²

During the load test, since ultimate bearing capacity failure was not observed intensity of stress q_1 on the marine clay surface may be found from Tomlinson (1974) Fig. 29).

$$q_1 = q_n \left(\frac{B}{B + d}\right)^2$$

where, d =0.6 m, B = 2. 13 m, q_n = 15.43 t/m²

q_1 = 15.43 [2.13/(2.13 + 0.6)]² = 9.41 t/m²

Thus, an intensity of stress equal to 9.41 t/m² will be experienced on the soft clay surface layer.

(2) RCC Column Footing (1.8 m x 1.8 m)

Simailarly the intensity of desing stress on the ma ne clay due to a live and dead load of 54 tonnes, on a 1.8 m x 1.8 m footing was found to be as 9.37 t/m2 and under the RCC column 1.15 m x 1.15 m for a load of 22 tonnes was obtained as 7.1 t/m^2 also for 1.8 m x 1.8 m footing under 40 tonnes load it was found to be 6.9 t/m^2.

Hence, the soft clay layer surface will be subjected to a design load of 7 t/- 9.4 t/m^2 on the virgin marine clay layer through the column footing varying in size.

MURUM PAD AS A FLEXIBLE RAFT

Considering murum pad as a flexible raft (40 m x 14 m x 0.6 m) transferring a total load of 2500 tonnes through 77 columns, to the surface of the raft giving an intensity of 4.46 t/m^2. Hence, on the virgin soil having soft marine clay the intensity of load along the width and length of the raft q_B (14 m) and q_l (40 m) were found as 4.1 t/m^2 and 4.32 t/m^2 respectively.

Therefore, the variation in the desing intensity of stress from 4.1 to 4.32 t/m^2 along the width and the length of the raft may be experienced respectively. This value will increase to a minimum of 7 t/m^2 and a maximum of 9.3 t/m^2 under the colum footings, sustaining 54 tonnes and 40 tonnes repectively.

The reason for arriving at such a conclusion is that in the above computation, classical theory of bearing capcity has been resorted to, which is applicable to a sub-soil stratum which is homogeneous isotropic and semi infinite soil mass. In the present case the subsoil stratum consists of two layer system, where a hard strata of compacted murum pad is underlain by a weak soft clay deposit of limited depth (3.5 m -4m) which is marine clay (Fig. 29). As such, the very attempt to base the bearing capacity computations on the classical bearing capacity theory, which does not recognise the advantage of hard murum stratum over-lying the soft marine clay is not justified. It may further be noted here that in the present case, since the depth of the murum pad, (H) below the footing is 0.6 m only, which is one third of the width of the footing B (i.e. 1.8 m). Thus in such cases where H << than B, punching shear failure will occur in the murum pad before the general shear in the weaker clay layer (Fig. 30).

The available ultimate bearing capacity q_u of the combined two layer system (which is hard murum pad underlain by a soft marine clay layer) will consist of (a) ultimate bearing of capacity the weak clay layer q_b, (b) increase in capacity due to adhesive force C_u in thepad material, (c) increase in capacity due to vertical component of the passive force ($P_p \sin \delta$) and (d) the surcharge γD_f (Fig. 30), and finally (e) the contribution of Rest Time for the weak clay layer which has been under almost full design load for a period of more than 12 months.

BEARING CAPACITY OF VIRGIN MARINE CLAY

The ultimate bearing capacity q_u of the virgin soil is given by Eq. 29.

$$q_u = c.\ N_c\ (1+0.2\ B/L) \qquad \text{...(29)}$$

Input data

Cohesion c = 2 t/m^2, N_c = 5.14 for $\phi = 0$, γ_{bulk} = 1.9 t/m^2 fot murum pad.

For a square footing, B/L = 1, shape factor Sc = 1.2 and

Surcharge γ Df = 2.28 t/m^2.

Therefore, q_{safe} = 4.9 + 2.28 = 7.18 t/m^2

with a factor of safety equal to 2.5.

GENERAL OBSERVATIONS/COMMENTS

Study of the foregoing para suggests that the available safe bearing capacity of the weak layer of marine clay deposit is 7.18 t/m^2. However, the respective design stresses due to dead and live loads for (a) the smallest columns under 22 tonnes of load is 7.1 t/m^2, (b) the largest columns under 54 tonnes of load is found to be as 9.37 t/m^2 and (c) for 40 tonnes - columns the design stress in only 6.9 t/m^2. These clearly demonstrate that the design stress for the largest column (9.37 t/m^2 exceeds the safe bearing capacity of 7.18 t/m^2 leading to a marginal factor of safety.

Therefore, at the face, it may be concluded that the above observation is at variance with those of two reports submitted to the author.

TABLE 3

LABORATORY TEST RESULTS-DEPTH 1.85 m

SI. No.	W/C %	L L %	P L %	P I %	Cu (KPa)	Q3 (KPa)
1.	42	65	32	33	18.75	325
2.	42	65	32	33	17.5	425
3.	42	65	32	33	22.75	425

TABLE 4

LABORATORY TEST RESULTS-DEPTH 3.15 m

SI. No.	W/C %	L L %	P L %	P I %	Cu (KPa)	Q3 (KPa)
·1.	57	73	34	39	22.75	425
2.	57	73	24	49	-	-
3.	57	73	24	49	-	-

value of cohesion was found to be 0.6 Kg/cm² and angle of internal friction ϕ 13°

LABORATORY INVESTIGATIONS

The Atterberg's Limits, undrained shear strength C_u, and moisture content have been shown in Table 3 & 4.

The undrained shear strength for samples from 1.85 m depth was found equal to 0.2 Kg/cm2 and from 3.15 m depth the value was 0.225 Kg/cm².

Consolidated undrained triaxial tests were also carried out on the undisturded samples collected from the compacted pad material. The value of cohesion was found to be 0.6 kg/cm² and angle of internal friction ϕ 13°

TEST RESULTS AND DISCUSSION

The undrained shear strength of soft marine clay was found to be 0.2 Kg/cm² and for the pad material 0.6 Kg/cm² with the value of angle of internal friction 13°. The preconsoilidation pressure of the soft marine clay deposit (Table 5) varies from 1.25 Kg/cm² at 1.85 m and at 3.15 m depth it is found to be 1.70 Kg/cm². The compression index were found to be 0.46 and 0.299 respectively.

The maximum and minimum values of the design load varied between 22.68 tonnes to 40.82 tonnes on (1.14 x 1.14 m) and 1.22m x 1.83 m in footing size repectively. It would thus be seen that a design load equal to 18 t/m² at the interface of the concrete footing resting on murum pad may be taken for further computations. Incidentaly this pressure is almost equal to the preconsolidation pressure (1.7 Kg/cm²). Im such a situation if the design load is equal to or less than the preconsolidation pressure then the settlement should essentially be elastic and corresponding differential settlement is not likely to be large (Thonburn, 1987) and the structures would try to adjust such additional stresses through soil structures interaction.

THEORETICAL CONSIDERATIONS

BEARING CAPACITY OF TWO LAYER SYSTEM

The analysis for the ultimate bearing capacity of shallow rough continuous foundations supported by a

TABLE 5
CONSOLIDATION CHARACTERISTICS OF SOFT MARINE CLAY DEPOSIT

Depth (m)	Compression Index C_c	Preconsolidation Pressure (Kg/cm²)
0.5	0.265	1.7
1.85	0.46	1.25
3.15	0.299	1.7

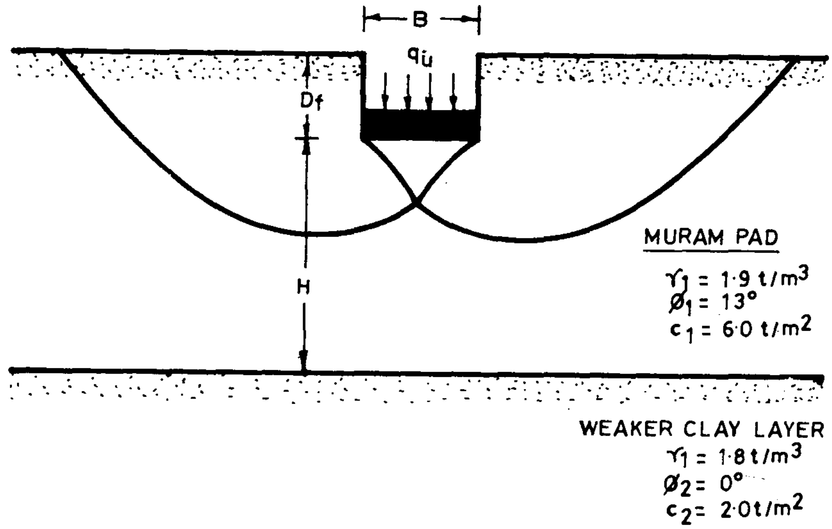


Fig.31 CONTUNIOUS ROUGH FOUNDATION LAYERED SOIL H/B IS LARGE

strong murum pad underlain by a weaker layer may be developed by assuming a two layer system and the failure mode as shown in Fig.30. The failure surfaces are developed by considering the failure as an inverted uplift problem (Meyerhof and Adam, 1968). Thus at ultimate load, the murum pad, having an approximately truncated pyramid shape is pushed into the clay so that in the case of general shear failure, the adhesion Ca and friction angle ϕ in the murum pad and undrained cohesion C_u in weak clay are mobilised in the combined failure zones. It may be noted that if the thickness H, is relatively small compared to the foundtion width B, a punching shear failure will occur in the murum pad (Fig. 30) followed by a general shear in the weak clay layer. When the thckness H, is large compared to B the failure pattern will be as shown in Fig. 31 Thus in the present case the ultimate bearing capacity for square footing can be computed by Eq. 30, when B/L=1.

$$q_u = q_b + 2\left(\frac{C_a H}{B}\right) S_a + 2_{1\gamma 1} H^2 \left(1 + \frac{2Df}{H}\right) \frac{K_s \tan \phi 1}{B} S_s + \gamma_1 H \qquad ...(30)$$

The punching shear coefficient K_s can be determined by using the passive pressure coefficient chart (Caquot and Kerisel, 1949) which relates K_s with) ϕ_1 for values of q_2/q_1 varying from zero to one where q_1 and q2 are the ultimate bearing capacities of a continuous footing of width B laid on surface of homogeneous murum pad and bottom weak clay deposit.

Input data :

Hard Murum

$C_1 = 6t/m^2$, $\phi_1 = 13°$ $\gamma_1 = 1.9$ t/m²

shape factors for $\phi \geq 10°$ $\therefore$ $S_{c(1)} = 1.89$, $S_{\gamma 1} = 0.8$

and $N_{c(1)} = 10$, $N_{\gamma 1} = 1$

$\therefore$ Substituting above values q1= 59.28 t/m2 and for weak clay layer

$\phi_2 = 0$, $N_C = 5.14$, N = 0, $S_{c(2)} = 1.2$, $C_2 = 2t/m^2$

Thus $q_2 = 12.33$ t/m²

$$\therefore \text{ The Ratio } q_2 / q_1 = \frac{12.33}{59.65} = 0.2$$

Thus from Caquot and Kerisel chart for ϕ_1=13° and $q_2/q_1 = 0.2$

The value of S_S=1 for strip footing and for a square and circular footing S_s= 1.1 ~ 1.27.

Therefore, the ultimate bearing capacity q_u of two layer sytem is given by Eq. 30, as presented below.

Here S_a= 1,S_S =1.2,C_a = 5.4 K_s =1.5

$$q_u = 14.61 + 2\left(\frac{2 \times 5.4 \times H}{1.8}\right)(1) + (2 \times 1.9) H^2 \left(1 + \frac{2 \times 0.6}{H}\right)$$

$$\times \frac{1.5 \times 0.23}{1.8} + 1.9H$$

$\therefore q_u = 23.89\ t/m^2$ since H = 0.6 m in this case

hence available F. S. $= \dfrac{23.89}{12.33} = 2$

Further it would be of interest to know that for the weaker clay layer not to have any adverse effect on the ultimate bearing capacity of the combined two layer system, the minimum value of the pad thickness will be such that it will satisfy the following relationship.

$$q_u = 14.61 + 12H + 3.8\,H\,(H + 1.2)\,(0.23) + 1.9H = 122.33$$

$$\therefore\ 0.87\,H^2 + 14.94H - 14.61 = 0$$

or H = 5.46 m

and $H / B = \dfrac{5.46}{1.8} = 3$

SIGNIFICANCE OF REST-TIME

Because of the excess pore water pressure that remains at the centre of clay layer (Fig. 32) the settlement under the foundation can be large in comparison to the estimated settlement and is not affected until the end of the consolidation process. the excess pore water pressures dissipate first close to the drainage layers in the clay (Broms, 1987).

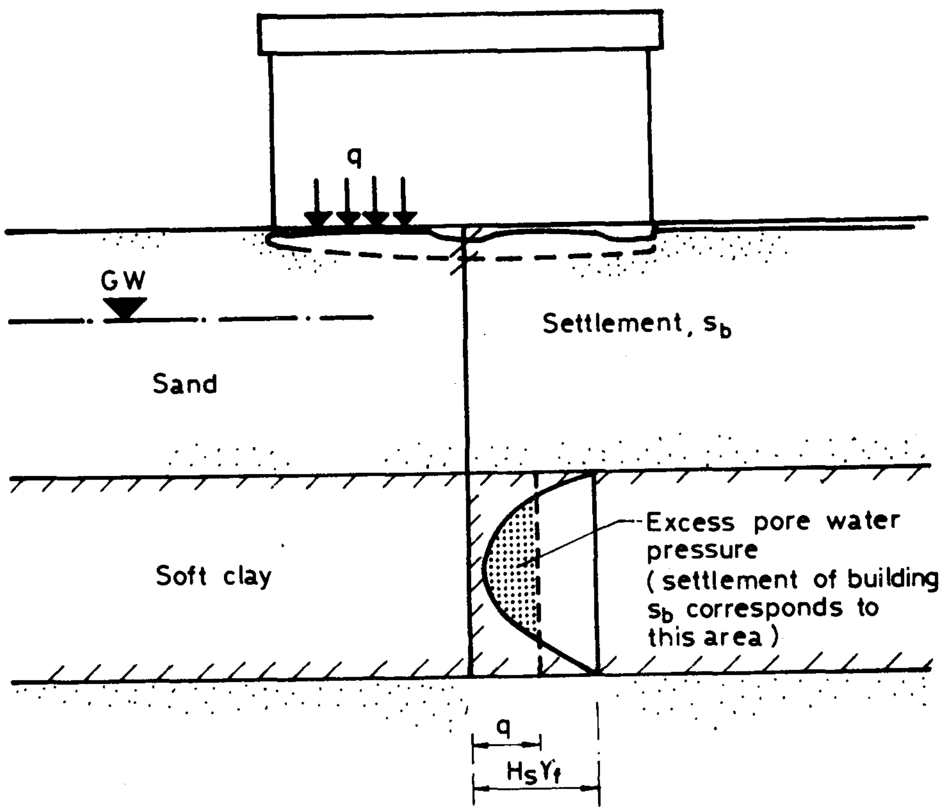


Fig. 32 Settlements after preloading (after Broms 1987)

If the preloading is interrupted when the settlements correspond to the settlements of the future buildings, without the preloading, then there will be an excess pore water pressure in the centre of the clay layer (Fig. 32). The building may then be damaged by the settlement when the remaining pore water pressure dissipates.

Thus reduction in pore water pressure in the soft saturated clay deposit under load, leads to compression in the compressible startum with time. Bacause of this compression, there is improvement in the clay strength and the time for which the soft clay stratum has been subjected to load, is known as rest time.

In the present case almost a year had passed since the constrution of building blocks was completed. Thus the weak clay stratum (3.5 m - 4 m thick) under the murum pad has been subjected to almost 90 per cent of the design load (18 t/m^2) for a period of about 12 months. During this period, the clay stratum must have undergone settlement resulting into improved shear strength.

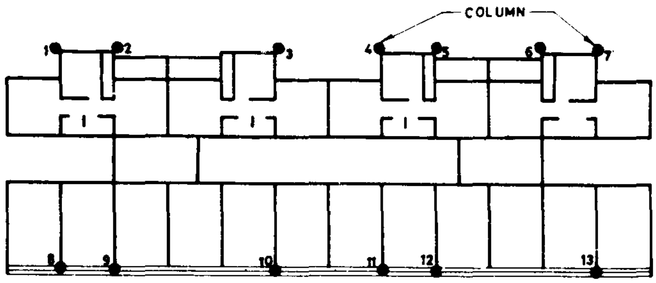


FIG. 33: GROUND FLOOR PLAN BUILDING- G10

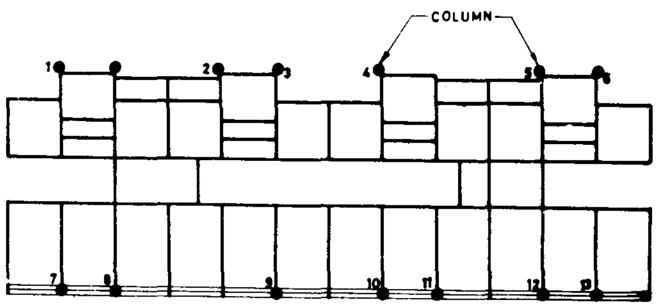


FIG. 34 GROUND FLOOR PLAN BUILDING- K2

MONITORING OF THE SETTLEMENT DURING REST-TIME

Regular settlement measurements on the 13 RCC column footings of blocks No. G10 and K2 (Fig. 33 and 34) were carried out to have a check on the total and differential settlement of the column footing foundation. The settlement records for 18 months for the 13 columns footings for these two blocks have been presented in Fig. 35 & 36. A glance at the time settlement curve of block G10 and considering the upper limit.

It is clearly demonstrated that the settlement in the initial stages (first 90 days) were 36 mm which decreased to 22 mm in the next 3 months. Further lapse of time indicated that the settlement in another three months were found to be 12 and 5 mm only. Thus the rate of settlement which was initially 12 mm per month was found to reduce to only 1.7 mm after an elapse of 12 months. Further, it is also noticed clearly that the time-settlement curves beyond 12 months becomes fully asymptotic along x-axis, indicating clearly that settlement had fully stablised well before 18 months. Similar behaviour was observed from the time-settlement curve for block No. K2 also.

CHECK FOR IMPROVEMENT IN CLAY STRENGTH

Improvement in clay strength due to sustained load for a period of over 12 months, was determined through (i) Dynamic cone penetration tests on Virgin and improved ground and (ii) laboratory shear strength tests on undisturbed samples.

DYNAMIC CONE PENETRATION TESTS

Dynamic cone penetration tests were carried out starting from the present ground level surface (after raising the level by 1.2 m) at three loccations BH1, BH2 and BH3 (Fig. 18). These tests were performed far away from the existing blocks of the buildings. Further to confirm the improvements in clay strength Dynamic Cone penetration tests were also carried out on all the 14 existing blocks G1 to G10, F1, F2 and K1, K2, at locations very near to the column footings. The DCPT results have been presented in Figs. 37 and 38. In Fig. 37 which shows the N_c values of the Virgin ground from 1.2 m to 7.2 m depth and on the same figure improvement in or increased in N_c values of blocks G_2, G_5, G_6, G_8 & K_1, K_2 have been superimposed. Similarly in Fig. 38 the improved

TABLE 5 (a)

IMPROVED UNCONFINED COMPRESSIVE STRENGTH

Depth (m)	Water Content (%)	Bulk Density (t/m^3)	UCC (Kg/cm^2) sample (1)	UCC (Kg/cm^2) sample (2)	Undrained Cu(Kg/cm^2)
0.5	24	1.93	1.20	1.12	0.56
1.6	30	1.82	1.48	1.10	0.55

values of N_c for the blocks G_1, G_3, G_4, G_7, G_9, G_{10} and F_1, F_2 have been super-imposed. The study of Figs. 37 and 38 clearly demonstrate an improvement in cone penetration value by a minimum of 100%. While considering the lower limit conservatively.

The clay stratum immediately below the murum pad (12 m to 21 m depth) has undergone more compression than the lower clay stratum (2.1 m & 3 m depth). The average improved N_c values in the upper clay stratum is found to be equal to 8 and for the lower stratum N_c is 4. Thus the improvement in strength of the upper clay layer is found to be 400 per cent and in the lower layer it is 100 percent.

POST CONSTRUCTION LABORATORY STRENGTH EVALUATION OF MARINE CLAY SAMPLES

In addition to dynamic cone penetration tests and with a view to further confirm the earlier conclusions, undistrubed samples of the marine clay deposits were collected under all the precautions and utmost care by

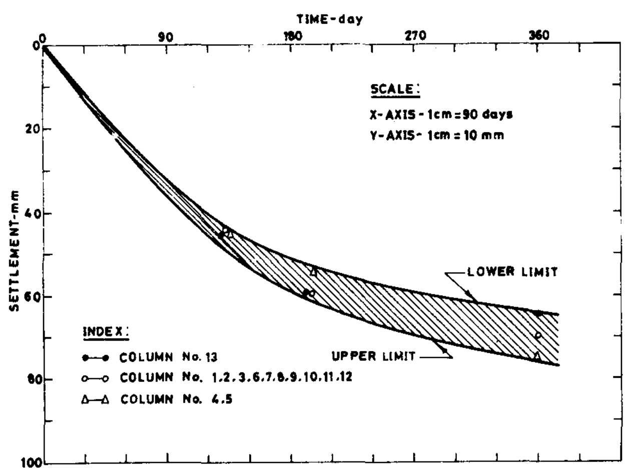


FIG. 35 : TIME-SETTLEMENT CURVE FOR BUILDING BLOCK-G10

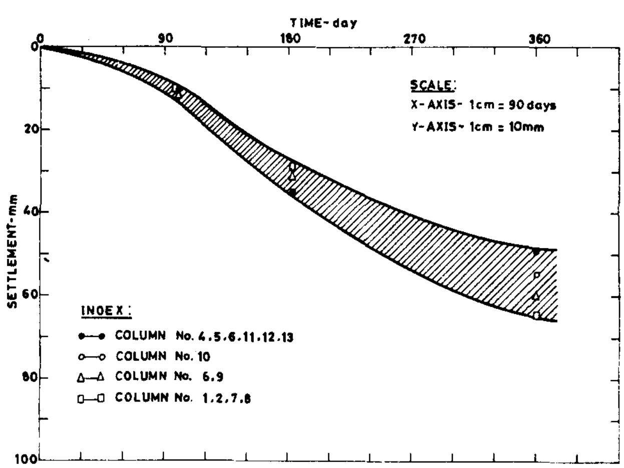


FIG.36 : TIME-SETTLEMENT CURVE FOR BUILDING BLOCK-K2

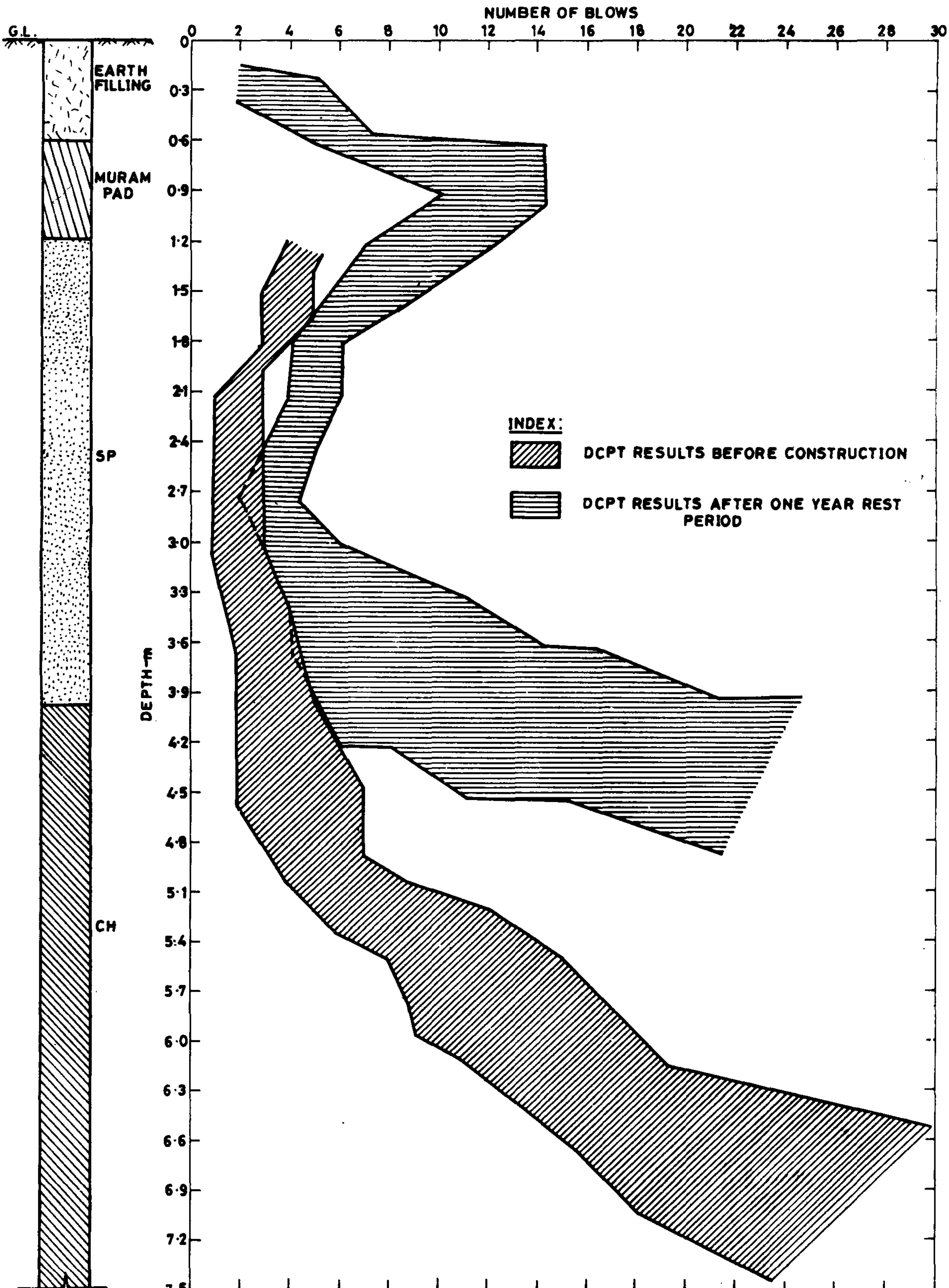


FIG. 37 : DYNAMIC CONE PENETRATION TEST RESULTS BEFORE CONSTRUCTION AND AFTER ONE YEAR OF CONSTRUCTION

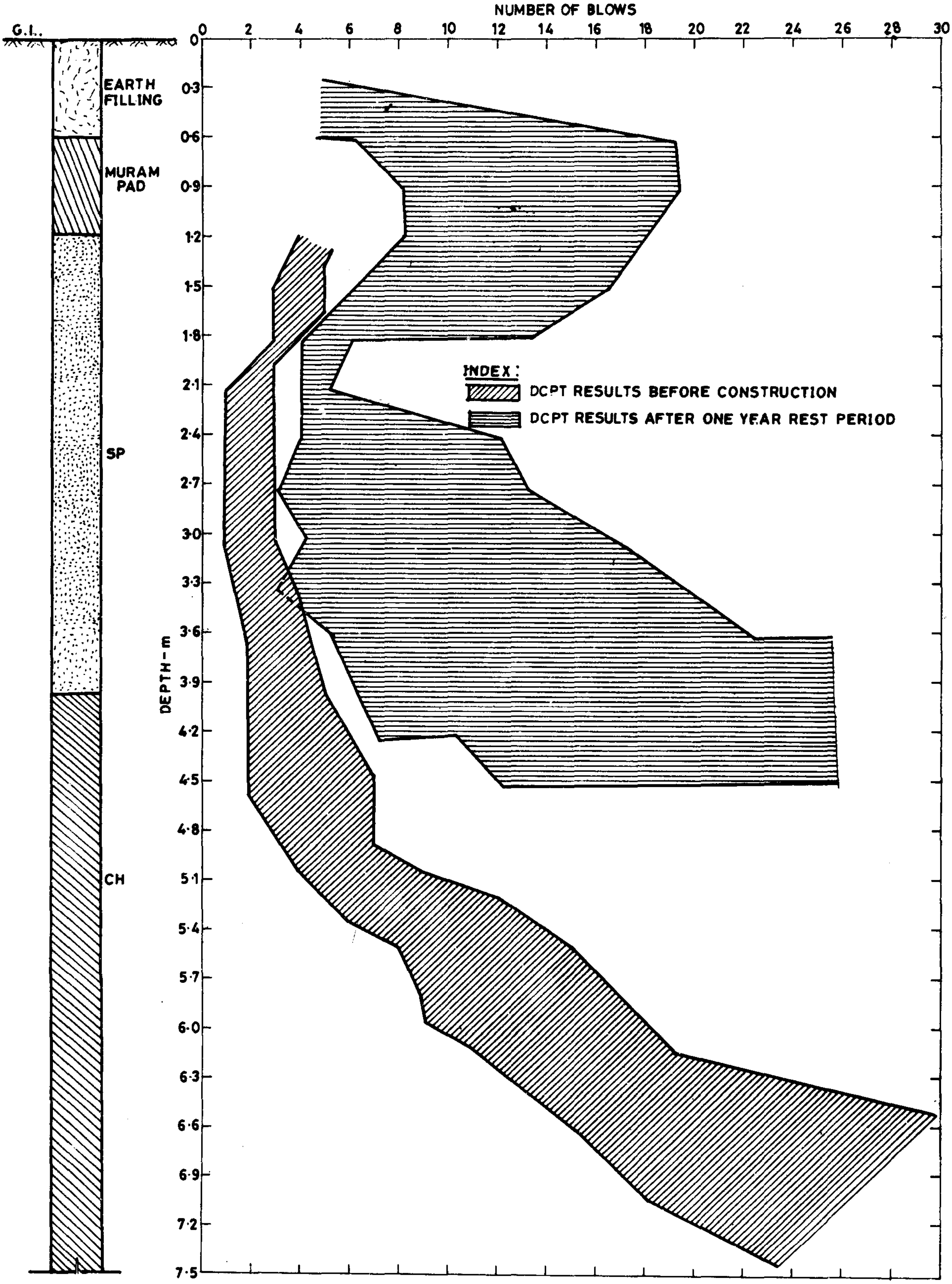


FIG. 38: DYNAMIC CONE PENETRATION TEST RESULTS BEFORE CONSTRUCTION AND AFTER ONE YEAR OF CONSTRUCTION

opening a pit by the side of the footing from 0.5 m and 1.6m depth , in the open courtyard.

The laboratory test results on these samples indicated presence of silt and clay content between 93 and 95 per cent. The P.I. values were found to be 39 indicating highly plastic clay, classified as CH.The natural water content was found to be 24%.

The unconfined compressive strength values for two samples have been given in Table 5(a). The lower strength for 0.5m depth sample wea 1.12 Kg/cm^2 with corresponding values of undrained cohesion C_u equal to 0.56 and 0.55 Kg/cm^2 respectively. These values check well with those determined from the cone penetration test results earlier.

INCREASE IN SHEAR STRENGTH

Further it may be mentioned that the natural water content of the marine clay sample from 1.85 m and 3.15 m were 46% and 57% respectively. However, the water content after the Rest-Time was found to be 24% and 30% which clearly indicate a significant reduction in water content due to dissipation of pore water pressure due to settlement (compression) in marine clay layer.

The increase in shear strength can be estimated from plasticity index of the undisturded clay (Brom, 1987) At a PI of 45 which is typical of marine clays a change of water content of 6.3 per cent the average change in shear strength is found to be 90 percent.

In the light of the foregoing para, the improvement in shear strength is found to be about 140 per cent from C_u equal to 2 t/m^2 to C_u equal to 5.5 t/m^2 which appears to be reasonable.

Therefore, it may be appropriate and rational to recognise the improvement in clay strength due to REST-TIME EFFECTS based on the check test results obtained from the field and laboratory tests, in the bearing capacity computation.Hence for the assessment of bearing capacity after the Rest-Time, the improvement in clay strength by 100 percent (C_u=2t/m^2 to 4t/m^2) is appropriate and fully justified. This is likely to govern the future performance of the RCC column- footing foundations supporting the ground plus three storey of the various blocks of the buildings.

ASSESSMENT OF THE IMPROVED BEARING CAPACITY

Recognising the influene of the REST-TIME in the improvement of the strength of the weaker clay layer deposit, the ultimate bearing capacity of the two layer system, utilising the improved parameters of the soft clay, was computed.

Input data

Murum pad :

$C_1 = 6$ t/m^2, $\phi_1 = 13°$, $\gamma_1 = 1.9$ t/m^3

Improved Clay layer :

C_2=4 t'/m^2, $\phi_2 = 0$ γ_2 =1.82 t/m^3

B = 1.8 m, Df = 0.6 m, H = 0.6 m

The buildings have undergone a settlement of 70-80 mm during the rest-time of about 12 months. This amount of settlement must have caused sigificant improvement in the available bearing capacity now. Thus for a square footing the ultimate bearing of the weaker clay layer q_b is equal to 27.04 t/m^2

Further q_t = 22.66 t/m^2 and q_1 = 59.65 t/m^2

Thus the ultimate bearing capacity qu of the two layer system after incorporating the effect of rest-time, and using Eq 30 is found to be as 36.64 t/m^2 for H = 0.6m

$$q_u = q_b + 2\left[\frac{2\ CaH}{B}\right] S_a + 2\gamma_1 H^2 \left(1 + \frac{2D_f}{H}\right) \frac{K_s \tan\phi_1}{B},\ S_s + \gamma_1 H$$

Since Sa = 1, S_s = 1.2, Ca = 5.4

$$q_u = 27.04 + 2\left(\frac{2 \times 5.4 \times H}{1.8}\right)(1) + (2)(1.9)(H^2)\left(1 + \frac{2 \times 0.6}{H}\right)\left(\frac{2.5 \times 0.23}{1.8 \times 1.9h}\right)$$

$$(1.2)\left(\frac{2.5 \times 0.23}{1.8}\right)(1.2) + 1.9\ H$$

$= 27.04\ 12\ H + (3.8\ H^2 + 4.56\ H)\ (0.31) + 1.9H$

Thus $q_u = 27.04 + 15.3H + 1.178\ H^2$

Since H = 0.6m $\therefore q_u = 27.04 + 9.18 + 0.424 = 36.644 t/m^2$ that it will satisfy the following relationship.

$q_u = 27.04+15.3H + 1.178\ H^2 = q_{ut} = 122.66\ t/m^2$

Thus, H = 4.6 m and H/B =2.5

Since H = 0.6 m in the present case,

$q_u = 36.64\ t/m^2$

Taking a F.S equal to 3, and accepting shear mode of failure the safe bearing capacity of the present two layer subsoil- footing foundation is found to be 12.2t/m²

$q_s = 36.64/3 = 12.21\ t/m^2$

Recalling the earlier observations that the desing intensity due to the heaviest column footing 54 tonnes and 40 tonnes were found to vary between 7 t/m² to 9.3 t/m² on the clay layer, are found to be well within the safe bearing capacity of 12.2 t/m².

Therefore, it may be concluded that the present foundation system adopted for the ground plus three storey buildings, was considered safe against bearing capacity failure.

LONG TERM POST CONSTRUCTION SETTLEMENT

A close examination of the settlement observations on block G 10 and K2 (Fig. 35) on the various colummns, it was noticed that the rate of settlement in the column footing (Block G 10) which was intially 12mm per month, reduced to 1.7mm after a lapse of 12 months . This rate is likely to further reduce Also the time settlement curves reflect towards stabilisation with the further lapse of time. Similar behaviour was also observed in Block K2 (Fig. 36). However, the importance of secondary consoilidation on the overall behaviour of the present blocks of buildings can not be overlooked. State of the art on the subject reveal that with the use of preloading though the secondary settlement can be reduced or eliminated, however, the effectivness of the method is diffcult to predict from laboratory tests. Koutsoftas et. al. (1987) have reported a reduction in the value of coefficient of secondary consolidation C_S by 60 per cent from a 25 per cent surcharge (over loading).

It can therefore be inferred that though secondary consoilidation will continue to add to the settlement with the passage of time it may not be significant to cause any anxiety. However, in case, the past history of the locality indicate any chances of natural disaster such as earthquakes submergence due to flooding or scour in the time to come, provision of a reinforced cement concrete skirt may be considered around the blocks of buildings encompassing the exterior column footings in accordance with Fig. 39.

CONCLUDING REMARKS :

Based on the detailed field and laboratory tests and the analysis of the data

It was concluded that the foundation system adopted for the ground plus three storey buidings may be considered adequate to support the design load without any distress. However in case of any further settlement occuring, shall essentially be elastic with, corresponding negligible differential settlement thus the structure is expected to adjust such additional stresses through soil structure interaction. These building have not shown any undesirable problems and since have been (1988) occupied.

OIL DRILLING RIG FOUNDATION

INTRODUCTION

The drilling site was situated in district Cachar, Assam by the side of a river which used to flood the site during monsoon. The Rumanian drilling rig (F400-4DH) was to be erected for oil exploration.

The loading plan due to dead weight and also during operation of the rig have been shown in Fig. 40. The total load due to the weight of the drilling rig, weight of casing and load due to drilling operation, etc. is supported by two concrete pedestals 0.3m wide, 8.51m, long on either side of the drilling well in the cellar pit, the load intensity on each pedestal varied from 4 Kg/cm² to 12.5 Kg/cm². Besides, other footings, which are subjected to a load intensity of 1.0 Kg/cm², in the rear and in front have not been shown in the figure.

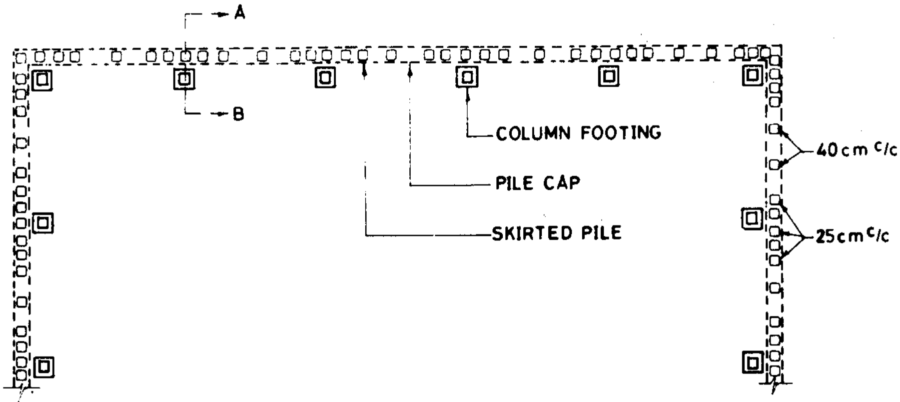


FIG. 39(a): PLAN SHOWING THE LAY-OUT OF R.C.C. SKIRT

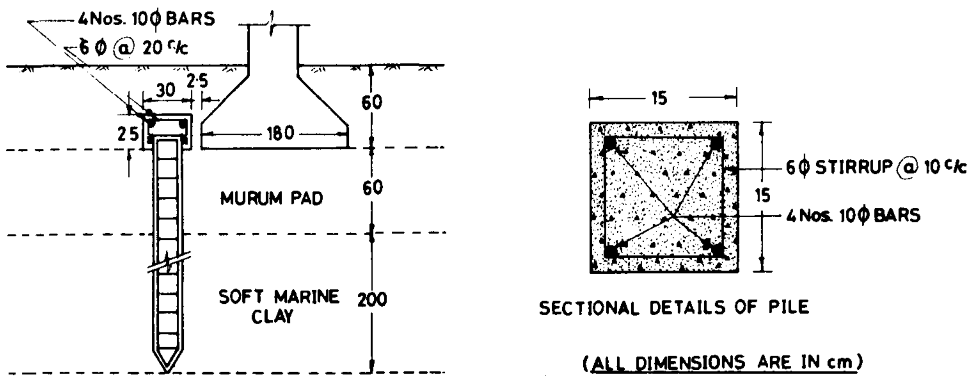


FIG. 39(b): SECTION-AB SHOWING THE DETAILS OF THE SKIRTING EDGE BEAM AND THE FOOTING

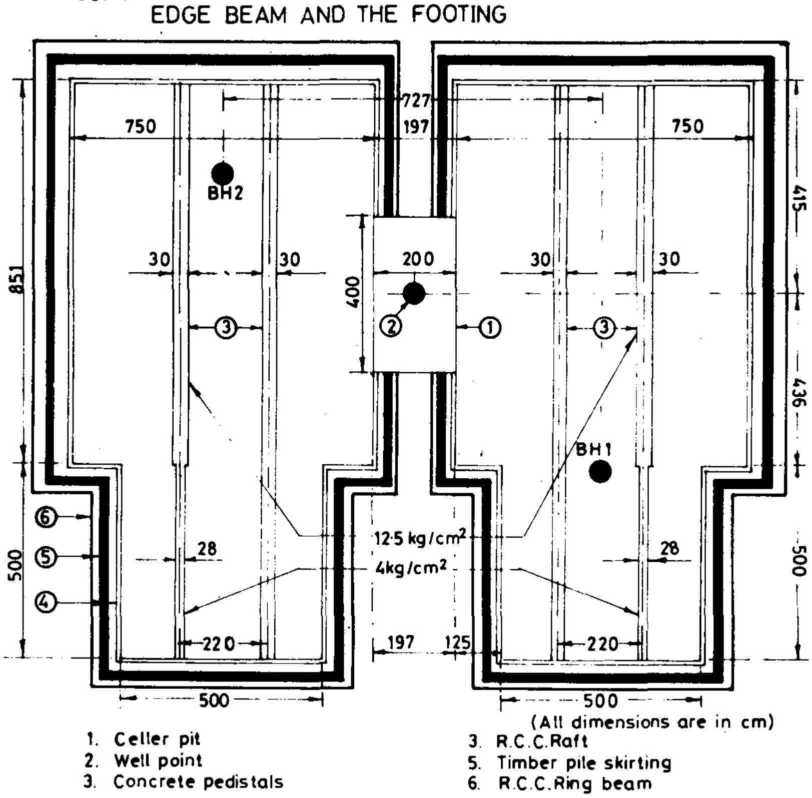


Fig.40 LOADING PLAN

SUB SOIL CHARACTERISTICS :

The field investigation like boring, sampling and standard penetration tests and also detailed laboratory tests were completed in December, 1978.

WORK PLAN :

The work plan included (a) Design analysis of a cost effective and efficient foundation for the (F400-4DH) Rumanian Rig based on the sub soil investigations, both in the field and laboratory, (b) Drawing construction specification of the proposed foundation, (c) modifying the design/ specification based on the insitu load testing and (d) Providing occasional guidance and supervision during the construction of the foundations.

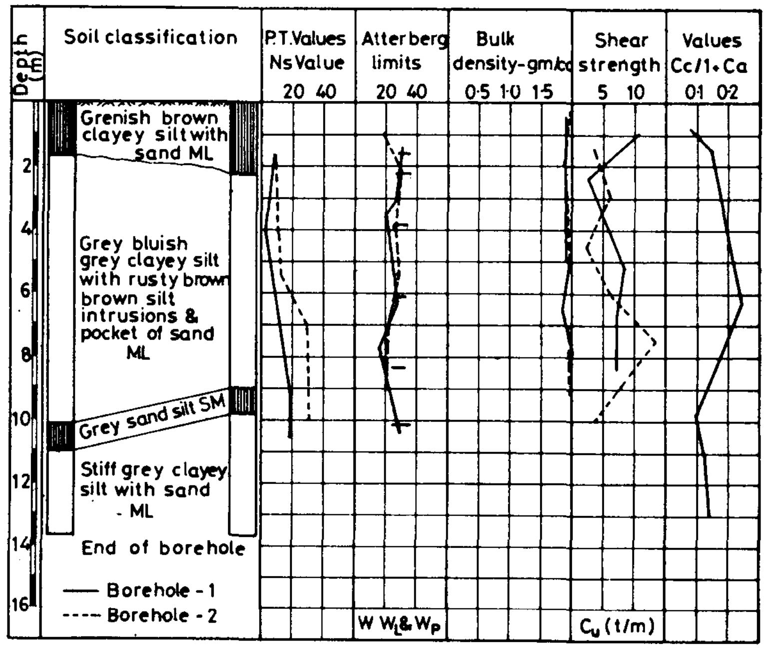


Fig.41 SUB-SOIL CHARACTERISTICS

FIELD & LABORATORY TESTS :

During the field test two bore holes 150mm diameter and 20m deep were sunk, one each under the proposed raft. Undisturbed sampling and standard penetration tests were carried out during boring operation besides collection of disturbed samples also for classification purposes, (Fig. 41).

The bore-holes data revealed the presence of clayey silt upto a depth of about 2m overlaid on 8m thick deposit of grey bluish grey silt with brown silt intrusions and pocket of sand. Beyond this depth a 0.6-0.7 m thick layer of grey sandy silt was noted. Further extension of bore hole indicated the presence of stiff grey clayey silt layer with sand. The SPT values, N_s in bore I and II have been shown in fig. 41. The bulk density was found between 1.9-2.07 g/cc and undrained shear strength was found to vary between 0.49 and 0.51 kg/cm^2.

Two large pits one meter deep, following the shape of the two rafts with a clear distance of 200 cms, between them, had already been made at the site. The position of cellar pit and BH-I and 2 are marked on Fig.40.

The sub-soil classification along with the bore log 2, standard penetration test results, Atterberg limits, bulk density and shearstrength have been presented in Table 6 and Fig. 41 respectively. The water table at the time was located at 1.6-1.7 m below the ground devel. However, it was likely to be submerged during monsoon due to floods in the river by the side of the site.

FOUNDATIONS LOADINGS :

The study of the loading plan (Fig.40) indicates that the portion AB of the loading strip (28 cm wide) is lightly loaded and is subjected to a stress intensity of 4 kg/cm^2 and the portion BC is subjected to higher stress intensity of 12 kg cm^2 on a loading strip of 30 cm width and 8,51 m in length. A clearance of about 100 cm was required to be left between the two edges of the raft on either side of cellar pit alongwith the well point. Thus

TABLE 6

DEAILS OF BORE-LOG AND SOIL PROPERTIES

Stratum	Depth (m)	Description of the soil	LL (%)	PI (%)	γ bulk (t/m²)	C_u (un-drained strength (t/m²)	Compressibility $\frac{(C_c)}{1+e_o}$
I	Upto-2	Greyish brown sandy clayay silt - ML	38	13	2.0	4.0	0.1
II	2-10.5	Medium grey/bluish grey clayey silt with pocket if sand -ML	38	14	1.9	5.0.8.0	0.12
III	10.5 11.20	Grey sandy silt lying immediately below stratum II SM	-	-	2.0	Ns - 20	D_r =50 %
IV	Beyond 12m	Stiff grey clayey silt with sand -ML	37	13	1.9	9.0	0.1

* Classification as per Indian Standard (IS : 1498-1959)

**Standard penetration test value, N_s

the maximum width for the two symmetrical rafts on each side of the well is 750 cm. Such a raft if placed at 100 cm below the ground level in stratum I, its pressure bulb will extend in Stratum II also (Table 6.)

BEARING CAPACITY COMPUTATION :

The stress intensity on 30 cm wide loading strip = 12.5 Kg/cm² and the load per cm run both the strip = 2 X 12.5 X 30= 750 Kg. Therefore the total load on each raft = (750X851) 1000 + 638.25 tonnes. Hence design stress = 638.25/63.825 = 10 t/m² = 1 Kg/cm².

The average cohesion upto a depth of 10 m is taken as 0.48 Kg/cm² in accordance with Table 7 and Fig. 41.

SETTLEMENT COMPUTATION

The immediate settlement of the raft was computed from Eq. 31 as given below :

$$S_1 = \frac{q\,B(1 - \mu^2)\,I_p}{E_s} \qquad \text{...(31)}$$

Substituting the appropriate values for different parameters as :

Width B of the equivalent raft = 7.5 X 8.51

q = 1 Kg/cm², μ = 0.50, I_p = 0.9,B = 800 cm, E_s=45 Kg/cm²

The immediate settlement S_i is given by

$$S_i = \frac{qB(1 - \mu^2)\,I_p}{E_s}$$

$$S_i = \frac{1 \times 800(1 - 0.25)\,0.9}{45}$$

Immediate settlement = 12.00 cm

CONSOLIDATION SETTLEMENT :

The consolidation settlement of the raft placed at Im below the ground surface was computed from Eq. 32. The subsoil stratum was divided into four layers thickness as indicated in Fig. 42 and values of compressibility from Fig 41. Similarly the values of P_o and Δ p were also taken from Fig 42 and appropriately subsituted in Eq. 32 for different layers.

TABLE 7

VALUE OF THE UNDRAINED SHEAR STRENGTH

Depth of soil samples (m)	Value of C_u Kg/cm²		Average value of
	BH I	BH II	Cu Kg/cm²
1.51 - 1.85	1.1		$\frac{\text{Av.}C_u \text{ BH I} + \text{Av. } C_u \text{ BH II}}{2}$
2.8 - 3.20		0.43	$\frac{0.46+0.51}{2} = 0.48$
4.5 - 4.80	0.39		
6.0 - 6.30		0.60	
7.5 - 7.95	0.59		
9.0 - 9.3		0.35	
10.5 - 10.93	0.84		

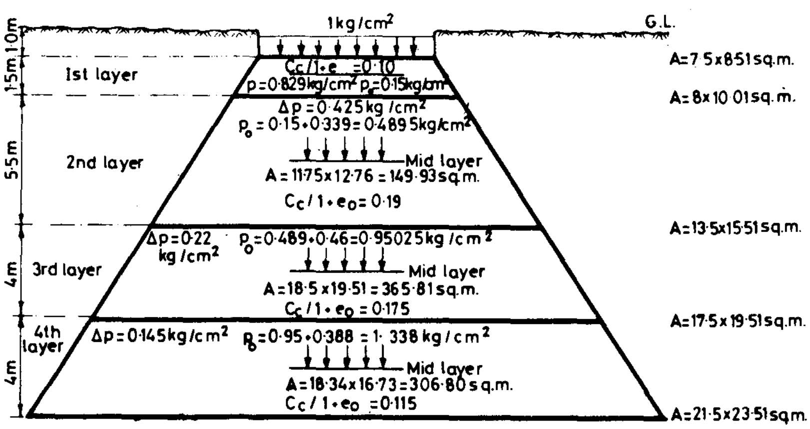


Fig.42 COMPUTATION OF CONSOLIDATION SETTLEMENT OF THE VIRGIN GROUND

$$S = H. \frac{C_c}{+e_o} \log_{10} \left[\frac{P_o + \Delta P}{P_o} \right] \quad ....(32)$$

$$\text{Settlement of Layer I} = 150 \times 0.1 \log_{10} \frac{0.15 + 0.8294}{0.15} = 12.22 \text{ cm}$$

Similarly the settlement of IInd to IVth layers were 28.38cm, 5.14 cm & 1.43 cm respectively.

Thus the total settlement of the raft = 47.15 cm

Since the compressive layer was thick and unrestrained, the lateral deformation of ground under applied load of 10t/m² may significantly alter the consolidation behaviour of the stratum.

According to Skempton & Bjerrum (1957) the settlement (St) after a time t is given by Eq.33.

$$S_t = S_i + U\lambda S_{oed} \quad ...(33)$$

where S_t = settlement after time (t), S_i = immediate settlement, U = degree of consoildation, λ = factor taken as 0.8 and the degree of consolidation as 0.9 for a normally consolidate clay.

Thus the total settlement (S_t) was found as

S_t = 12.00 + 0.9 X 0.8 X 47.158 = 45.95 cm

Hence the raft foundation was not considered safe from settlement considerations under the load of 10 t/m².

ELASTIC SOIL MODULUS :

The elastic soil modulus E_s has been correlated with undrained shear strength, C_u from unconsoilidated undrained UU test on soft to medium consistancy cohesive soils and given by Eq. 34.

$E_s = 500\ C_u$ or 20 to 40 kg/cm²

For normally consolidated sensitive clays and for sensitive clays having OCR less than 2, the value of Es is given by Eq. 35 (Bowles 1977).

$E_s = 1000\ C_u$ or 40 to 80 kg/cm² .....(35)

For very stiff clay

$E_s = 1500\ C_u$ or 80 to 200 kg/cm² .....(36)

For silty soil (Lambe & Whiteman 1969)

E_s = 20 to 200 kg/cm² .....(37)

For silt and sand, Begmann (1974) has recommended Eqs 38 & 39

$E_s = 40 - C\ (N - 6)$ for $N > 15$ .....(38)

$E_s = C\ (N + 6)$ for $N < 15$ .....(39)

Where C = 3 for silt and sand

For cohesionless soils Vesic (1967) proposed the following relation correlating relative density D_r and static cone resistance q_c Eq. 40.

$E_s = 2(1+D^2_r)\ q_c$ kg/cm² .....(40)

For clayey sand Bachelier and Parez (1965) have proposed Eq. 41.

$E_s = (1.3 - 1.9)\ q_c$ kg/cm² .....(41)

and clayey sand below water table Webb (1969) proposed Eq. 42.

$E_s = 5/3\ (15 + q_c)$ kg/cm² .....(42)

Schemertmann (1970) has recommended Eq. 43 for poorly graded submerged sand and clayey sand.

$E_s = 1.67\ (15 + q_c)$ kg/cm² .....(43)

The average SPT, N_s values upto 9m depth was found to be 9 number of blows and between 9 to 16 m depth it was 15 number of blows indicating thereby the corresponding values of E_s to be between (40-80) kg/cm² and (80-200) kg/cm² (Lambe & Whiteman, 1969).

Utilizing $q_c = 2N$ upto 9 m depth and (3N) between 9m-16m and substituting $C_U = 0.48$ kg/cm² in the appropriate equations the elastic soil modulus values were found to vary between (34-55) kg/cm² and (85-100) kg/cm² respectively. Therefore, $E_s = 45$ kg/cm² for strata upto 9m and 100 kg/cm² between 9 m - 16 m was adapted for the design.

CHOICE OF FOUNDATION :

In view of the excessive settlement due to design load various options were considered. The partial or full replacement of the weak soil stratum has been an age old approach with diminsihing trend of application. In view of the current accent towards heavier structures with more sringent performance requirement such as for the (F 400-4 DH) Rumanian oil drilling rig foundation. Transporation of huge volume of soil adding towads overall cost besides ecological degradation and time delays were found to be the severe constraints. The other option could have been deep piling, or caisson foundations. Non-availability of agencies and the equipment in the remote areas, implications of methods of installation of piles or that of advancing of a pier and ignorance about the magnitude of negative drag which is almost a certainty in compressible soil stratum, over conservation in design gain favour.

There are number of case records of cost effective and successful foundations resting on weak soil deposits improved by preload actuated sand drains, sand wicks band drains, also serve as means of accelerating effect they provide. Since considerable amount of settlement may take place during the stage of ground treatment such foundations are often trouble free from the long range maintenance point of view. However, the time required for preloading is large in relation to limited construction time available hence the method was not preferred.

In view of the discussions noted above skirted granular pile foundation (Rao et al. 1979) with a retrievable skirting fabricated out of waste mild steel pipes or contiguous timber piles joined together at the top by a rigid RCC edge beam at the cut off level was favoured.

STRAIN COMPATIBILITY PROCESS

Load of the structure is transmitted through the concrete footing or raft to the surface of the cohesive soils reinforced with granular piles. A large portion of the total load is initially resisted by the very dense granular piles which are rigid relative to surrounding soft cohesive soils. The remaining load is carried by the soft ambient cohesive deposit in contact with the footing. The initial load on the pile top produce radial strain in the soft cohesive soils surrounding the roughly cylindrical walls of granular pile, thereby mobilising the radial resistance of the soil (Thornburn, 1975). The magnitude of the radial strain necessary to develop signifcant radial stress are small due to the fact that considerable radial preloading that occurred during the construction of the granular pile. The vertical strain in the granular pile due to the application of the structural load cause transference of load from the yielding pile top to the soft soil under the footing. As the consolidation of the soil takes place which then experience further vertical strains till an equilibrium condition is reached.

The strengthening of ground may be accomplished by compaction if the ground is non-cohesive, or by providing reinforcement in the form of uncemented piles of aggregates. pile foundations need not be a first consideration (Thorburn, 1975). Granular piles, though have been used widely to support large, embankments etc. the design methods are however semi empirical (Hughes & Withers, 1974).

ULTIMATE PILE CAPACITY :

SEMI EMPIRICAL APPROACHES

In semi empirical approaches, the total structural load is assumed to be entirely supported by granular piles. Such an approach though ensures adequate factor of safety with respect to the bearing capacity of the reinforced soil and provide ground for considerable stiffness, but are highly conservative.

The pile capacities were computed using various approaches such as Thorburn and Mac Vicar (1968), Hughes and Withers (1974), Hughes et al, (1975) and Mori (1979). The ultimate capacity and the stiffness of the ground may be varied by permitting a portion of the design load to be shared be ambient ground however the precise theoretical solutions for the prediction of strength and stiffness of the reinforced ground are yet to be established (Thorburn, 1975).

Input data

Cohesion $C_u = 4.8$ t/m^2

Elastic soil modulus $E_s = 450$t/m^2

Pile diameter = 0.45 m

Elastic modulus of pile material $E_p = 2300$ t/m^2

Angle of friction of pile material (Engelhardt & Golding 1975) = 30° = ϕ

Submerged density $\gamma_{sub} = 1.00$ t/m^2

Bulk density 2.0 t/m^3

(1) Hughes and Withers (1974)

$$q_{ult} = 25.2\ C_u \qquad \text{...(44)}$$

Substituting appropriate value, the ultimate bearing capacity

$$q_{ult} = 25.2 \times 4.8 = 120.96 \text{ t/m}^2$$

(ii) Hughes et al. (1975)

$$q_{ult} = K(\sigma_{ro} + c_u) \qquad \text{...(45)}$$

Where σ_{ro} is the initial ground stress which is equal to 2 C_u and

$$K = \frac{1. + \sin\phi}{1 - \sin\phi} \qquad \text{...(46)}$$

Substituting the appropriate values

Lateral stress coefficient $K = 4.2$ $\sigma_{ro} = 9.6$ t/m^2

q_{ult} = 4-.2 (9.6 + 4 x 4.8) = 12.096 t/m²

(iii) Mori (1975)

The main difference between Hughes et al. (1975) method and Mori (1975) is that the initial ground stress is equal to 1/(2γ_{sub} . H) in place of 2C_u and H= 4 (diameter of pile).

Thus

$$q_{ult} = K(1/2\, \gamma_{sub} H + 5\, C_u) \qquad ...(47)$$

Hence substituting appropriate values, the ultimate load capacity, (q_{ult})

$$q_{ult} = 4.2\ (0.5 \times 1.0 \times 1.8 \times 5 \times 4.8) \qquad ....(47)$$
$$= 104.58\ t/m^2$$

(iv) Thorburn (1975)

The ultimate bearing capacity was found to be 157.5 t/m² (Rao, 1982; Ranjan & Rao, 1987- 1991)

MODIFIED CAVITY EXPANSION APPROACH :

In practice the load acts on a finite area of soil reinforced by granular piles. The applied load is shared by the granular pile and the weak ambient ground. The sharing of load is found by the compatibility of strains between the two as discussed earlier (Thorburn, 1975). Apart from the load supported by the ambient soft ground increases the mean normal stress (σ_m) within the soil mass, thus increasing the cavity expansion stress ($\sigma_m F'_q$). This is termed as effect of over burden due to applied load (Rao, 1982; Ranjan & Rao, 1991).

Utilizing appropriate values of the soil parameters obtained from field and loboratory investigations and using Eq. 6 provided earlier, the total ultimate load for a single pile was found to be 311.52 t/m² wherein the ultimate capacity of the pile alone was found to be as 157.5 t/m² and contribution of ambient ground in increasing the ultimate bearing capacity was 154.0 t/m². Thus the total ultimate capacity of a single pile was found to be 49.56 tonnes based on undrained shear strength C_u = 4.8 t/m², effective submerged unit weight γ_{sub}= 1.0 t/m³, the installed pile diameter d = 0.45 m and the critical pile depth L_c equal ot five times the installed piles diameter which is equal to 2.25 m.

In the above computation the total design load has been taken as 10 t/m² and the load shared by the pile (q_p) is found to be 8.33 t/m² and that by soil (q_s) is 1.67 t/m²

It is therefore, indicated that the load shared by the granular pile is 83.3 per cent of the total design load while 16.7 per cent is transferred to ambient ground. The observations are based on the concept that within elastic

TABLE 8

ULTIMATE FILE CAPACITY

	Ultimate Bearing Capacity t/m²		Total Pile Capacity	Ultimate Bearing Capacity (tonnes)
	Applied load is fully supported by pile only	Increase in pile capacity due to sharing of load by ambient soil		
(a) **Empirical Approaches**				
(i) Hughes and Withners (1974)	120.96	154.0	274.96	43.74
(ii) Hughes withers and Greenwood (1974)	120.96	154.0	274.96	43,74
(iii) Thorburn (1975)	146.7	154.0	300.7	47.84
(iv) Mori (1979)	104.58	154.0	258.58	41.07
(b) **Modified Cavity** Expansion Approach (Rao 1982)	157.5	154.0	311.52	49.56

limit, sharing of load between pile and soil is assumed to be proportional to their elastic modulii (Rao, 1982; Rao Ranjan, 1985-1988). Similar experiences have been reponted by Engelhardt & Golding (1975) for a weak cohesive sub soil deposit reinforced with stone columns in a rectangular pattern 1.22 m x 0.98 m wherein 83 per cent and 17 per cent were the applied load sharing between stone columns and surroundings ambient soil.

The computed ultimate bearing capacities obtained from various approaches have been presented in Table 8.

If the ultimate load capacity arrived at from the empirical approaches by different method are increased by additional loads due to load shared by ambient soil under the footing, the ultimate load capacities arrived at by these method are found to be conservative in comparison to Rao (1982) approach.

The study of the Table 8 indicates that the ultimate load capacity ranges from 41.0 tonnes from 47.84 tonnes using different semi empirical approaches and 49.56 tonnes from Rao (1982) approach which is in close agreement with Rao (1982) method. adopting conservatively an increase in load carrying capacityby 25 per cent based on Rao & Bhandari (1979) and Rao & Ranjan (1985) towards the contribution of collective skirting, the ultimate load capacity of a single skirted pile work out to be 62.0 tonnes. Thus,

safe load for each pile $= \left[\dfrac{62.0}{F.S.}\right] =$ 20.0 tones

Heavily loaded portion of the raft

Area of rcc raft =7.5 X 8.51 = 63.825 m²

Hence the total load on the raft with a safe bearing capacity if 10 t/m²

= 638.825 tonnes.

Thus number of piles under heavily loaded portion of the raft

$$= \frac{638.825}{20.0} = 31.9 \text{ say } 32$$

Providing 0.45 diameter granular piles at a spacing of three times the pile diameter (1.35m) and using triangular pattern, total number of piles required in the heavily loaded portion of the raft shall be 45. Also keeping the same pile spacing and adapting a triangular pattern 17 number of piles will be required in lightly loaded portion.

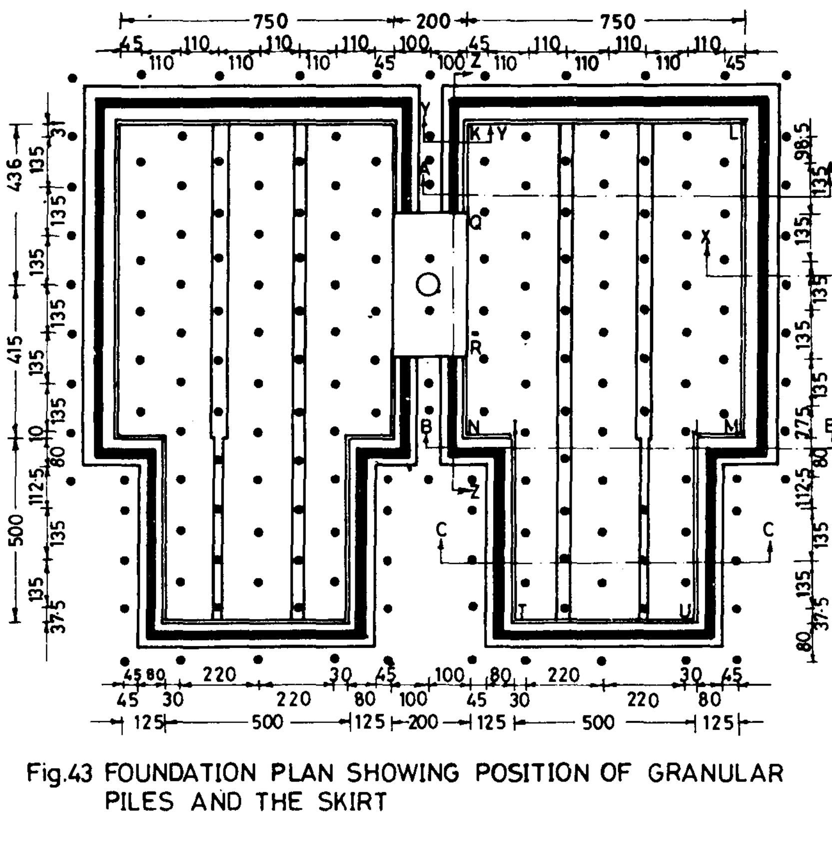


Fig.43 FOUNDATION PLAN SHOWING POSITION OF GRANULAR PILES AND THE SKIRT

In view of maintainig the pile spacing equal to three pile diameter and uniform distribution of piles. 45 Nos. is adapted under heavily loaded portion. Thus each raft under the rig will be supported by 62 numbers of granular piles (Fig. 43).

IMPROVEMENT IN BEARING CAPACITY :

The composite ground reinforced with granular piles will have an improved strength and deformation characteristics. The gain in strength though can be measured in the field by static or dynanmiccone penetration tests and compared with the resistance in virgin ground. However, there is no analytical approach availabel to in compute the improvement precisely.

Thus taking 20 tonnes as safe load for each pile

Total load taken by each raft = 62 x 20 = 1240 tonnes

Area occupied by 62 piles alone = 0.15991x 62 = 9.864 m^2

Total area of the raft = (8.51 x 7.5+5x5) = 88.825 m^2

Net area of ambient soil under raft = (88.825 - 9.864)= 78.968 m^2

Assuming a minimum of 50 per cent improvement in bearing capacity of weak ambient clayey silt deposit; the total load taken by the soil alone shall be = 78.968 x 15 = 1184.52 tonnes

Hence total load taken by each raft= 1240 + 1184.52

= 2424.52 tonnes

$$\text{Improved bearing capacity} = \frac{2424.52}{88.225} = 27.295 \text{ t / m}^2$$

$$\text{Hence the improvement} = \frac{27.295}{10} = 2.72 \text{ i. e. } 272 \text{ per cent}$$

Thus by adopting 62 number of granular piles having 0.45m installed pile diameter, placed at a spacing of three times the pile diameter under a RCC raft duly skirted with a rigid skirt, the improvement in safe bearing capacity of virgin silty clay deposit is expected to be increased by almost 272 per cent.

DEPTH OF GRANULAR PILES :

The depth of granular pile may be increased to reach the harder strata at shallow depth , but is normally determined by the depth of pressure bulb predicted by elastic theory, which is usually two to three times the plan width of the footing. However, the raft foundation or the closely spaced footings stress deeper zones. In practice, it has been experienced that depth of granular piles between 6 to 8 m below footing level is adequate (Greenwood 1970). This is found sufficient to develop arching in treated zone to bridge loose pockets in untreated zone below.

Study of Fig. 41 indicate that the average N_s value is found to be 9 number of blows upto 8 m depth and further increased to 20 number of blows beyond 9 m. It is further noted that undisturbed shear strength Cu (Fig. 41) is found to be 0.48 Kg/cm^2. Similarly the compressibility value ($C_c/1+e_o$) is found to increase from 0.15 to 0.25 at 1.5m to 8 m and then again decreases to 0.1 at 12 m and attains a value of 0.18 at 20 m depth. These data suggest that the strata upto 8 m depth is highly compressible and required ground treatment by any suitable technique which is speedy, efficient and cost effective. As indicated earlier, the depth of granular piles equal to 9 m could be ideally suited for the treatment.

CONSTRUCTION OF GRANULAR PILES :

The installation of granular siles is to be carried out by simple auger boring method (Rao, et. al. 1979, Rao 1982, Ranjan and Rao, 1983)using 20-70 mm crushed stone aggregates compacted by internal operating 200 Kg hammer through a tripod and winch. The set criteria was kept 20 mm for the last blow to achieve uniformity in compaction. The crushed stones may be replaced by river shingles of the same size, however, the strength of the pile is greater if the anglar fill material is used rather than round one (Thorburn, 1975).

In the present rig foundation, 124 granular piles were required under the two rafts and 52 piles are needed around outside skirt making a total number of 176 (Fig. 43).

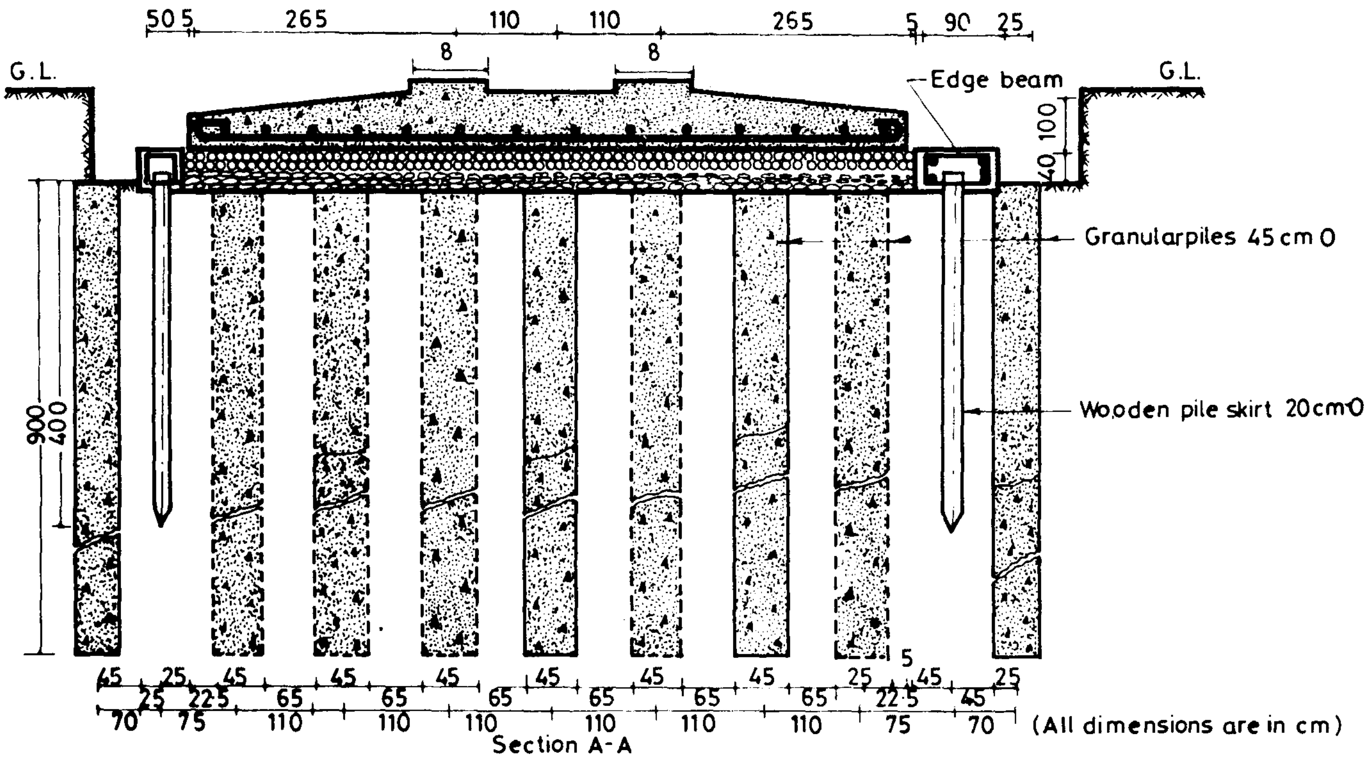


Fig. 44 SKIRTED GRANULAR PILE FOUNDATION
(Section A-A heavily loaded footing)

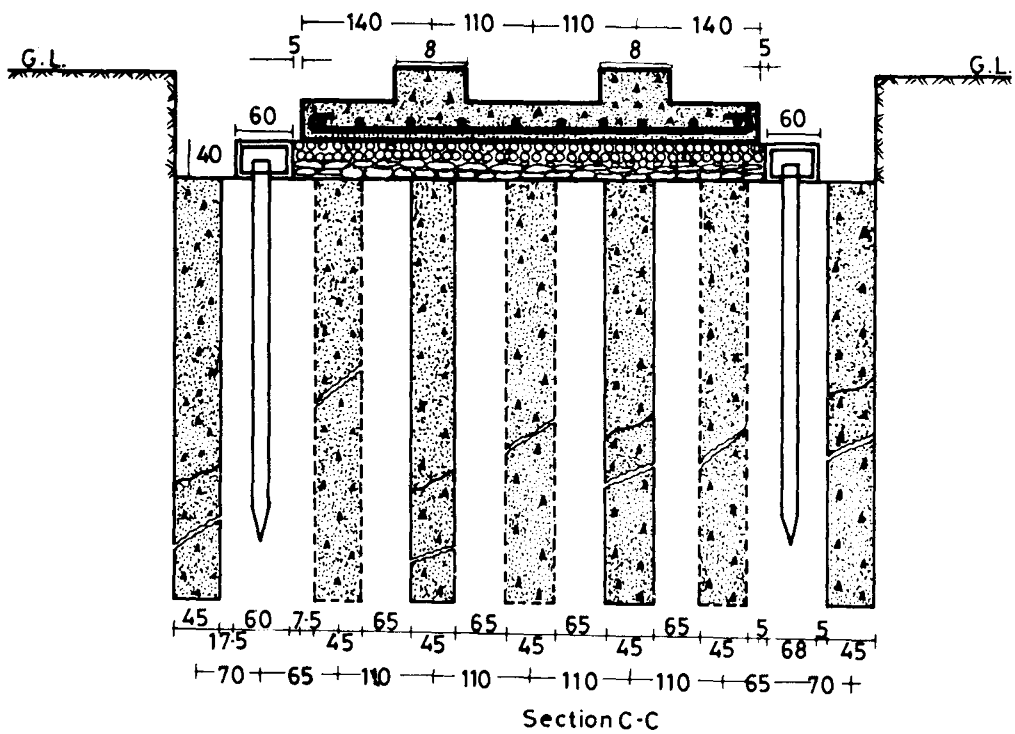


Fig. 45 SKIRTED GRANULAR PILE FOUNDATION

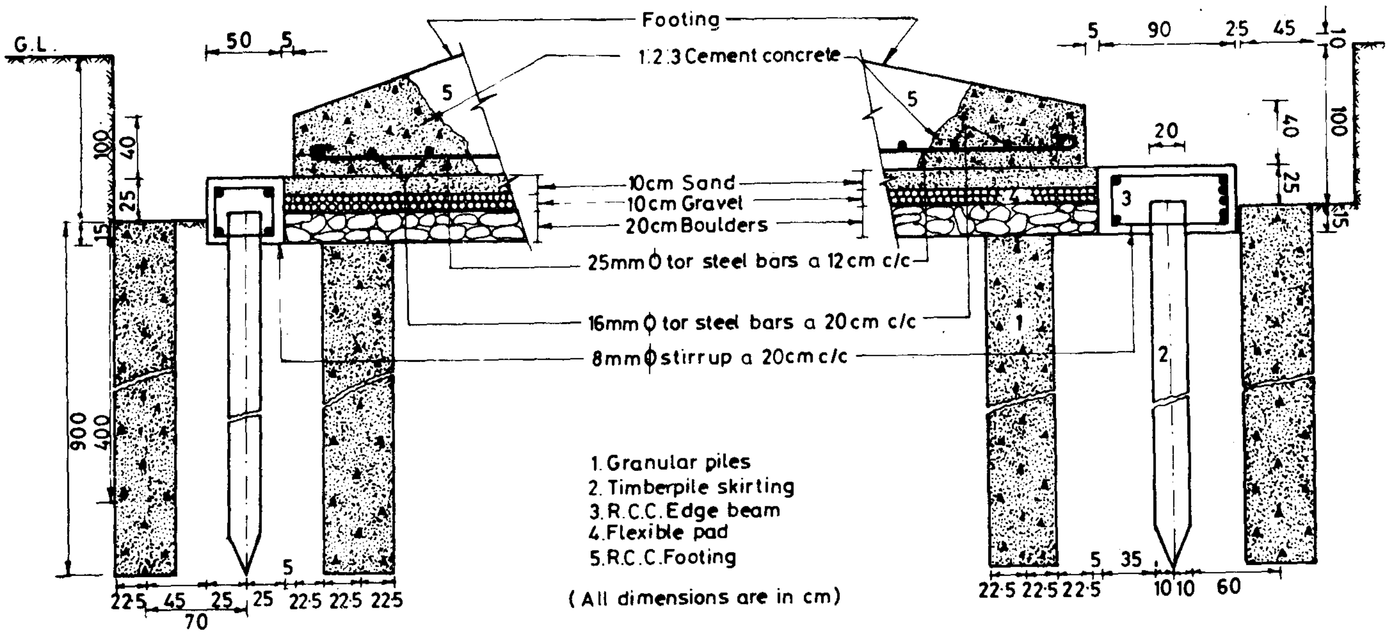


Fig.46 DETAILS OF EDGE BEAM & TIMBER PILE SKIRTING

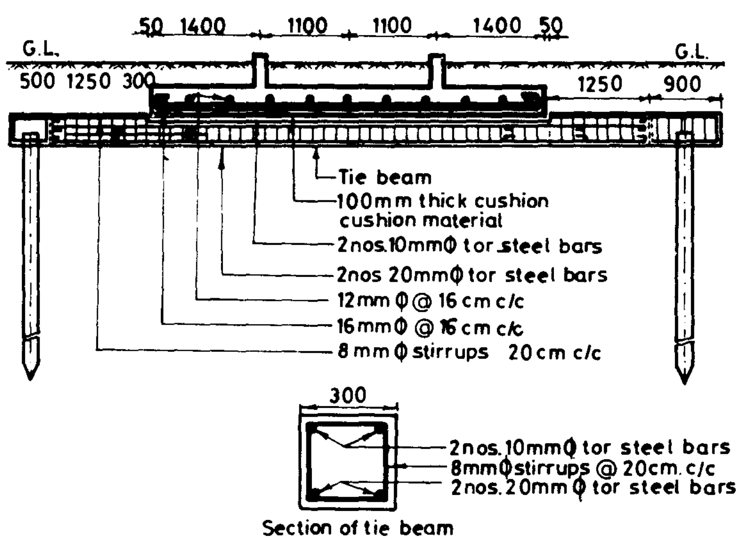


Fig.47 DETAILS SHOWING TIE BEAM

RCC RAFT ON GRANULAR PILES :

The reinforced cement concrete raft was placed over the flexible pad consisting of boulders/gravel and sand layers having an overall thickness of 65 cms. The details of a proposed flexible pad and the RCC raft is shown in Fig. 44. The thickness of lthe footing (65 cm) was kept for the heavily loaded portion to sustain load intensity of 12.5 Kg/cm^2 on each pedestal having a width of 30 cms on a 2.5 m wide footing. The intensity of loading on the lightly loaded footing having a width of 5 m is reduced to 0.48 Kg/cm^2 and the thickness of the footing was reduced to 35 cms. The levels of the pedestal in lightly loaded and heavily loaded footing were kept same. The details of the footing are shown in Fig. 45.

RCC EDGE BEAM AND TIMBER PILE SKIRTING

To achieve the benefit of confinement, the heavily loaded footing KLMN and ligthly loaded footing IJUT (Fig. 43) were surrounded by a rigid skirt 4 m deep, consisting of contiguously driven wooden piles, 20 cm in diameter. These piles were jointed at the top by a reinforced cement concret edge beam, where the various dimension of the edge beam have also been shown (Fig. 46). A tie beam (38 cm x 30 cm) was also provided between I&J (Fig. 47) to check against splaying of the edge beam, NI and JM. The various other details such as sectional elevation of heavily loaded raft (Fig. 45) edge beam, connecting the top of the timber piles (Fig. 48) and cellar pit (Fig 49) have also been presented.

INSTALLATION TECHNIQUE AND INSITU LOAD TESTS :

Although there are many techniques such as vibroflotation, vibroreplacement (Engelhardt and Kirsch 1977), vibro composer (Aboshi and Suematsu 1985), soil vibratory stabilization and rammed stone column nethod, (Datye and Nagaraju 1985) are available and being used for the construction of stone columns. Most of these method call for partial or full mechanisation requiring special equipments, trained personnel and are time consuming. In this project, a simple method known as "Simple Auger Boring Method" using indigenous knowhow (Rao, Bahandari & Sharma 1979; Rao, 1982 and Ranjan and Rao, 1983) was recommended for the granular pile installation to treat the weak sub soil underthe (F-400-4DH) Rumanian Rig foundation.

Thus with a view to demonstrate the construction procedure on full scale granular piles, 40 cm diameter 9 m deep in single and group of 2 piles at a spacing of 1.35 m c/c, were installed successfully in clayey silt deposit under high water table following the simble auger boring method (Rao et al 1979).The pile group was provided with a RCC pile cap 1.75 m x 0.4 m in size and 0.40 m depth and was skrited collectively using contiguous 20 cm diameter and 4 m deep around the RCC pile cap. The edge of the timber pile skirting was connected through a rcc beam 40 cm x 40 cm These prototpye foundations were subjected to vertical compressive load equal to three time the design load to verify the validity of design assumptions.The test results have been presented in Fig. 50.

VALIDITY OF THE DESIGN ASSUMPTIONS :

Study of curve (Fig. 50) clearly indicate that the settlement of the foundation even at an intensity of 40 t/m^2 does not exceed 26 mm and under the design stress of 10 t/m^2 it is found to be almost negligible (4-5 mm). The results therefore suggest that the design assumptions and the predictions based on analytical procedure developed were found to be in full agreement with the full scale prototype tests. Based in this it was recommended that the RCC supporting the (F-400-4DH)Rumanian Rig will not settle more than 25 mm.

SETTLEMENT OF SKIRTED GRANUALR PILES

(A) EQUIVALENT COEFFICIENT OF VOLUME COMPRESSIBILITY APPROACH :

For the computation of the settlement of the raft (7.5 m x 8.1 m) placed on 62 granular piles, collectively skirted by 4 m deep 200 mm diameter driven contiguously around was computed by equivalent coefficient of Volume Compressibility Approach (Rao, 1982; Rao & Ranjan, 1988).

Input data:

Soil Modulus E_s above 8 m = 450 t/m^2 soil modulus E_s below 8 m and up to 16 m = 1000 t/m^2. Installed pile diameter =0.55 m, Area of the raft *(A)* = 63.82 m . Hence replacement factor (α) = 0.1676 and (1 - α) = 0.8323, Equivalent modulus of composite mass = 877.335 t/m^2. Therefore equivalent coefficient of volume compressibility = 1.1398 x 10^{-3} m^2/tonne. Substituting appropriate values for different parameters and utilizing Eq. 10 through 13 and Fig. 51 the settlement of the raft was calculated in two stages.

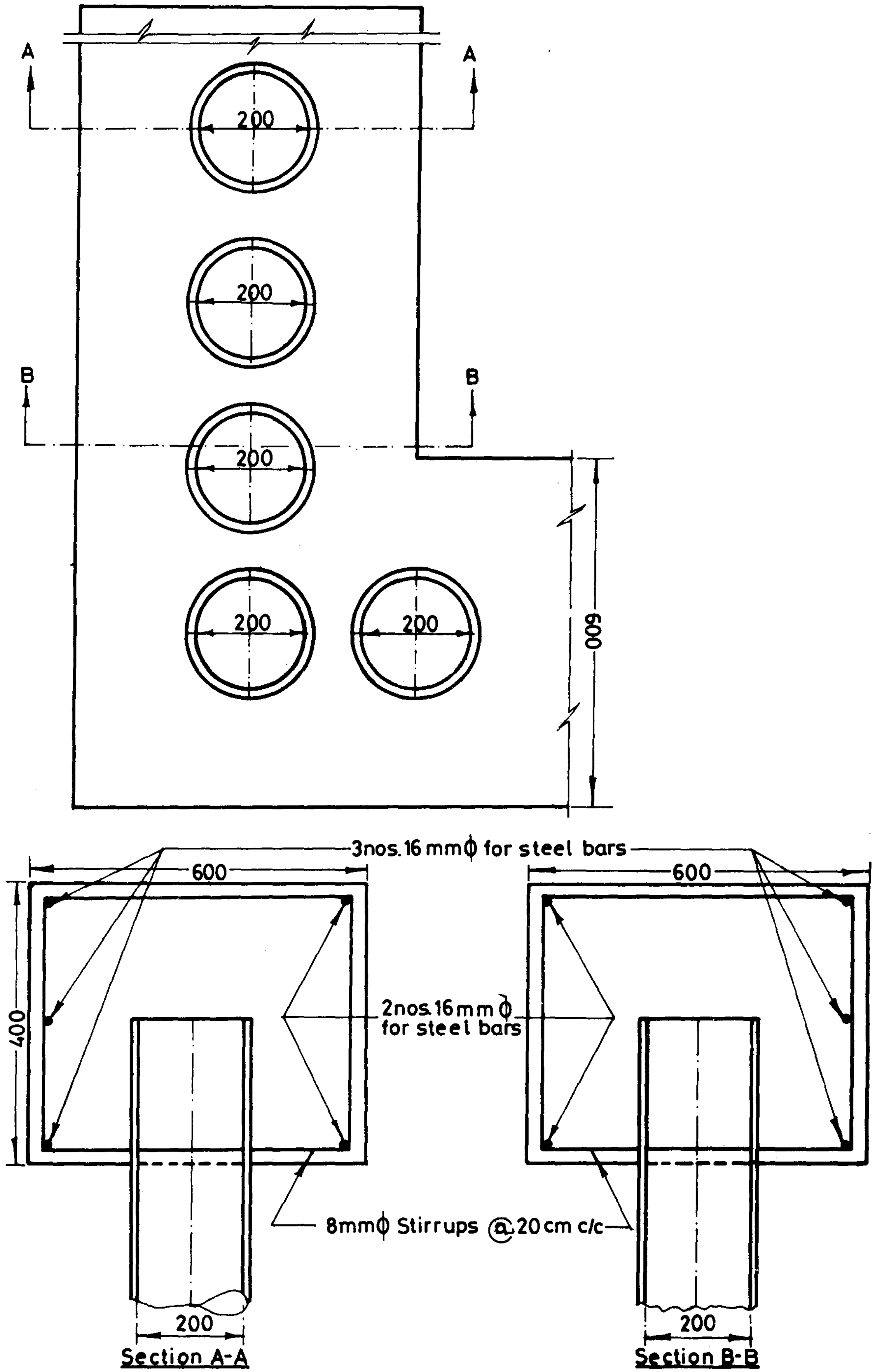


Fig.48 DETAILS OF TIMBER PILES / PIPE PILE SKIRTING

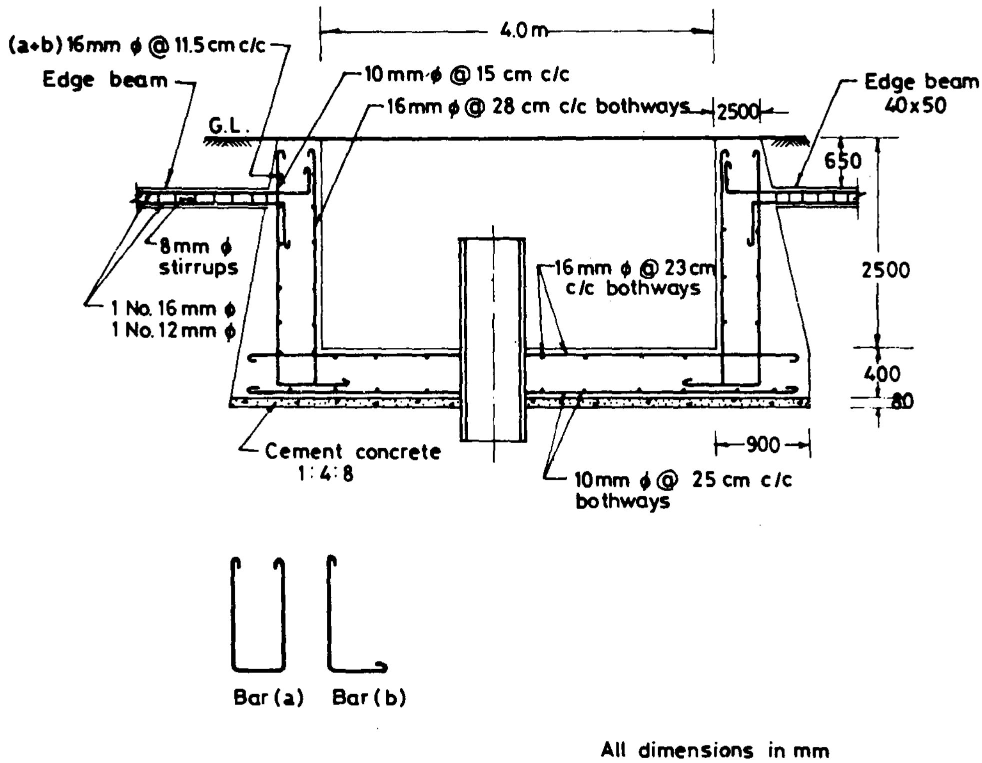


Fig. 49 Details showing cellar pit and edge beam
(section Z-Z refer fig. 43)

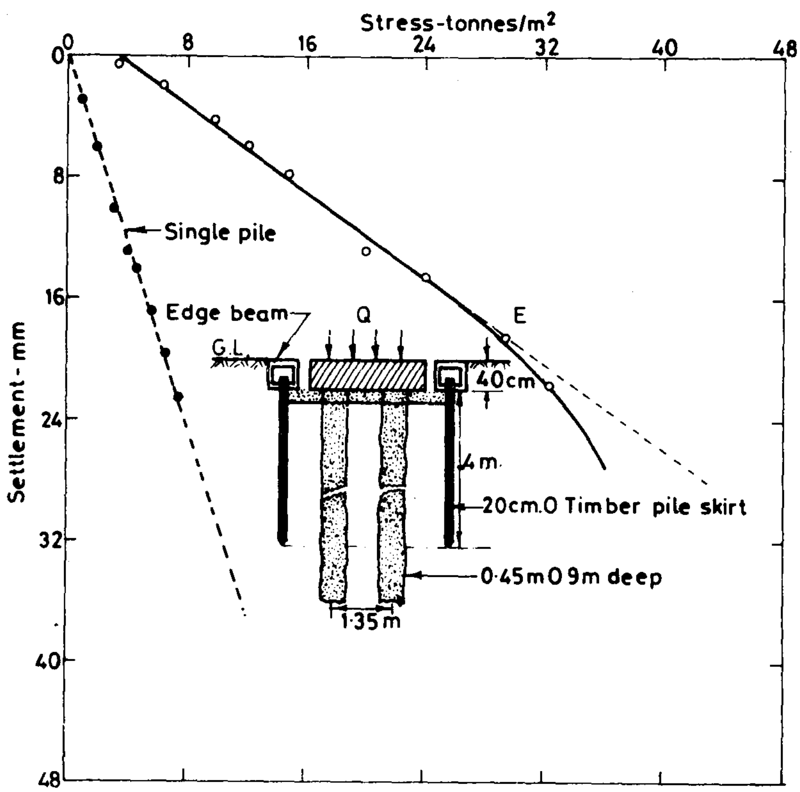


Fig.50 STRESS-DEFORMATION BEHAVIOUR OF TWO GRANULAR PILE GROUP USING TIMBER PILE SKIRTING

(a) Settlement of Reinforced Soil Plug

Settlement of the soil plug reinforced with granular piles having a thickness of composite soil layer equal to 4 m using load dispersion from the raft edge one meter below ground level (Fig. 51) was predicted

Thus

$\Delta S' = 6.39 \times 4 \times 1000 \times 1.1398 \times 10^{-3} = 29.13$ mm

Using a reduction factor of 0.25 due to timber skirting alround the raft

$\Delta S = \Delta S'\ RF = 29.13 \times 0.25 = 7.28$ mm

(b) Settlement of reinforced strata

Settlement of the strata reinforced with granular piles

In this part the raft was assumed to be placed at 5 m below the ground level and applied load was dispersed by 2:1 from the raft edge, (Fig. 51)and the subsoil was divided in three layers, each 4 m thick.The settlement in layers I and II were found from Eq. 9 to 11 and 13 to be 33 and 9 mm respectively, and in layer III it was 7.88 mm.Thus the total predicated settlement was 57 mm only, which is found to be with permissible limit in accordance with IS : 1904-1978. Since the settlement unintreated soil was found earlier = 33.95 cm.

Hence the reduction in settlement = 33.93 -5.70 = 28.25 cm

Hence settlement reduction ratio $\beta = \dfrac{28.25}{33.95} = 83.2\%$

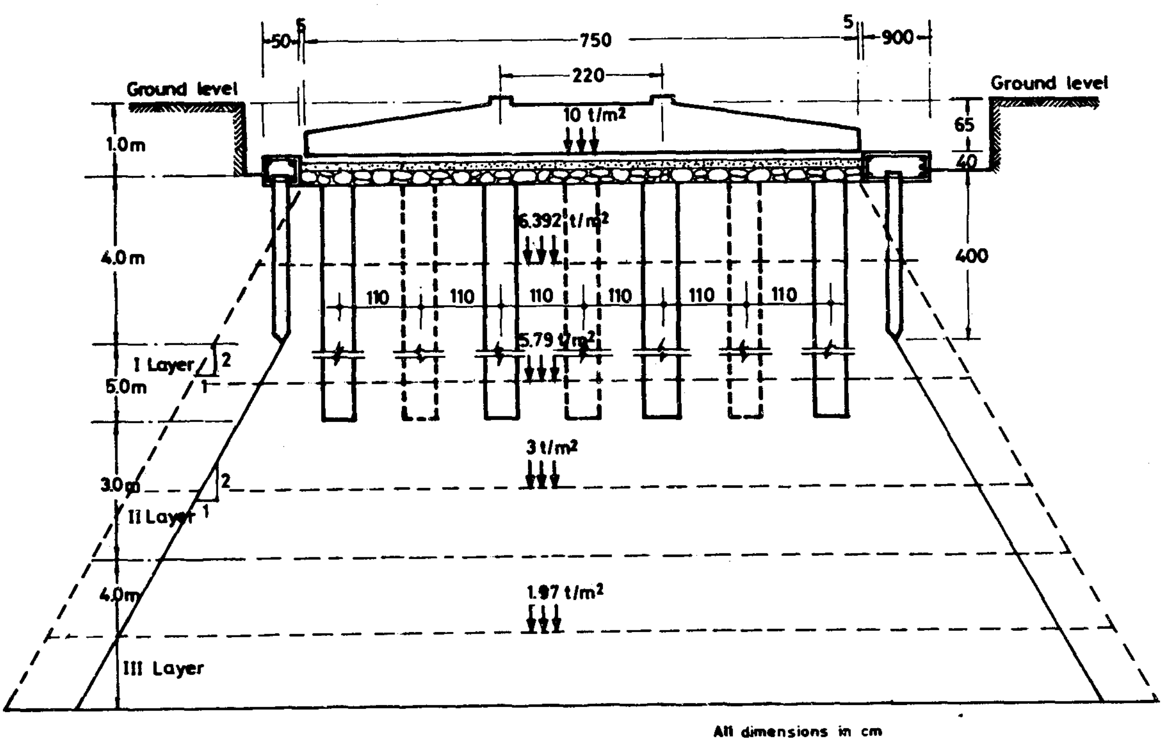


Fig. 51 Computation of settlement of skirted granular pile foundation

(B) EMPIRICAL METHODS

Computation of Settlement of Reinforced Ground

As stated earlier, till the begining of eightees no reliable method was available for computing the settlement of reinforced cohesive deposits. Based on compatibility of vertical strains,betrween the granular piles and the ambient clay, though Hughes and Withers (1974) empirical approach provided an indication of vertical ground deformation which is true only when the stress-strain characteristics of the composite soil mass is known. On the other hand the vertical deformation of top pf granular piles within the working range of stresses, were half of the radial strains in the pile which itself were very small since considerable radial strains had already been completed during pile installation. Although the reponse of ground reinforced with granular piles was understood qualitatively, the complexity of soil-pile interaction did not pernmit a simple solution (Thorburn, 1975). Based on extensive field tests, it was suggested that within the working range of stresses, a total settlement from 5 mm to 9 mm in the cohesive soil reinforced with granular piles may be expected. For computing the total settlement, a value of 5 mm - 9mm be added to athe settlement of the untreated virgin clay beneath the granular piles.Still

TABLE 9
Computed Settlements (Empirical Methods)and Equivalent Coefficient of Volume Compressibility Approach

Source	Empirical Equations	Settlement (mm)	Remarks	
Kezdi (1967)	$S=0.36\ p\ B/E_s$	45.67	p	: applied design stress
Schmertmann	$S=0.6(p-p_{o)}E/E_s$	64.84	p_o	: Overburden stress at foundation level
Thorburn (1975)		25.88		
Trofimenkov	$S=0.12\ pm\ B/E_s$	26.47	p_m	: Effective applied stress at foundation level
Greenwood (1970)	5% of untreated soil	34.38		
Rao (1982), Rao and Ranjan (1985)	$\sum_{i=1}^{n} q_i\ m_{veqi}\ h_i$	57.0	m_{yeqi}	: Equivalent coefficient of volume compressibility

another empirical simple chart proposed by (Greenwood, 1970), relating to pile spacing and settlement reduction ratio β for different undrained shear strength values for 20 and 40 KN/m^2, provides an underestimation of settlement.

In view of the above limited approches the empirical methods, proposed by Kezdi (1957), Schmertmann (1970) and Trofimenkoov (1979) for piled mat foundations designed to support heavy buildings and other structures were used to predict the settlement of cohesive soils reinforced with granular piles with their modified elastic soil modulii.

COMPUTATION OF SETTLEMENT- EMPIRICAL METHODS

INPUT DATA

Undrained shear strength C_u =0.48 Kg/cm^2 , appiled stress P = 1 Kg/cm^2, overburden stress one meter below ground level p_o = 0.19 Kg/cm^2 effective net stress at cut off level = 0.81 Kg/cm^2, width of the raft = 750 cm the weighted modulus of deformation E_s = 76 Kg/cm^2 and the E_s value beyond 9 m depth and upto 16 m = 100 Kg/cm^2.

Utilising appropriate values for different parameters the computed settlement from and modified empirical approaches are presented in Table 9. The computed settlement, from these empirical approaches includes the settlement of untreated virgin clay beneath the pile toe equal to 9 mm in layer 2 and 7.88 mm in layer 3 making a total of 16.88 mm.

Study of the Table 9 indicate that the total settlement under the raft from different method, placed over the reinforced cohesive soil varies from 26.5 mm to 65 mm which is found to be well within permissible limits (IS : 1904-1974). Further the predicted settlement is expected to be completed during the period of construction including the erection of the rig. Thus almost no settlement was anticipated during service period.

DESIGN SEPCIFICATIONS :

1. Granular piles having 55 cm installed pile diameter, 9 m deep below cut off level (one meter below natural ground level) shall be installed at a spacing of three times the pile diameter (1.35 m) in a triangular patten (Fig. 43).
2. Crushed stone aggregates (20 mm -70 mm) shall be used for the construction of piles in layers of 30 cm with 22-25 per cent of clean locally available sand and compacted with a hammer of 200 Kg with a height of fall equal to 75 cm in accordance with the simple auger boring method.
3. The bore hole sides, shall be protected during construction by a mild steel casing or alternatively 5 % sodium bentonite slurry shall be used to fill the bore hole. The charging of the bore hole shall be carried out by tremie method.

4. The installation of piles shall be carried out from inside towards outside. Number of piles under each raft shall be 62 and further 52 piles shall be installed around both the raft outside the skirt thus making a total of 176 piles.
5. Improvement of the clay strength between the piles shall be checked near the pile either by static or dynamic cone penetration tests.
6. The group of 124 piles shall be skirted using timber piles, 4 m deep and 20 cm in diameter driven contiguously and a rcc edge beam shall be provided in accordance with Fig. 46 joining the heads of piles together.
7. The installation of a full size pile 55 cm in diameter and 9 m deep shall be demonstrated to increase confidence and also a group of 2 piles collectively skirted with contiguous timber piles skirting with edge beam shall be installed for in-situ load test at least upto three times the design load.

RECOMMENDATIONS

1. The settelment of the proposed skirted granular pile foundation for (F-400-4DH) Rumanian rig, under a working load ot 10 t/m^2 is not expected to exceed 25 mm during the service period.
2. The improvement in bearing capacity has been found to be 272% over the virgin subsoil and hence the ground treatment by granular piles was found to be effective and economical.
3. The settlement of the rig foundation shall be monitored during the opeartion of the rig.

FEEDBACK STUDIES

In accordance with the design specifications and recommendations provided to the client in August, 1980, the installation of prototype piles were demonstrated to the Engineers of the department and in-situ load test on single and group of two piles (collectively skirted) were carried out successfully in May, 1982 under a maximum load equal to 65 tonnes. (Fig. 50).

Gupta and Dev(1988)in their paper have reported that under a load intensity of 3.8 Kg/cm^2 the corresponding settlement did not exceed 29.6 mm and also under a design intensity of 1.2 Kg/cm^2, it was found to be within 6 mm only (Fig. 50). Hence, the factor of safety equal to 3 was achieved even without reaching the ultimate load of the pile group.

Again on single pile the settlement was not more than 2-3 mm for a load of 6.28 Kg/cm2. While concluding Gupta & Dev (1988) have reported that :

- the full scale prototype test results lead to a great technical success and the design recommendations were fully confirmed at site skirted through in-situ load tests on full size granular piles.
- During the erection of the Rumanian rig and later during service period of one year, the settlements were monitored periodically. The observed settlements under the raft were found to be within 5 mm only.
- Based on the above, Gupta & Dev (1988), concluded that these observations are found to be mile-stones in the success history of skirted granular pile foundation under heavy loads. The technique was found to be both cost effective and speedy.

PERFORMANCE OF THE FOUNDATION OF AN UNDERGROUND POWER HOUSE ON IMPROVED GROUND

GENERAL

The excavation of the pit for the construction of an underground power house had already been started and reached the level of the rcc raft foundation EL 246.65 m. whereas the natural soil level (NSL) was at EL 269.28 m. During the subsequent monsoons, the pit had got fully submerged. The area around the underground power house site was fertile and cultivated lands having a plain topography (Fig. 52). The hills were far away on the northern side and a river was running on the eastern side. Also a natural drain was running near by which provided drainage of the area around with its outfall into the river. Geologically the site of the power house was free from any recent deposit. The case study provides the details for the subsoil treatment, analysis and construction technique adapted for supporting the high design load through a rigid rcc raft 36 m x 22 m x 3.6 m in size, proposed to be constructed at 22.63 m below NSL.

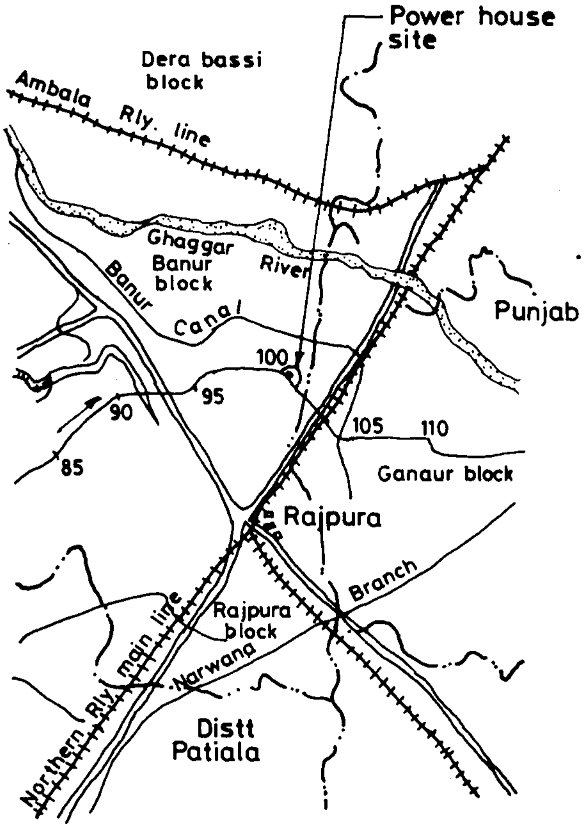


Fig. 52 Site plan

WORK PLAN

The work plan was :

(a) to suggest a cost effective and efficient ground treatment method to support rigid raft without undergoing shear failure and satisfy stringent settlement criteria.

(b) Analysis and interpretation of the subsoil properties to arrive at a rational and speedy ground treatment method which could be used effectively below ground (NSL) at a depth of 22.63 m under water.

(c) To advise and occasionally supervise the method of subsc il treatment.

(d) to verify the design assumptions through a full scale load test on a rcc rigid footing after the treatment.

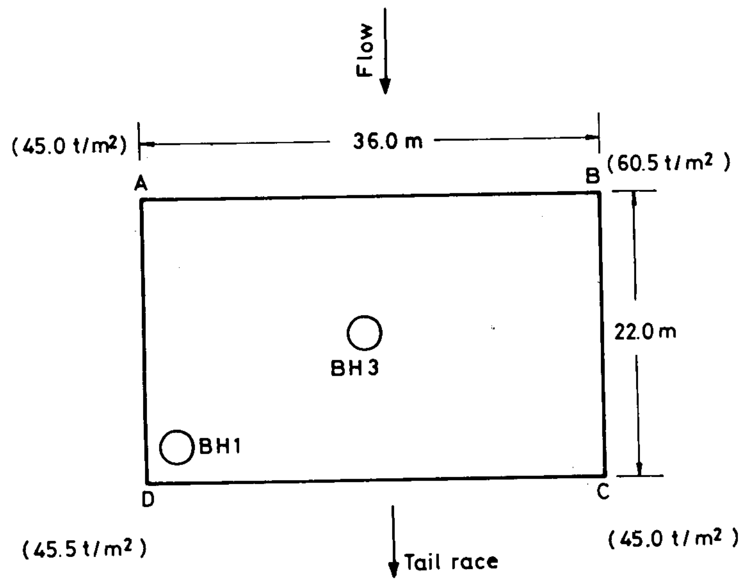
(e) Occasional supervision during actual ground treatment below the rigid raft.

SUB-SOIL INVESTIGATION AND ANALYSIS

The sub-soil investigations of the site mainly consisted of two bore holes as per locations shown in Fig. 53.

The details of the bore log, the N_s values and sub-soil classification for the two bore holes are given in Fig. 54. The bore logs indicate that the sub-soil upto El.235.65 level i.e. about 33.5 m consists of silty clay deposit classified as (CL) in accordance with IS : 1498-1976, Standard penetration test results indicate that the avercge N_s value from 23 m depth is 10 increasing to 20 at about 50 m depth indicating an average value of 15 (Fig. 54). The corresponding shear parameters such as angle of internal friction, ϕ varying from 10° to 15° and cohesion, c as 0.5 kg/cm2 (5.0 t/m²).

Later, some additional field tests such as boring and standard penetration tests, plate load and dynamic cone penetration tests were carried out. The location of these tests with respect to the foundation raft are given in Fig. 55. The data obtained from dynamic cone penetration tests and standard penetration tests, is shown in Fig. 56. The N_s values have also been tabulated in Table 10.



Natural ground level = E.L 269.28 m
Raft foundation level = E.L 246.65 m
Cobblepack level = E.L 245.65 m

Fig. 53 Layout of foundation raft with location of bore holes & design intensities

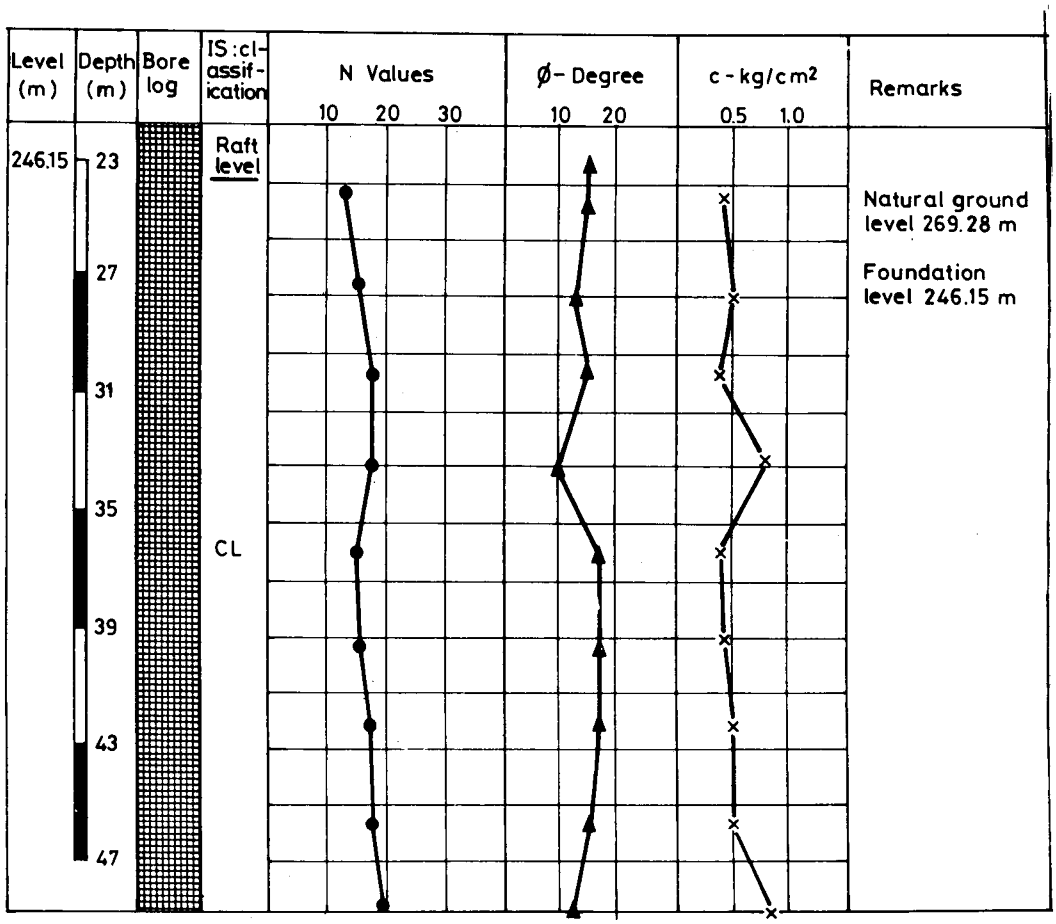


Fig. 54(a) Bore log & sub-soil characteristics from, bore hole - 1

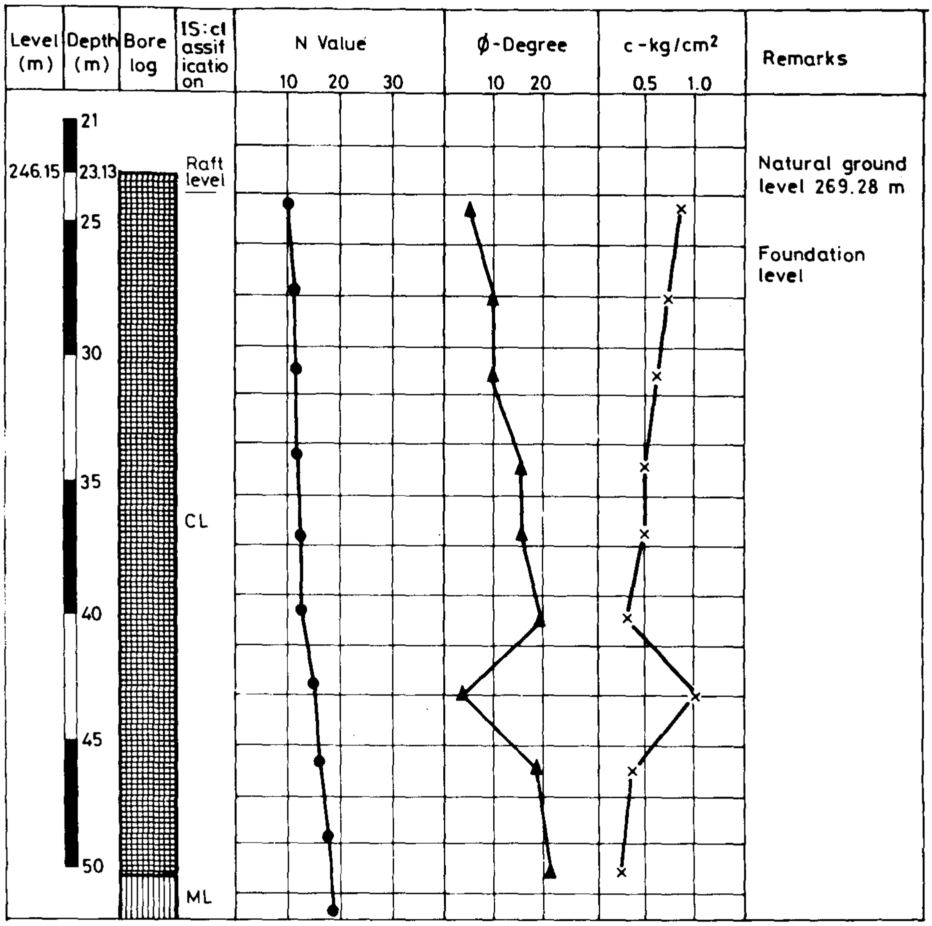


Fig. 54(b) Bore log & sub-soil characteristics from bore hole-2

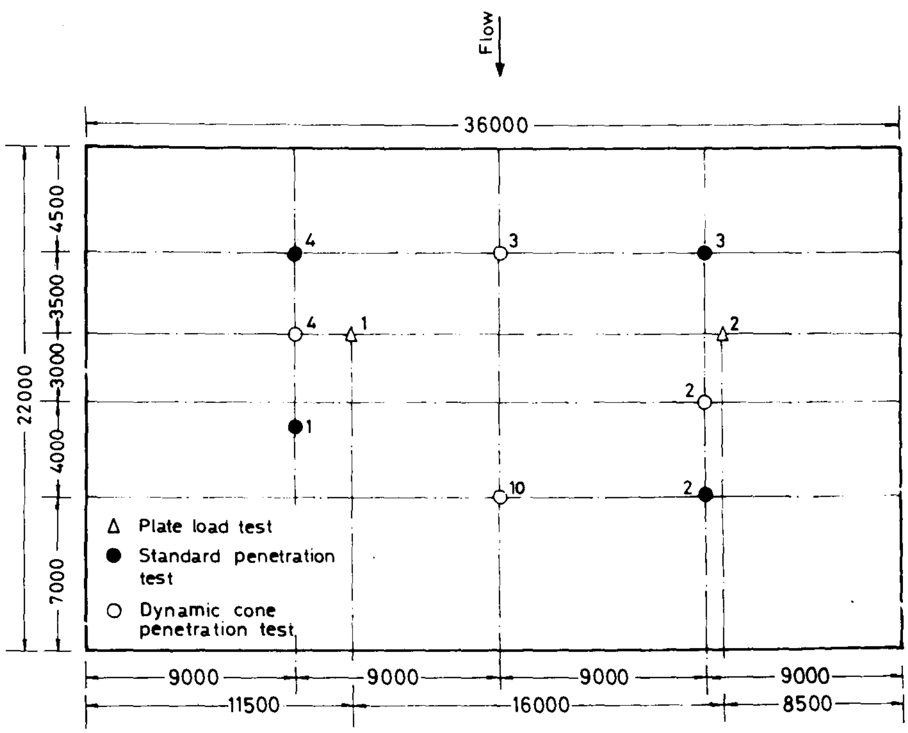


Fig.55 Additional test locations

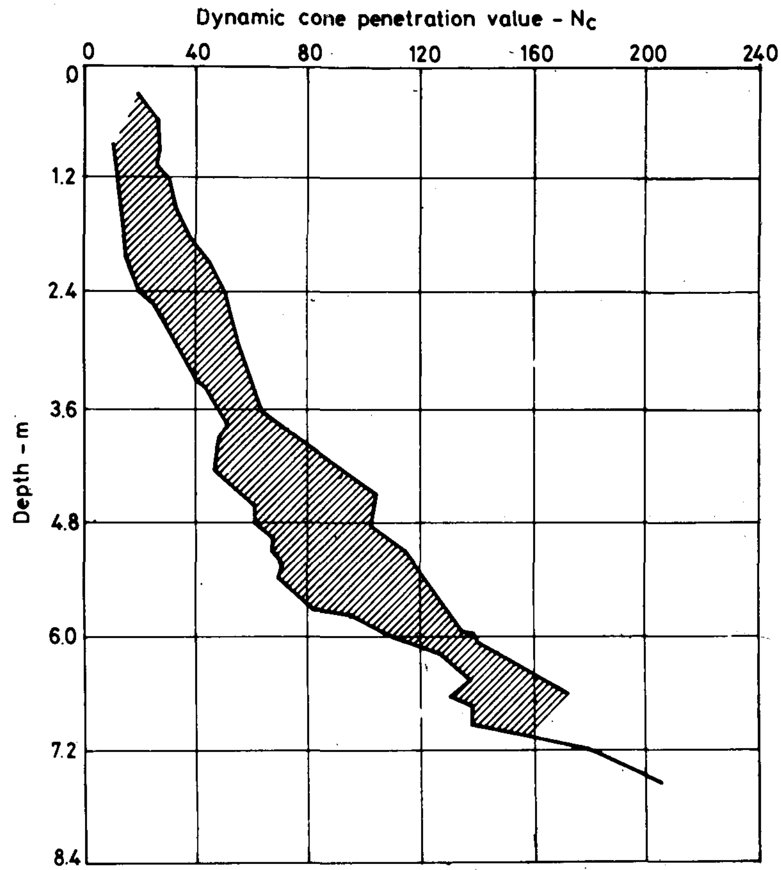


Fig. 56 (a) Dynamic cone penetration test value vs depth (Gep-gr - 208 C)

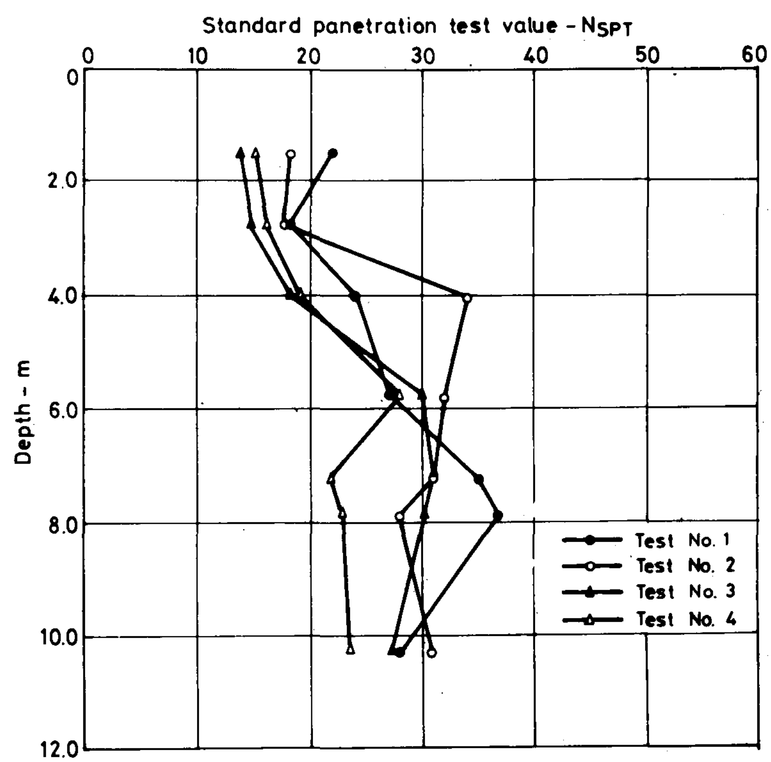


Fig. 56(b) Standard penetration test value vs depth

TABLE 10

N_{SPT} OBSERVED WITH DEPTH

S. No.	Depth	N_{SPT}	Observed			Av. N_{SPT}
1.	0					
2.	1.50	22	16	14	15	17.25
3.	2.75	18	18	15	17	17.00
4.	4.00	24	34	19	19	24.00
5.	5.75	27	32	30	28	29.25
6.	7.25	35	31	31	22	29.75
7.	7.75	37	28	32	23	30.00
8.	10.25	28	31	28	24	27.75

A glance of Fig. 56(b) indicates that the sub-soil within the foundation raft zone is practically uniform though the penetration resistance continuously increases with depth down to 10 m.

The N values (Table 10) suggest that the sub-soil between the base raft level (El.246.650 m) and upto 22.0 m (i.e. the width of base raft) could be considered to be made up of four layers (Table 11), the first layer 3.0 m thick with N_s of 17, the second layer 1.0 m thick with N_{SPT} of 24, and the third layer 6 m thick of N_s 29 and below 10 m depth the fourth layer. These four layers were found to be of stiff consistency and the unconfined compressive strength obtained from NSPT value was ranged from 22 t/m² to 40 t/m² (Table 11). The water table was found to be 9.0 m below NSL. The fourth layer is very stiff, since during the installation of test piles, also it was observed that the rate of penetration of bore hole was very slow confirming stiff soil with higher NSPT values.

In view of N_s values the elastic soil modulus, E_s values adapted in design have been shown in Table 11.

TABLE 11

SOIL LAYERS FOR ANALYSIS

Level (m)		Layer No.	Layer thickness (m)	Av. N_{SPT}	Soil modulus E_s (t/m²)
E1.	245.650*				
		1	3.0	17	2500
E1.	242.650				
		2	1.0	24	3000
E1.	241.650				
		3	6.0	29	3500
E1.	235.615				
		4	12.0	30	4000
E1.	223.650				

*Bottom of proposed cobble pack level (the proposed raft level being E1. 246.650 m).

DESIGN LOAD INTENSITIES

The raft for the power house was 22 m x 36 m in plan located at E1 246.650 m level i.e. at a depth of 22.63 m below the natural soil level of E1 269.28 m. The raft was considered as rigid being 3.0 m thick. The maximum design load intensity was 60.5 t/m². Later, these design load intensities were revised, giving an average of 39.3 t/m². These design intensities have been shown in Fig. 53. Further, the summary of stresses at four corners was provided which gave an average stress of 39.3 t/m².

Since the raft was proposed to be located at a level of E1 246.650 m i.e. 22.63 m below the natural soil level and the excavation of the pit for the raft (22 m x 36 m x 3 m) had already been completed, (Fig. 57) the advantage of relief of stress due to removal of over burden was taken and the settlement of raft was computed for the net intensities of pressure.

Since the water table below NSL was at a depth of 9 m and bulk unit weight γ_{bulk} was 1.5 t/m³. Hence the net intensity of load below raft level was found to be (60.5-27.83) = 32.67 t/m² since the effective overburden stress at 22.63 m below NSL was found to be 27.83 t/m². It may be noted that maximum intensity of stress as worked out in stability analysis should be adapted as total bearing load for settlement considerations which ultimately governs.

ULTIMATE BEARING CAPACITY OF VIRGIN SOIL :

Utilizing in-situ cohesion Cu = 5 t/m² from Fig. 54, depth Df = 22.63 m, submerged unit weight γ_{sub} = 1 t/m³, the ultimate bearing capacity was found from Eq. 48 as :

$$q_u = 1.2 \times 5.7\, C_u + \gamma_{sub}\, D_f \qquad ...(48)$$

$$\text{or} \quad q_{safe} = \frac{12 \times 5.7 \times 5}{Fs = 3} + 1 \times 22.83 = 34.03 \ t/m^2$$

Say 34.0 t/m²

Hence safe bearing capacity was found to be 34 t/m².

It may, therefore, be noted that the average design intensity of stress is equal to 37.3 t/m² which is higher than the safe bearing capacity. Also the requirement of stringent settlement criteria (3 mm per 10 m height of shaft)

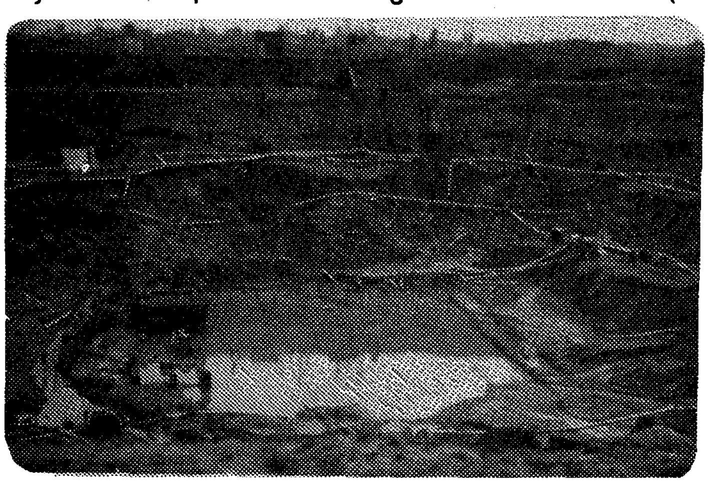


FIG 57. A VIEW OF THE EXCAVATED PIT FOR THE UNDERGROUND POWER HOUSE ALONGWITH THE LOADING PLATEFORM FOR IN-SITU LOAD TEST

demanded the need for ground improvement of the sub-soil strata below the raft level. Thus the improvement required was 60.5/34 = 1.78 times.

CHOICE FOR THE METHOD OF GROUND TREATMENT

There are various methods of ground treatmens such as drains with and without preload,replacement and stabilization, soil reinforcement (vibro replacement) also reinforcement by granular piles, compaction by falling of heavy weight and grouting etc. as summarized in the begining in Fig. 2. However, the choice of a particular method depends on many factors as indicated in the introduction part of the lecture besides the various constraint associated with the particular project. In the present assignment these constraints were *(a)* the depth of the raft was very large (22.63 m), *(b)* under water table construction, *(c)* limited, short time available for treatment,*(d)*demand for fulfilling stringent settlement criteria and finally *(e)* the cost of treatment and the site location keeping in view the various constraints a number of ground treatment methods were examined. Finally a group of skirted granular piles was favoured in view of the experience gained in the past, the high design intensity, satisfying required settlement criteria and the time constraint besides the speed of installattion, and also because large number or rigs could simultaneously be deployed at a time even when the depth of foundation was large (Fig. 58), to overcome the time constraint.

DESIGN AND ANALYSIS OF GRANULAR PILE SYSTEM

INPUT DATA

The soil properties used in the analysis were (a) In-situ cohesion, $C_u = 5.0$ t/m², obtained from vane test in the field and also from unconfined compression test in laboratory (Fig. 54), (b) Angle of shearing resistance = 15°(ignored), (c)Thickness of raft = 3.00 m (have been accounted for), (d)Effective (submerged) unit weight of soil =1.0 t/m², (e) Diameter of the bore hole for granular pile= 45 cm (f) Modulus of pile material ; $E_p = 5000$ t/m² (Bowles, 1977; Rao, 1982), (g) Soil modulus and layer thickness for settlement analysis as per Table 11.

GRANULAR PILE CAPACITY AND PILE ARRANGEMENT

The granular piles be installed by charging the bore hole with well graded crushed stone layer and sand

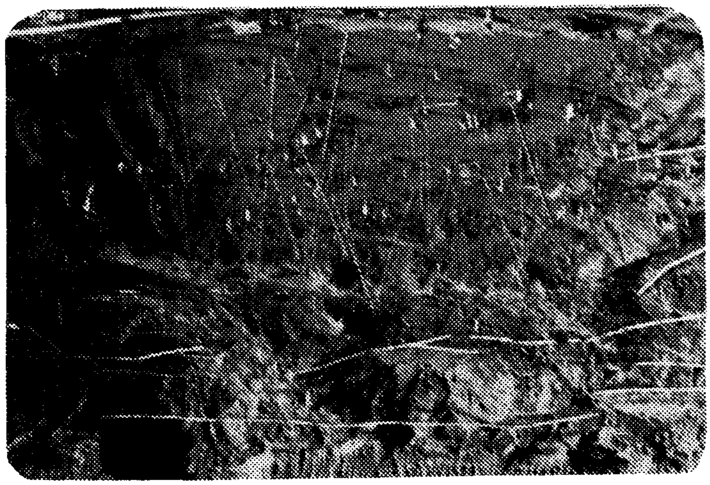


FIG 58. INSTALLATION OF GRANULAR PILES AT THE BASE OF THE RCC RAFT

layer and compacting it with an internal operating hammer. The repeated compaction of charged bore hole results in increase in diameter. Based on research and experience (Rao, 1982; Rao & Ranjan, 1983; Ranjan & Rao, 1987) 20 per cent increase in the pile diameter due to compaction, the installed pile diameter was 1.20 x 45 i.e. say 55 cm with a cross-sectional area of 0.238 m².

Recognising the contribution of the load shared by the ambient clay (Rao, 1982; Ranjan and Rao, 1986) the ultimate bearing capacity of a single pile was estimated from Eqs.1, 6 and 8

According to Rao (1982) the load shared by the surrounding soil, q_s and pile q_p vary as $q_p : q_s = 12.5 : 1$.

Since the maximum design load was 60.5 t/m² then q_s was found as to be 4.48 t/m², q_p = 56.02 t/m².

Substituting, $K = 6$, $C_u = 5.0$ t/m², $q_s = 4.48$ t/m², $\gamma_{sub} = 1.0$ t/m³, d=0.55 m, the ultimate capacity of single pile worked out to 79.76 tonnes. With a factor of satety equal to 3, the safe bearing capacity of a single pile Q_s was found as 26.58 tonnes.

As stated earlier assuiming water table at 9.0 m below NSL, the unit weight of soil above water table as 1.5 t/m², the overburden stress at foundation level of raft was 27.83 t/m² and the net intensity was 32.67 t/m²

Hence total load on soil = 32.67 x 36x 22 = 25874.64 tonnes.

$$\text{Thus, the number of piles required} = \frac{25874.64}{26.58} = 973.46 \text{ say } 974 \text{ piles}$$

Based on these considerations alone 974 granular piles were needed. However, it becomes necessary to provide larger number of piles to keep settlement within the specified limits. Thus adopting a pile spacing of 0.90 m centre to centre with piles arranged in a zig-zig pattern (Fig.59) total 1204 number of piles were provided.

SETTLEMENT ANALYSIS

The total settlement, S of improved ground reinforced with partially penetrating granular piles (Fig. 60) was computed from Eqs. 9 t0 11 and 13. (Rao, 1982; Rao and Ranjan, 1985 and 1988).

In view of the raft being rigid (3 m thick) the computations for settlement were made taking the average pressure on the raft.

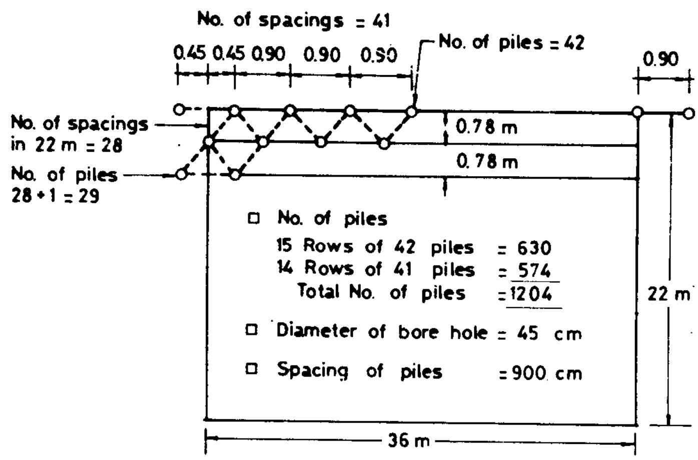
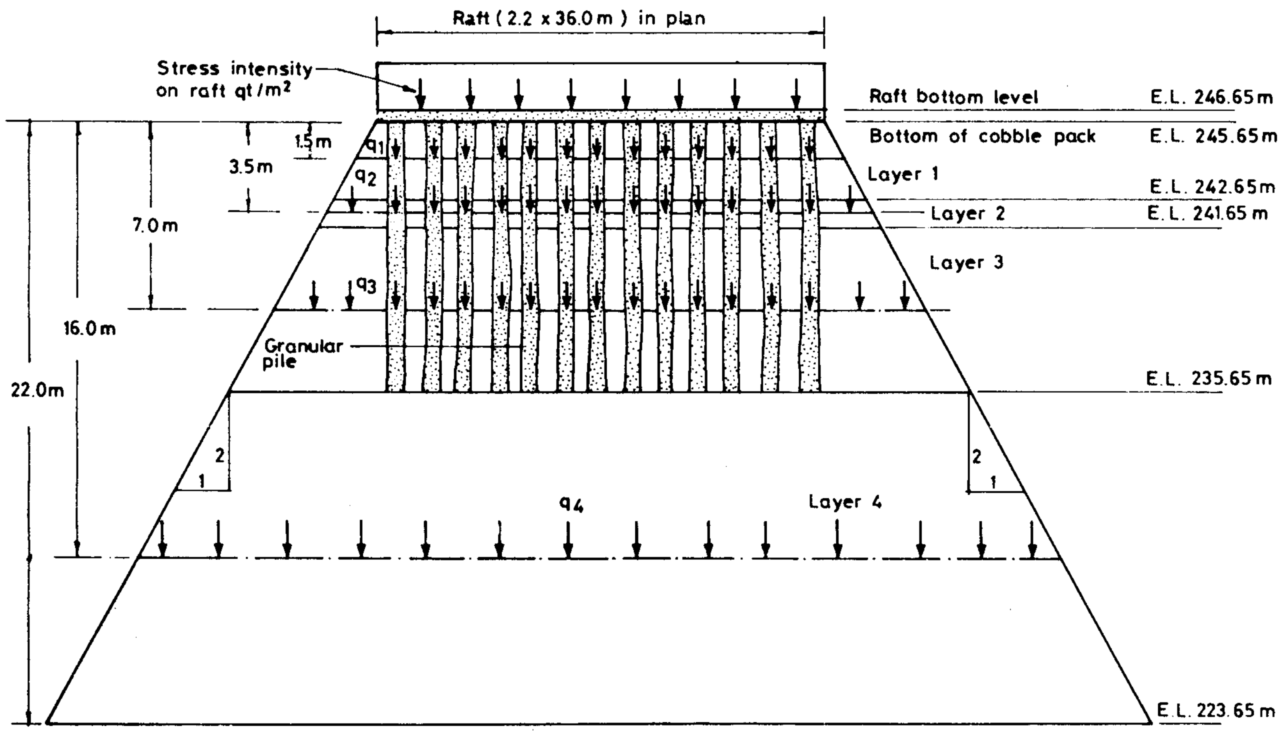


Fig. 59 Arrangement of piles



Intensity on raft - q t/m²
Intensity on centre of layer 1:2:3:4 being q_1 q_2 q_3 q_4 respectively
Length of granular pile varied from 9.0 m to 16.0 m below raft level (table - 3)

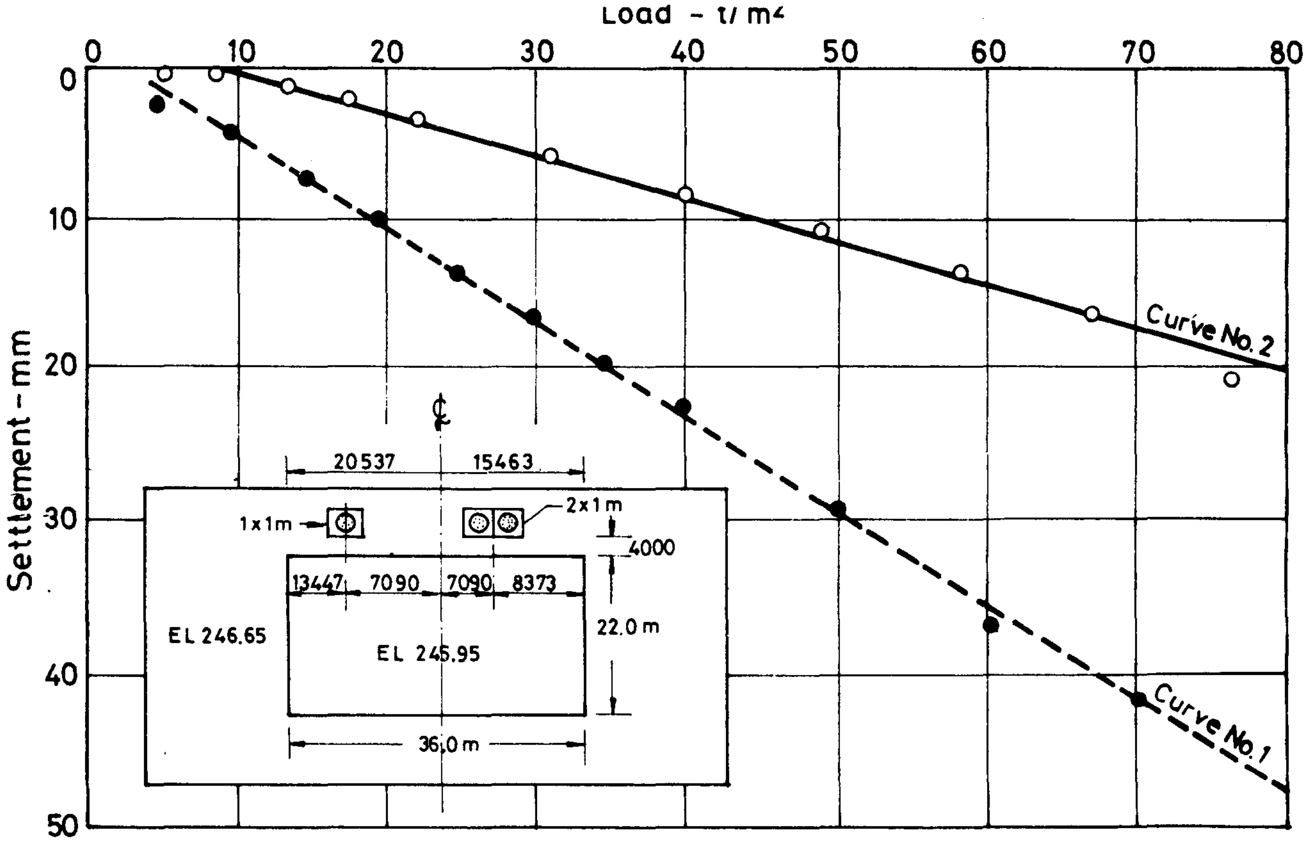
Fig. 60 Sub-soil strata for settlement computations
(Gep-gr-208 C)

The equivalent coefficient of volume compressibility, m_{veq} of the composite mass was computed from Eq. 19 (Rao & Ranjan, 1985, 1988).

As discussed earlier since the stiffiness of soil varied, the values of soil modulus vary accordingly. Further, computations were made with varying length of granular piles from 10 m to 16 m. These computations are summarised in Table 12.

IN-SITU LOAD TESTS ON GRANULAR PILES

With a view to verify the validity of design assumption through full scale in-situ load tests a single pile, 60 cm installed pile diameter, as observed in pile test results, 11.4 m deep, and a group of two piles 60 cm. installed pile diameter and lengths 10.2 m & 10.7 were carried out. These test piles were installed in the pit by the side of the main raft (Fig. 61 & 62). Each pile was installed in accordance with Rao, 1982; Ranjan & Rao, 1983 and



Pile group	single pile	Two pile group
Pile cap area	1.0 m²	2.0 m²
Index	—●—	—○—
Design load	26.0 t	52.0 t
Settlement at design load	14.0 mm	4.0 mm
Settlement at three times design load	45.0 mm	21.0 mm

Fig. 61 Stress vs settlement of single & group of two granular piles at Rajpura site (gep-gr-208 C)

TABLE 12

THEORETICALLY COMPUTED SETTLEMENT S WITH VARYING THICKNESS REINFORCED LAYER AND STRESS INTENSITIES OF RAFT

Sl. No.	Thickness of reinforced layer (reinforced with granular pile) (m)	Stress on raft (t/m^2)	Total Settlement (mm)
1.	10.0	39.30*	123.8
2.	12.0	"	123.4
3.	14.0	"	123.0
4.	16.0	"	122.2
5.	10.0	32.67**	103.0
6.	12.0	"	102.6
7.	14.0	"	102.1
8.	16.0	"	101.6
9.	10.0	11.47***	36.2
10.	12.0	"	36.0
11.	14.0	"	35.9
12.	16.0	"	35.7

* The average stress on raft
**The net intensity on raft i.e. (60.50 - 27.83 = 32.76 t/m^2)
***The net ensity on raft i.e. (39.30 - 27.83 = 11.47 t/m^2)

Ranjan, 1987. The uniformity of compaction of the pile throughout the length was achieved by adhering to the specified set criteria. The set criteria was established by imparting 15 blows of 500 Kg hammer with 1.0 m fall. A set of less than 20 mm was fixed against the 16th blow on the charge. The same criteria was maintained for the compaction of various layers throughout the length of pile. The details, of main piles, ancillary piles, loading or reaction platform, load application and recording of settllements for each load increments have been provided else where (Rao, et al. 1989). The stress versus settlement behaviour of single and group of two granular piles as observed in full scale field tests (Rao. et al. 1989) is presented in Figs. 61 and 62.

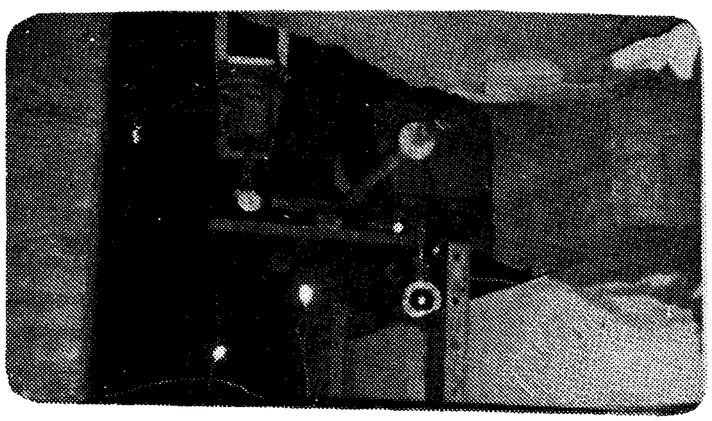


FIG 62. CLOSE UP OF IN-SITU LOAD TEST

The test results indicate that for a single pile having a pile cap of 1 m^2 area, for a safe load of 26 tonnes the settlement was 14 mm, whereas for three times the design load it is 45 mm only. However, for a 2 pile group with pile cap area of 2 m^2, the corresponding settlement for a safe load of 52 tonne was 4 mm whereas for three times the design load it is 21 mm only. Utilizing the load intensity-settlement curve of composite ground the settlement behavior of the 22 m X 36 m raft has been predicted in a separate report on test piles (Rao, et al, 1989).

SETTLEMENT OF RAFT

EQUIVALENT COEFFICIENT OF VOLUME COMPRESSIBILITY APPROACH

Utilizing composite mass characteristics, settlement computations with varying pile length and stress intensities utilizing concept of equivalent coefficient of volume compressibility (Rao, 1982; Rao and Ranjan, 1985) was made. In these computations length of granular piles has been varied from 10.0 m to 16.0 m and stress intensities from 39.30 t/m² to 11.47 t/m². The computed settlement have been presented in Table 12 using 1204 number of granular piles.

It may be noted from Table 12 that the total settlement of the raft on the soil layer reinforced with 10 m deep piles under an intensity of stress of 39.30 t/m² is found to be 123.8 mm. When the pile length was increased to 16 m the total settlement reduced to 122.2 mm i.e. a reduction in settlement of 1.6 m only for corresponding increase in pile length of 6 m. Taking the intensity of 60.50 t/m² and release in stress due to removal of overburden i.e. 27.83 t/m² the net intensity on raft is 32.67 t/m². The reduction in settlement for the intensity on 32.67 t/m² on the raft with increase in thickness of reinforced soil layer from 10.0 m to 16.0 m is only 1.4 (Table 12).

These observations indicate that under the anticipated stress intensities and the available sub-soil strata below the rigid raft, increase in pile length beyond 10.0 m (measured from the base of the cobble pack layer i.e. El. 245.65 m) did not result in any significant reduction in settlement. These observations were further substantiated by the actual observation of time required to install the 45 cm diameter bore hole during the pile installation at the site. The time required for installation of 88 No. of piles under the raft have been shown in Fig. 63 (Ranjan & Rao, 1990). The figure indicates that the average time taken for depth 0 to 9 m was about 6 hours, for 9 to 10 m is about 1 hour whereas for 10 to 11 m is about 4 hours. Thus the rate of advancement of bore hole beyond 10 m is observed to be very slow confirming that the strata is very stiff/hard. Keeping in view these facts, it was not advisable to disturb the virgin compact structure of the hard clay available beyond 10.0 m below the cobble pack level, thus it was recommended that the granular pile should go upto the stiff 4th layer. Thus the granular pile tip was maintained at El. 245.65 m - 10.0 i.e. El. 235.65 m.

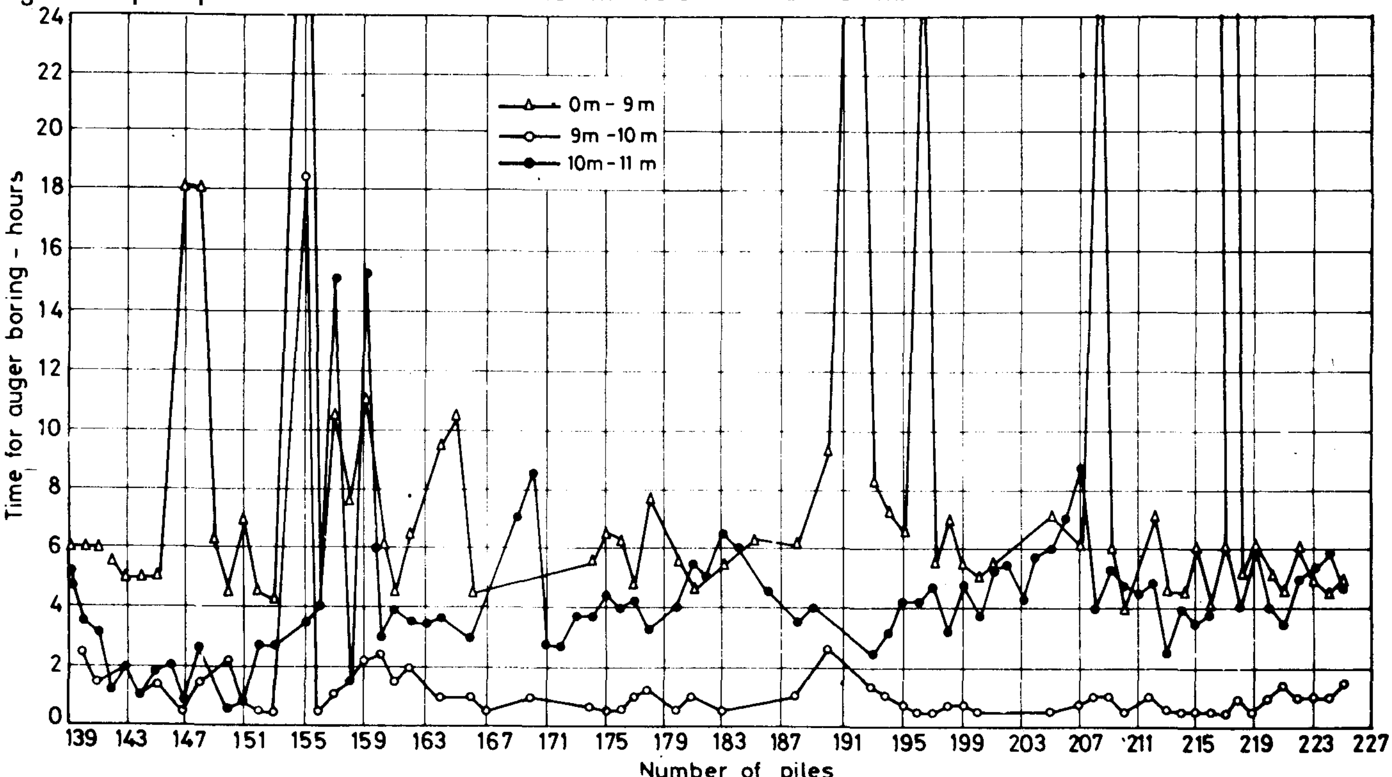


Fig. 63 Time required for installation of piles / bore hole for different depths

INFLUENCE OF CONFINEMENT

It was further observed from Table 12 that with granular piles of 10.0 m and stress intensity of 32.67 t/m² on the raft the settlement of the raft is 103.0 mm. This intensity of stress was the stress on the raft in addition to the removed overburden pressure of 27.83 t/m². Thus, 103.0 mm is the total settlement which was expected under the raft under full design load. The computations of the settlement were based on the assumption that the

TABLE 13

SEQUENCE OF LOADING ON RAFT

S.No	Load per month (tonnes)	Duration from the starting of concereting raft (months)	Total load (tonnes)	per cent of load
1.	2850	0-4.5	12,825	40.08
2.	770	4.5-9.0	3,465	10.83
3.	3370	9.0-12.0	10,110	31.59
4.	7000	12.0-20.0	5,600	17.50
		Total	32,000	100.00

raft is placed at the ground surface i.e. without alround surcharge. However, in the present case the raft was placed at about 22.63 m below the ground level which provides a confining pressure of the order of 27.83 t/m^2 around the raft. On account of the confinement provided by the alround surcharge there is no reduction in load carrying capacity of peripheral piles; besides it provides reduction in settlement. Taking conservatively 50% reduction in total settlement due to alround confinement (Rao and Ranjan, 1988) the total predicted settlement under the raft was 51.5 mm only.

SEQUENCES OF LOADING

The anticipated sequence of loading on the foundation raft is presented in Table 13. Study of Table 13 indicates that during the first about 4.5 months the anticipated load on the raft is only 40% of the total design load. Hence, the corresponding settlement under this load is anticipated to be even less than 0.40 X 51.5 i.e. 20.60 mm due to the fact that with slow rate of loading the soil stiffness will increase resulting increase in the equivalent modulus of the composite mass, E_{eq}. This will further result in significant reduction in m_{veq} value and hence the settlement. Further it is supported by the current experience that the reduction in total settlement in a virgin clay can be reduced by 70% by mere treating the ground by granular piles and the consolidation of the composite ground may be completed within about five months of the design load application (Rao and Ranjan, 1988). However, in the present case the total construction time is more than 20 months within which the total settlement is likely to be completed and thus practically negligible settlement during service period of the power house is envisaged.

CONCLUDING REMARKS

Based on the analytical computations and analysis of in-situ test data on sub-soil and on granular piles, the following recommendations were made.

* The elevations of granular pile at cobble pack base and tip of the granular piles should be maintained as El.245.65 m and El. 235.65 m respectively.
* The total number of piles below the raft be 1204 (as per spacing, Fig. 59), arranged in a zig-zag pattern.
* Adequate instrumentation be installed to monitor stress below the raft and along the pile length besides porewater pressure measurements.

PERFORMANCE OF A LARGE MANDATORY CRUDE OIL STEEL STORAGE TANK FOUNDATION ON IMPROVED SOFT MARINE CLAY DEPOSIT

INTRODUCTION

One of the largest crude oil steel storage tank having 79 m dia and 13.5 m height with a capacity of 65,000 Cu.m. was proposed to be constructed for the first time in the eastern part of the country. The site was located near the sea shore. The preliminary investigations indicated presence of very soft to soft marine clay deposits upto large depth with high water table (at 5-6m depth) and having chances of rising upto ground level during monsoon. Due to low shear strength and high compressibility of soft saturated thick layers of marine clay deposit, both, the bearing capacity failure and excessive total/differential settlement for a flexible pad foundation subjected to such a high design load could not be over ruled. On the other hand, reduction of design load reducing the height and diameter of the tank was not favoured. The other alternative of locating the tank to a better site did not receive favourable considerations either. The case study describes in brief the design analysis, construction and performance of a cost effective and efficient foundation for the steel tank founded on deep deposits of soft saturated marine clays.

WORK PLAN

The work plan included (a) Identification and selection of a most efficient and economical foundation out of the several options available, (b) Design and Analysis of an efficient and cost effective foundation for the mandatory crude oil storage tank, (c) Drawing construction specifications for the proposed foundation (d) Modifying the design specifications based on full scale insitu load testing if necessary and (e) Providing occasional supervision during actual construction of foundations.

SUB-SOIL CHARACTERISTICS

The sub soil investigation consisted of boring upto 30 m depth, standard penetration tests and undisturbed sampling besides detailed laboratory tests for shear strength, compressibility and classification tests. The details of bore log alongwith depth, SPT (N_s) values and sub-soil classifications in accordance with IS: 1498-1970 have been provided in Fig.64 and the soil compressibility in Table 14. Cohesive sub-soil deposits having clay of high to low compressibility upto 30m depth. The average N_s value was found to reduce to 9 number of blows at 15 m depth. Beyond this depth the value was found to increase to 15 at a depth of 18m and then again decreased to 10 at 22 m which remained constant upto 24m. The SPT (N_s) value were further found to increase from 10 to 20 belows at 25m depth and remained constant till 30 m.

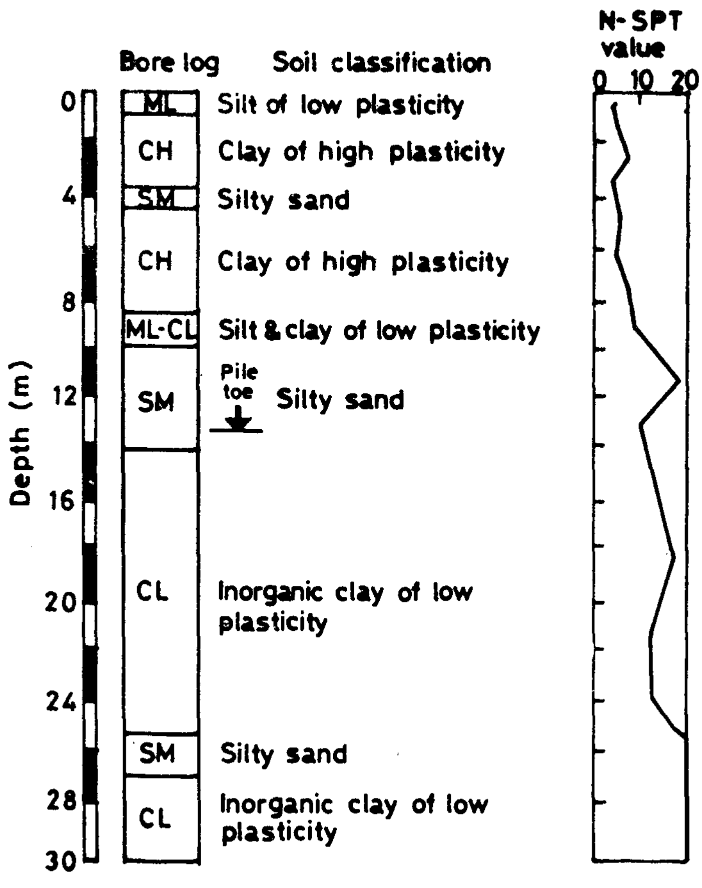


Fig 64. Details of sub soil, bore log and N-SPT values

The sub soil classification as per IS code indicated the presence of 2 m thick clayey silt deposit (ML) overlying 3m thick deposit of highly compressible clay (CH) followed by one meter thick silty sand (SM) deposit which was overlaid on a 4m thick layer of highly compressible clay deposit (CH). Further extension of bore hole indicated the presence of 1.5 m thick layer of clayey silt and clay deposit of low compressibility, (ML-CL) beyond which a 4 m thick layer of sandy silt deposit (SM) was encountered which was underlain by a 11.5m thick strata of clay deposit having low compressibility followed by 1.75m thick sandy silt (SM) layer and then again clay deposit of low compressibility started and continued up 30m depth.

Study of Table 14 and Fig. 64 revelaed the presence of highly compressible depsoit of soft clay upto 8m depth having SPT value N_s as 5 and hence the unconfined compressive strength equal to 0.5kg/cm² was taken for design purposes.

COMPUTATION OF SAFE BEARING CAPACITY :

For computation of ultimate bearing capacity, value of N_c = 6.2 and undrained cohesion C_u = 0.25 kg/cm² (2.5 t/m²). Hence the safe bearing capacity (net) $= \dfrac{6.2 \times 2.5}{F.S. = 3} = 5.16\ t/m^2$ and gross safe bearing capacity = 5.16+1x1.5 = 6.66 t/m²

SETTLEMENT OF THE VIRGIN CLAY DEPOSIT :

Intensity of design load was 14.28 t/m² (say 1.5 kg/cm²)

(i) ELASTIC SOIL MODULUS (E_s)

The elastic soil modulus E_s for soft clays according to Eq. 34 was found to lie between 20-40 kg/cm² for a SPT value N_s equal to 7 number of blows. Table 15, shows the SPT value measured with depth. and corresponding E_s values.

According to Bjerrum (1973) the value of E_s is related to undrained shear strength C_u. For normally Consolidated clay with high plasticity $E_s = 500\ C_u$. for low plasticity E_s= 1550 Cu and for soft to medium stiff clays E_s = (50-100) C_u and are expressed in kg/cm². However, Butler (1974) recommend E_s = 400 C_u for heavily consolidated clays.

Study of Table 15, indicate that between 2.4 m to 5.45 m the value of E_s is found to vary between 37.5 kg/cm² to 65 kg/cm². Therefore for the design purposes these values were accepted.

(ii) IMMEDIATE SETTLEMENT :

Utilizing appropriate values, the immediate settlement was computed in accordance with Eq.31, where μ =0.5, Applied stress q = 1.5 kg/cm²

I_p = 0.9 and B = 7900 cm, E_s = 40 kg/cm²

$$\therefore\ S_1 = \frac{q.B(1-\mu^2)}{E_s} I_p = \frac{1.5 \times 7900\ (1 - 0.25)\ 0.9}{40}$$

= 199.96 cm

S_i (say) = 200 cm.

(iii) CONSOLIDATION SETTLEMENT

The subsoil strata upto 24 m was divided in three layers, the thickness of each layer was kept as 800 cm. Utilizing the compressibilities of different layer their thickness (Table 14) and Eq. 32, the settlement for each layer was computed. The thickness of layer I was reduced by 1.5 m since the depth of foundation was 1.5 m.

TABLE 14

SOIL COMPRESSIBILITIES WITH DEPTH

Depth (m)	Compression Index (C_c)	Ratio $\frac{C_c}{1+e_0}$	Values of ($c_c/1+e_0$) used for calculations	Layer number and thickness (m)	Soil classifications as per IS:
0.83--1.28	0.467	0.091	0.091		
10.13-10.53	0.432	0.191		(0.8)I	CH
			0.091	(8-16)II	ML-CL & SM
18.0-18.48	0.464	0.224			
			0.217	(16-24)	CL
26.48-26.9	0.434	0.210			

$$S = \frac{C_c}{1 + e_0} \log_{10} \left[\frac{P_o + \Delta_p}{P_o}\right]$$

Thus, settlement in layer I $= 0.091 \times 650 \log_{10} \left(\frac{0.304 + 1.18}{0.304}\right)$

$= 40.28$ cm

S_2, settlement inlayer II $= 0.91 \times 800 \log_{10} \left(\frac{0.798 + 0.639}{0.798}\right)$

$= 39.03$ cm

S_3, settlement in layer III $= 0.217 \times 800 \times \log_{10} \left(\frac{1.406 + 0.399}{1.406}\right)$

$= 18.74$ cm

Therefore the total settlement $S_{oed} = S_1 + S_2 + S_3 = 98.06$ cm

Since the compressible layer below the tank was thick and unrestrained, the lateral deformation of the sub-soil under design load could significantly alter the consolidation behaviour of the stratum. Therefore, the correction proposed by Skempton and Bjerrum (1957) was applied to compute the settlement (St) after a time (t) which is given by Eq. 33.

$S_t = S_i + U\lambda S_{oed}$

Utilizing $\lambda = 1.0$ for marine clay and $U = 0.6$ for sixty per cent consolidation, Si = 200 cm, $S_{oed} = 98.0$ cm.

$S_t = 200+0.6x1.0x98$

=258.8 cm

Therefore, the total settlement of the pad foundation placed at 1.5m below the natural soil level was expected to be 2.59 m under the design load of 15 t/m^2.

Thus the available safe bearing capacity was not found sufficient to support the applied load without undergoing shear failure. Also the predicted settlement of the tank foundation was excessive to deserve any consideration for the pad foundation.

CHOICE OF FOUNDATION :

While selecting a particular foundation type, constraints of cost, available time, inadequacy of the design data, nonavailability of appropriate equipment etc. all tend to compound the problem. Though the decision making is primarily based on technical soundness of the foundation in terms of short and long term performance during service period.

In view of the low bearing capacity and high total and differential settlement of the pad foundation under high intensity of applied stress, several options were considered to select an efficient, speedy and economical method of foundation for the tank.

COMPENSATED RAFT FOUNDATION

Partially compensated rigid raft placed at 3m below the natural soil level was considered as one of the possible solution for (79 m dia x 14.5m ht) steel tank. However, in view of very low partial relief (2.29 t/m^2) available due to high water table under fully submerged condition. besides, the method called for a perfect dewatering system and an impermeable RCC diaphragm wall around the tank foundation upto 3m depth, and was also found costly and time consuming. Hence the method was not found acceptable.

DRIVEN CAST INSITU PILES

Driven cast insitu bored piles could have possibly provided satisfactory foundation alternatives. However, due to defects such as honey combing, necking, dislocation of pile toe during driving, in the system particularly in soft marine clays the method was not preferred.

In addition, the magnitudes and rate of development of negative drag, which is almost a certainty when

TABLE 15

ELASTIC SOIL MODULUS (E_s) VALUES WITH DEPTH

Depth (m)value	SPT, N_s (SPT)	Commulative average N_s (SPT)	Average N_s (SPT)	UCC Strength (t/m²)	Elastic soil modulus (t/m²) E_s=100 Cu
2.45	8	8	7	7.5	375
4.0	4	6			
5.45		6			
6.95	9	8	9	13	650
8.55	10	9			
10.15	10	9.5			
13.15	11	10.25			
15.25	11	11.0	11	15	750
18.00	12	11.5			
19.95	10	11			
21.45	10	11			
23.45	10	11			
26.45	20	16	16	20	1000
27.90	20	18			
30.00	20	19			

point bearing deep piles, are placed in soft clay which is highly compressible could not be overlooked since over conservation in disign gain favour, because in the present case, a pile called for to support an additional load of 42 tonnes (30 cm dia and 24m deep) with an undrained shear strength of 0.25 kg/cm².

PRECAST DRIVEN PILES :

Due to large length of 24m, such piles were not recommended since appropriate splicing method and design criterias were not available within the country, a decade and a half ago. Further no construction firm in the country had the adequate construction experience in these lines. Moreover, in precast piles, introduction of slip layer was a must to safeguard against negative skin friction which added to the cost of foundation.

It may be mentioned,that for steel storage tanks, flexible raft are generally preferable over the rigid rcc raft foundation because the development of cracks in the rcc raft cause concentration of stresses and result into rupture of the steel base plate of the tank. Lastly the cost of the rigid rcc cap on large number of piles was found to be much higher than the cost of the tank itself.

PRELOAD ACTUATED SAND DRAINS :

In present case, use of sand drains was found effective only when preload intensity was increased to 10 tons/m² to achieve 90 per cent consolidation within a period of four months. However, in practice, the quantity of sand required for achieving such a high intensity of stress and time to transport the sand from long distances, their stacking etc. was prohibitive and costly. Besides, the cost, the time required for achieving the desired improvement was found to be more than two to three times.

Considering the several options discussed above, and keeping in view the techno-economic factors besides availability of equipment, material and time constraint skirted granular pile foundation (Fig. 65) was recommended for the foundation of the large storage steel tank.

ULTIMATE PILE CAPACITY

Modified cavity expansion approach was used to compute the ultimate capacity of single granular pile (Rao, 1982; Ranjan and Rao, 1991).

Input data

assumed bore hole diameter	=	45 cm
Installed pile diameter	=	55 cm
Cross sectional area of the pile	=	0.2376 cm²
Undrained shear strength of clay upto 5 m depth for natural soil	=	3.75 t/m²

submerged density γ_{sub}	=	0.76 t/m^2
Elastic modulus of the pile material (E_p)	=	5000 t/m^2
Elastic soil modulus (E_s)	=	400 t/m^2
load (q_s) shared by the ambient clay when the design load was 15 t/m^2	=	1.1 t/m^2

Utilizing appropriate values of the soil and pile parameters obtained from field and laboratory tests and using Eqs. 6 the ultimate load capacity of a 55 cm dia single pile was found to be as 53.66 tonnes. Thus with a Factor of Safety equal to 3, the safe load capacity for a single pile was found as 17.88 tonnes.

Since the total load on foundation = 70,000 tonnes

$$\text{Therefore total number of piles } = \left[\frac{70,000}{17.88}\right] = 3915 \text{ piles}$$

Adapting a pile spacing of 1.65 m to c/c in a zig-zag triangular pattern, the actual number of piles worked out to be 3980 only.

DEPTH OF GRANULAR PILES

Study of Fig. 64 and Table 15 indicate that the average value of SPT (N_s), upto 4 m is 7 and 9 between 5 m to 9 m. Beyond which the SPT (N_s) values were found to increase from 9 to 11 upto 22 m depth. Also it was again found to increase further to 16 number of blows between 23 m to 30 m depth. In view of this and also keeping in view the large anticipated settlement under a 79.9 m diameter flexible raft the maximum depth of granular piles were recommended to be 15 m below the cutoff level, 0.55 m below natural soil level.

COMPUTATION OF SETTLEMENT :

The settlement of the tank was computed by using equivalent coefficient of volumecompressibility approach (Rao, 1982; Rao & Ranjan, 1985) by using 2 : 1 method of load dispersion and dividing the subsoil into ten layers of varying thickness (Fig. 66).

Input data :

Diameter and height of the tank = 79m and 13.5m

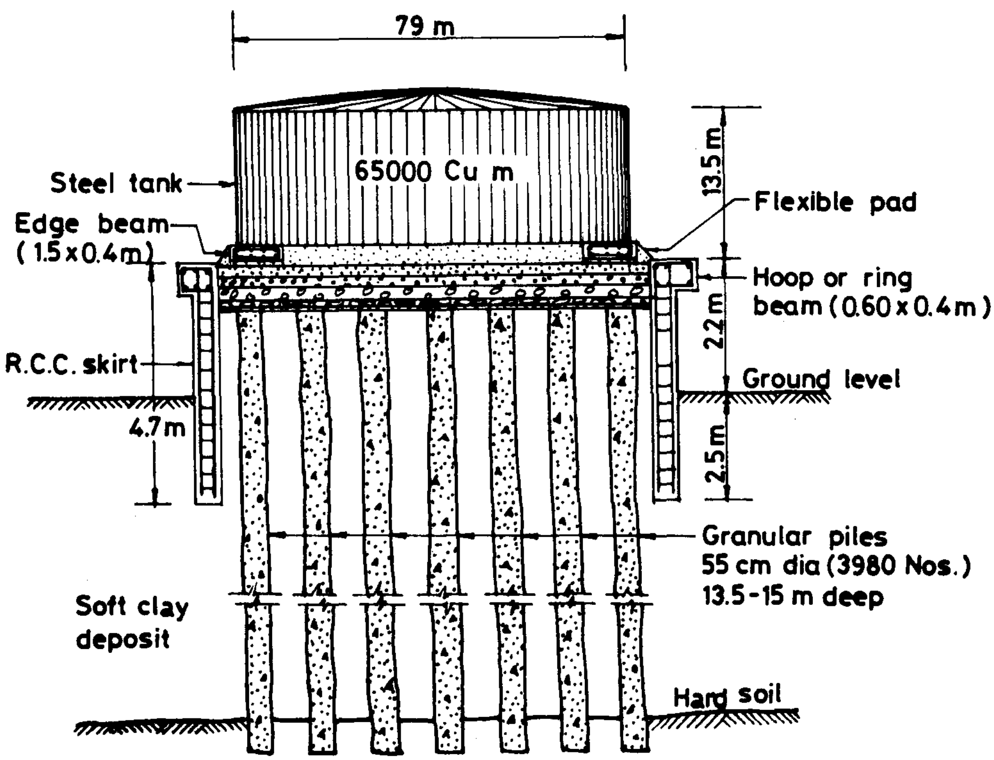


Fig. 65 Mandatory crude oil tank foundation

Full tank capacity	=	65000 Cu.m.
Density of the crude oil	=	0.88 t/m^3
Area of the tank base	=	4902 m^2
Area of the flexible pad below base plate	=	5014 m^2
Unit weight of pad material	=	2.0 t/m^3
Weight of the pad material	=	5014 x 2 x 1.55=15543.4 tonnes
Weight of the water	=	4902 x 12.5=61765.2 tonnes
Weight of the steel	=	1500 tonnes
Thus the total load	=	78808.6 tonnes
Hence intensity of stress below flexible pad	=	15.71t/m^2
and intensity of stress due to shell & pad	=	3.4 t/m^2

If the installed pile dia = 55 cm and total number of granular piles are 3980.

Hence the stiffness ratio α = 0.1885

$$: (1 - \alpha) = 0.81147$$

EQUIVALENT MODULII :

The elastic soil modulii for different layers of clay deposit is taken from Table 15 and also presented in Fig. 66. The modulus for pile material E_p is taken as 5000 t/m^2 and for pad material E_{pad} equal to 2000 t/m^2.

Therefore the E_{eq} of the composite mass of soil below the flexible pad and above the tip of skirt and in layer I also is found as

$$E_{eq} = 0.188 \times 5000 + 0.81147 \times 375 = 1247 \ t/m^2$$

and E_{eq} in layers II,III & IV 1470 t/m^2 and for layer V = 1551 t/m^2 respectively. Further the elastic modulii for virgin clay (unreinforced), for layers VI & VII is 750 t/m^2 and for VIII, IX & X is found as 1000 kg/cm^2.

Thus utilizing appropriate values of different parameters narrated above and presented in Fig. 66 and using Eqs. 9 to 11 and 13 (Rao, 1982; Rao & Ranjan, 1985), the total settlement due to pad, and shell plus floating roof of the tank for an intensity of stress 3.4 t/m^2 was found to be as 7.5 cms. Later on, this load was revised to 4.7 t/m^2 by the client.

Therefore the settlement due to pad + shell = 10.36 cm. Similarly the total intensity of load due to water and pad plus shell which was originally 15.17 t/m^2 which was also revised later to 16.6 t/m^2 hence the settlement under this intensity of stress was found to be as 50.58 cm which included the settlement due to pad plus shell and floating roof. Thus the settlement due to water load only is found as 40.22 cm, which is equivalent to 3.27 cm per unit stress due to water loading only. Therefore the above data suggested that during the construction of the flexible pad and the shell a total settlement equal to 10.36 cm had already been completed and during the hydrotesting of the tank the expected settlement was 40.22 cm only. Thus due to treatment of the weak subsoil deposits by skirted granular piles (3980 in number) the settlement reduction ratio β was found as

$$\beta = \frac{40.22}{259} = 15.22 \text{ per cent}$$

Hence percentage reduction in settlement = 84.47%

TIME DEPENDENT SETTLEMENT

The fact that the soft clay deposit was recommended to be reinforced with 3980 numbers, 55 cm diameter granulor piles resting on a silty sand layer 1.5 to 1.75 m thick, hence both immediate and short term primary settlement shall be over during the hydrotesting of the tank. It was further noted that about 55cm of settlement expected in the unreinforced clay below granular pile tip and upto 34m during tank construction and laying of flexible pad which may increase to 14.37cm during water load testing.

It was therefore felt essential that for the proper functioning of the tank during service period, an adequate provision was to be made for pipe line connections for filling and taking out the crude oil from the tank. This necessiated prediction of long term settlement of the tank foundation during service period. To achieve this requirement two important aspects were taken into consideration viz. (i) though the settlement due to water testing is predicted for a design intensity of 12.3t/m^2 however in actual practice this load intensity will be reduced

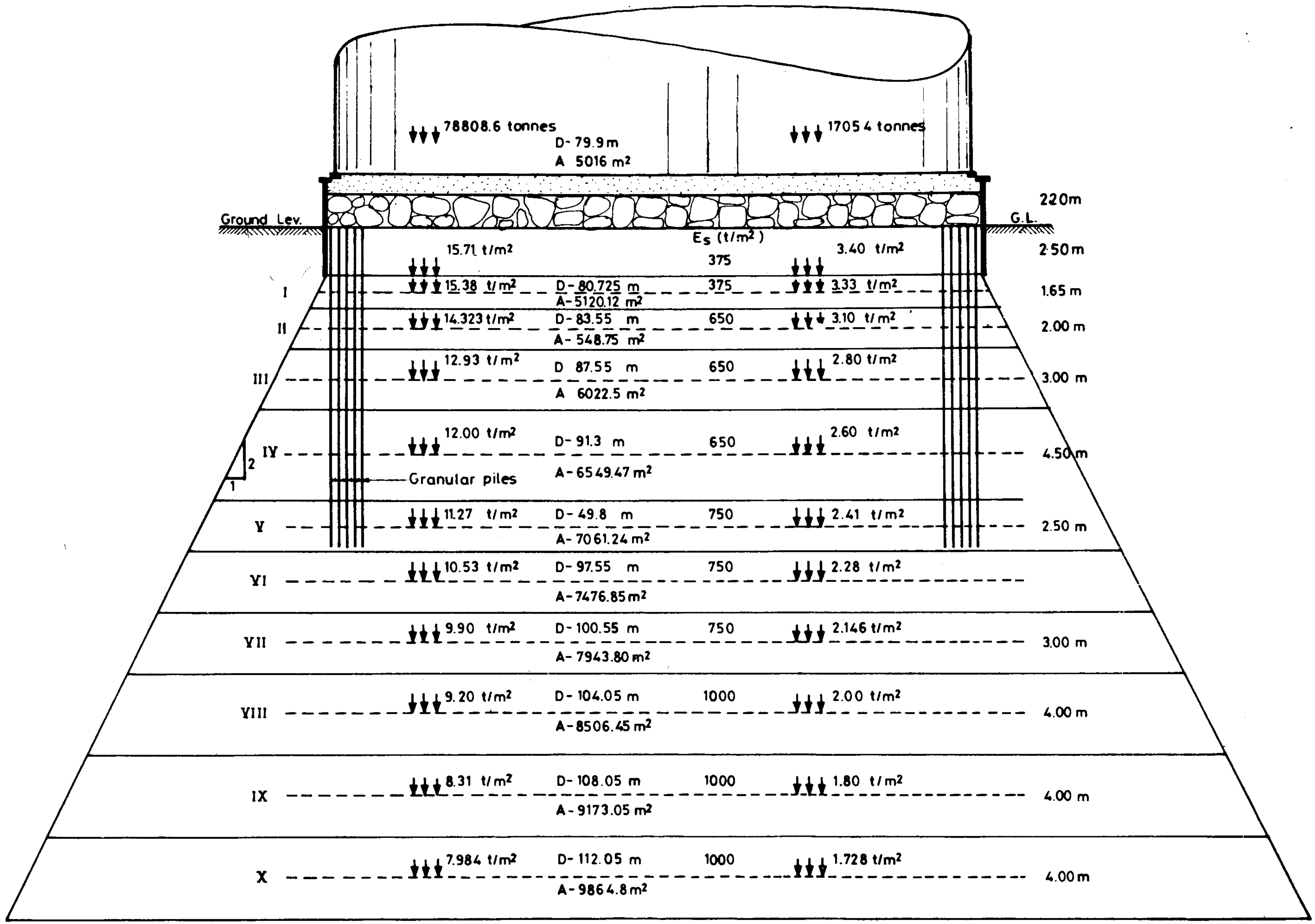


Fig 66 Computation of settlement of composite soil strata supporting mandatory crude-oil steel tank

to 10.2t/m² since the density of the crude oil being 0.85 gm/cc and (ii) the crude oil even filled up to full capacity of the tank, the corresponding intensity of stress will not increase the value more than 10.2t/m² and the crude oil is not likely to remain in the tank for a period of 3 months at a time. Therefore, keeping above points in view and also in the absence of any reliable method of prediction of time dependent settlement of a composite mass of cohesive soils deposit reinforced with granular piles a provision of 20% of the total computed settlement (i.e. equal to 8.0 cm) was recommended (Rao & Ranjna, 1988).

The total settlement of the skirted granular pile foundation (Fig. 66) as predicted on the basis of subsoil properties and using Eq. 9 to 11 and 13, based on equivalent coefficient of volume compressibility approach(Rao & Ranjan, 1985 and 1988) is provided below.

SETTLEMENT DURING INITIAL TANK LIFTING

The tank foundation was subjected to an initial intensity of stress equal to 2.34 t/m² during initial lifting of floating roof of the tank which is achieved by filling the tank upto 2.4 m height of water in the tank. This settlement was expected not to increase a value equal to 7.13 cm before hydrotesting. The settlement of the ring beam due to weight of pad + shell = 10.36

Settlement during initial roof lifting = 7.13 cm
Settlement during hydrotesting = 40.22 cm
and time dependent settlement = 8.00 cm

Therefore, a provision of 43.22 cm of settlement, during hydrotesting of the tank and 8 cm as time dependent settlement was made.

FIELD VERIFICATION THROUGH FULL SCALE INSITU TESTS :

A field demonstration on full scale, was carried out for the installation of granular piles using auger boring method (Rao, 1982; Rao & Bhandari, 1979; Ranjan & Rao, 1983; 1988-1991) for the benefit of engineers of the

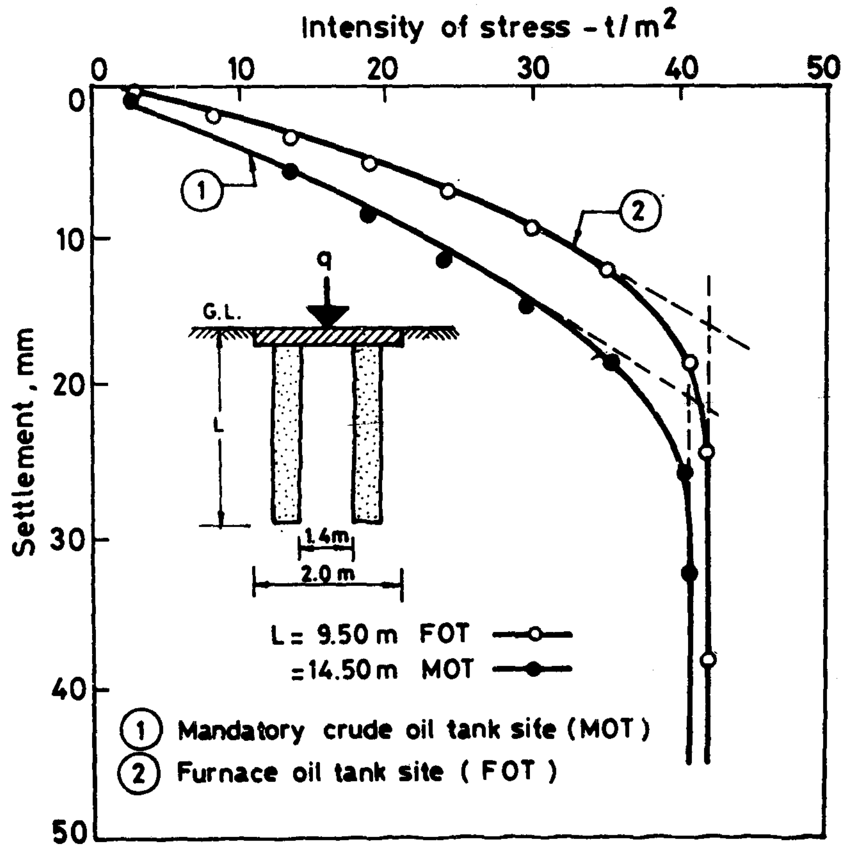


Fig. 67 Stress - settlement curve on composite ground reinforced with group of two granular piles in soft clay deposit

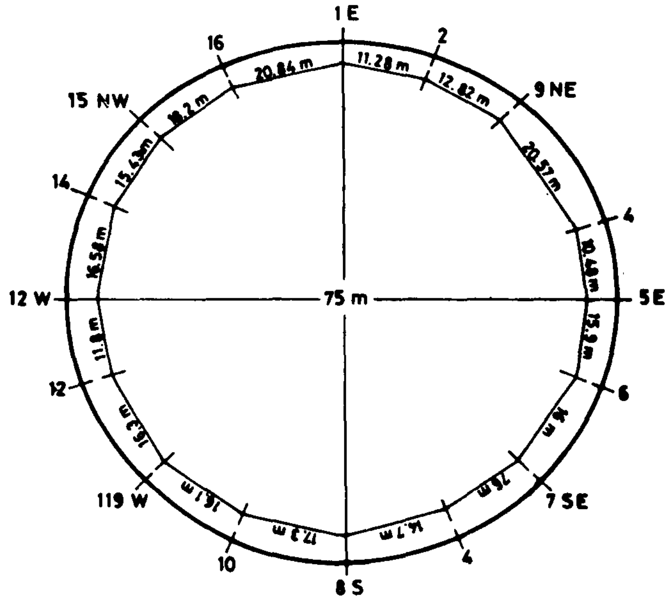


Fig 69 Level points on ring beam (MCO-108)
Haldia Refinery

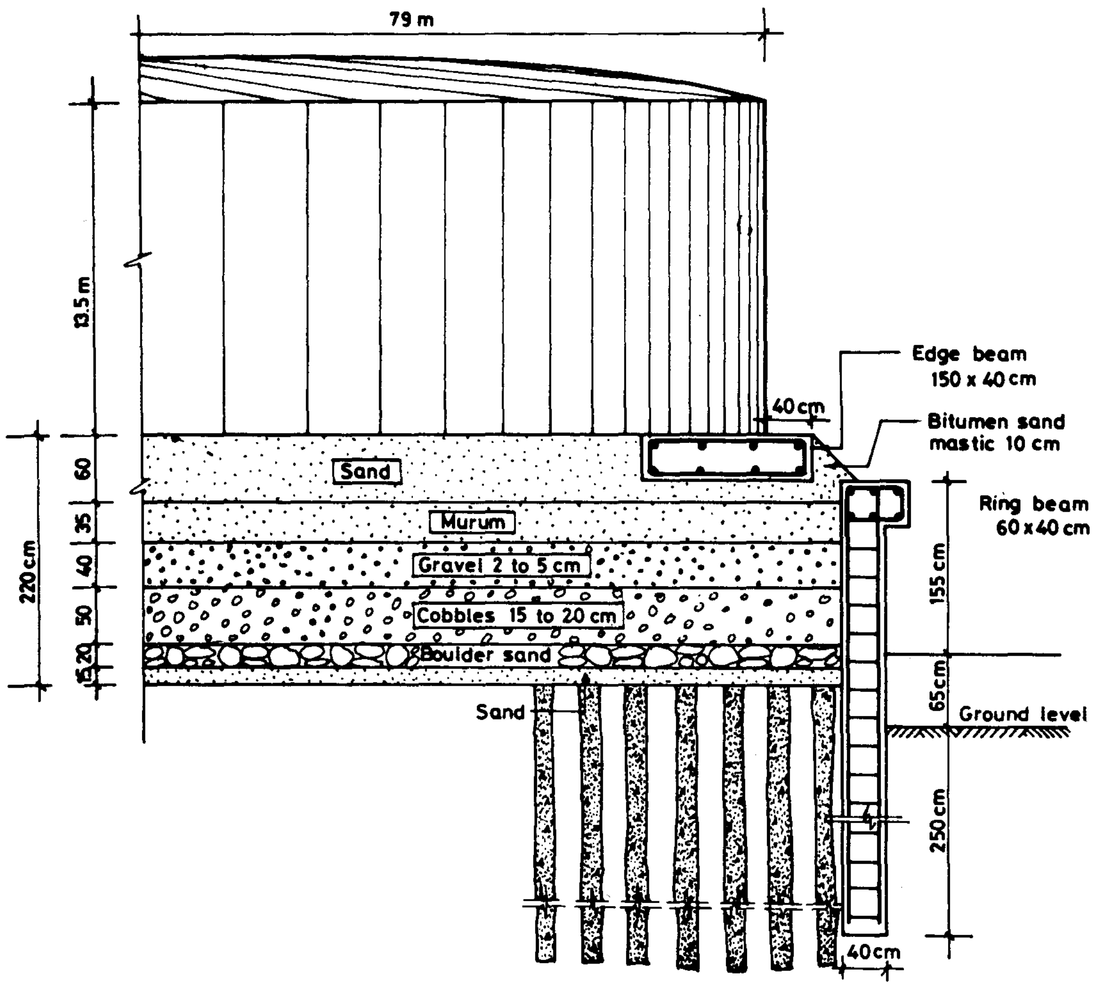


Fig. 68 Detail of flexible pad skirt edge beam ring beam etc.

department and also finding out the installed pile diameter, by actually measuring the material consumed in the bore hole. Secondly, these piles were to be load tested to their ultimate capacity, with a view to verify the validity of design assumptions made while predicting the granular pile capacity utilizing modified cavity expansion approach (Rao, 1982; Ranjan and Rao, 1987-1991).

The exercise of field demonstration was carried out at two sites viz. (i) at Mandatory crude oil tank (MOT) site and (ii) at Furnance oil tank (FOT), site, which was another site in the same refinery area. The diameter of the bore hole was kept 45 cm on both the sites and the depth of the bores were kept as 13.5m at mandatory crude oil tank (MOT) site, as proposed earlier on the basis of the sub-oil strata and the design load. However, the sub-oil strata and design requirements indicated that 9.5 m deep piles at Furnance oil tank (FOT) site will be sufficient. Three numbers furnance oil tanks (42m diax10m high) at the second site were proposed to be constructed. The sub-soil strata at MOT site was found to be better beyond 9 m depth.

On both the sites, the volume of the consumed material (stone aggregates and sand) indicated that the diameter of the installed piles, varied between 54cm to 57cm with an average of 55 cm. Thus a group of 2 piles 55 cm in dia and 14.5m in depth was installed with a RCC cap (2 m x 1 m x 0.3 m) with a center to center spacing as three times the pile diameter at MOT site and exactly another group of 2 piles having the same size, spacing and cap sizes were installed at FOT site. However the depth of piles were limited to 9.5 m only. These two pile groups were insitu load tested upto their ultimate capacity in accordance with IS : 2911 (Part IV)-1979. The load settlement relationship for both MOT and FOT sites have been shown in Fig. 67, which indicated that the pile group at MOT site reached its ultimate at a total load of 82 tonnes and at FOT site it was found to be 84 tonnes. Thus with a factor of safety equal to two on the ultimate load, the average safe load capacity for a single pile was found to be 20.5 and 21 tonnes respectively against the predicted capacity of 18 tonnes. The corresponding settlement for these safe load capacities were found to be 2.5mm for FOT and 6.5 mm for MOT site. Study of Fig. 65 further indicate that even at an intensity of 70 t/m^2 which is well withing elastic range the settlement does not exceed 12 mm and 17 mm respectively. On the other hand, from insitu load test results at both the sites, the bearing capacity of the composite mass reinforced with granular piles is found to be 21 t/m^2. However, the safe bearing capacity for the untreated sub-soil was computed as 6.66 t/m^2 using undrained cohesion C_u. Thus an improvement of almost 300 per cent in bearing capacity is clearly indicated and also, the basic assumptions made in design prediction is fully verified.

HYDRO-TESTING PROCEDURE

Sixteen reference points on the ring beam (Fig.68) were provided with a caution that no two points are located within a distance of 10 m from the centre line of the mark. Level points on ring beam has been shown in Fig.69. The filling of water (Table 16) was caried out in nine stages (3.5, 6m, 7m, 9.5m and 12.3m) in terms of height of water in the tank. The rate of filling was kept as 100 cu.m. per hours which corresponds to about 0.5m ht of water. The levels of all the 16 points were taken continuously to ensure that settlement does not exceed a rate of 10mm per 24 hours and differential settlement is not more than 1 in 400 before the filling.

The rate of water filling in terms of height of water and the time in days and the corresponding settlement for all the nine stages of water loading is shown in Fig. 70. The rate of settlement during filling of the water in the case of first three stages of loading was found to lie between 2.5 mm to 4 mm per 24 hours, while for the higher loadings (4th to 9th stages) the rate reduced to a value betwen 1.5 mm to 3 mm per 24 hours. The next stage of loading was applied only when the settlement rate was 1.5 mm per 24 hours or after a period of seven days, whichever was more.

MONITORING OF SETTLEMENT DURING HYDROTESTING

The settlement of the tank at each of the sixteen locations already marked on the ring beam were monitored continuously during the hydrotesting of the tank. The recording of settlements were carried out at an in interval of 12 hours both at 6 am in the morning and 6 pm in the evening. The hydrotesting was started on 6th Aughst, 1984 and completed on 24th May, 1985. The filling of water in the tank was carried out in nine stages. The details of water loading, unloading recording of settlement and pause time have been presented in Table 16.

As stated, the settlements of all the sixteen points on the ring beam were monitored at an interval of 24 hours, for each stage of loading and the same criteria was maintained till the desired rate of settlement was achieved. The total average settlement for each load increments were thus obtained and recorded in column 7 of Table 16. Further, column 8 shows the settlement corresponding to a pause time of 7 days and column 9 shows the settlement during pause time beyond 7 days for each stage of filling. The additional pause time beyond 7 days

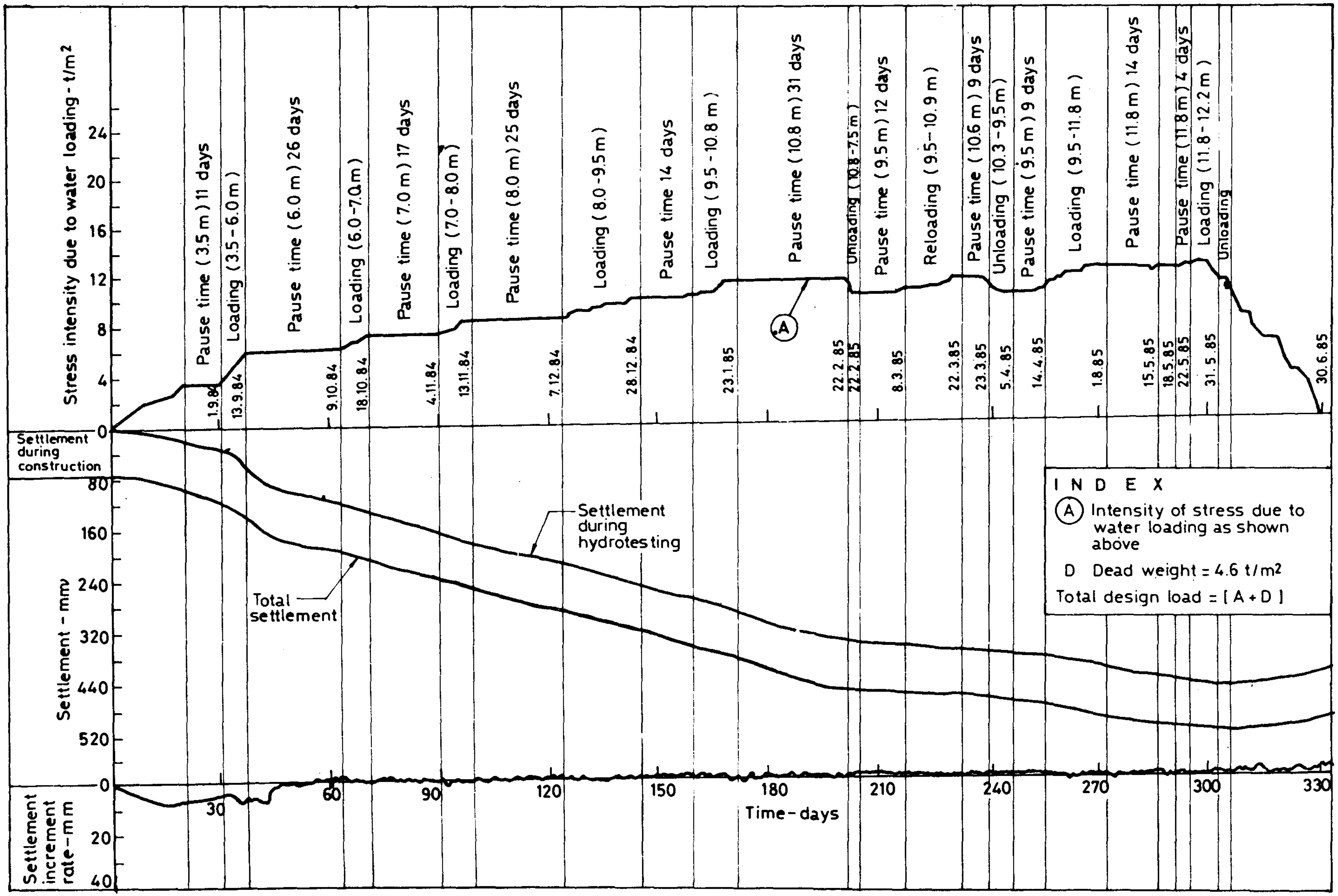


Fig 70 Field performance of oil storage steel tank during hydro-testing MCO-108 (79 m dia x 13.5 m height) capacity 65000 cu.m.

TABLE 16

SETTLEMENT OBSERVATIONS OF OIL STORAGE TANK (MCO-108) DUGING HYDROTESTING HALIDA REFINERY (WEST BENGAL)

Stage of Loading	Water Loading Time From	To	Date for Water Ht. Maintained at Each Stage	Height of Water (m)	Intensity of Stress (water and Dead Load) t/m²	Total Settlement in Each Stage Loading (mm)	Settlement Corres-ponding to 7 Days Pause Time (mm)	Settlement during Pause Time Beyond 7 days (Col. 7-8) (mm)	Additonal Pause Time Beyond 7 Days (Days)
1	2	3	4	5	6	7	8	9	10
I	6-8-84	26-8-84	6-9-84	0.0-3.5	8.00	34.76	29.95	4.80	6
II	9-9-84	13-9-84	9-10-84	3.5-6.0	8.96	90.73	67.55	23.18	15
III	10-10-84	18-10-84	4-11-84	6.0-7.0	11.49	49.75	33.13	16.63	19
IV	5-11-84	13-11-84	7-12-84	7.0-8.0	12.42	54.01	38.54	15.47	13
V	8-12-84	28-12-84	11-1-85	8.0-9.5	13.89	59.95	51.72	8.22	7
VI	12-1-85	23-1-85	22-2-85	9.5-10.8	15.15	74.96	39.58	35.38	24
Unloading	23-2-85	24-2-85	8-3-85	10.8-9.5	13.89	5.50	2.00	3.50	6
VII	9-3-85	22-3-85	29-3-85	9.5-10.8	15.15	16.18	14.68	1.50	1
Unloading	31-3-85	5-4-85	14-4-85	10.8-9.5	13.89	5.25	4.75	0.50	2
VIII	15-4-85	1-5-85	14-5-85	9.5-11.8	16.13	37.19	31.80	5.39	6
Unloading	14-5-85	15-5-85	23-5-85	11.8-11.3	15.32	0.25	-	-	-
IX	24-5-85	31-5-85		11.8-12.3	16.62	17.50	17.50	-	-
			Total Settlement			400.77	331.06	114.57	90

Total settlement including 7 days pause time = 331.0 mm
Total settlement during pause time = 114.6 mm
Settlement due to pad and initial lifting of roof = 71.3 mm
Final settlement during Hydrotesting = **516 mm**
say 517 mm

has been shown in column 10. Further Fig. 69 shows the time settlement behaviour of skirted granular piles, during hydrotesting under different intensity of stress. The upper curve present the time settlement behaviour only due to weight of water and the lower curve indicate the total settlement with time including the settlement due to dead load during the construction. The total settlement under 16.6 t/m² of stress = 331.0 mm for 7 days pause time was recorded.

The additional settlement due to pause time = 114.6 mm time beyond 7 days was found.

Settlement due to pad load and initial roof lifting = 71.3 mm
Total settlement during hydrotesting 516.9 mm
Say (517.0 mm)

The tank has since been commissioned and is in service since last ten years.

STRENGTHENING OF DISTRESSED STEEL TANK FOUNDATION

INTRODUCTION

A steel molasses tank (24 m x 6.75 m) was founded on a traditional foundation in accordance with the I.S. Code of Practice for design and construction of steel molasses tanks. The foundation was designed for a total load of 5000 tonnes with adequate safety against shear failure and angular distortion on loose to medium dense cohesionless soil deposit. Attempt were made by the department to complete the super structure before the start of crushing season during 1977 but without success. During the crushing season, the molasses had to be stored in open underground masonary tank, when the tank was filled up, additional kacha pits were also used (Fig.71). The locations of three pits in the lay out plan have been shown in Fig. 72 and the sectional view of the steel tank founded on ring foundation is presented in Fig.73.

The molasses from the open pits were suspected to have impregnated into the foundation subsoil below the steel tank base. When the errection of the steel tank was in progress, simple walking on the base of the tank indicated that supporting sub-soil was slushy. At this stage, the problem was referred to Central Building Research Institute for diagnosing the causes of distress and suggesting cost effective strengthening measures for the distressed tank foundation and also influence of acidity due to impregnation of molasses in brick masonary, concrete and subsoil.

SUB-SOIL INVESTIGATION

Detailed sub-soil investigation consisting of field and laboratory test were carried out near and inside the tank. The test locations have been marked on Fig.72 and the test results have been presented in Fig. 74. Study of Fig.74 indicated that the dynamic cone penetration test results in general corroborated well with standard penetration test values. Plate load test data indicated that the safe bearing capacity was 7.0 t/m². Whereas from extrapolation equation, (IS. 1888-1981), for 24 m diameter tank, it was found to be 12.4 t/m².

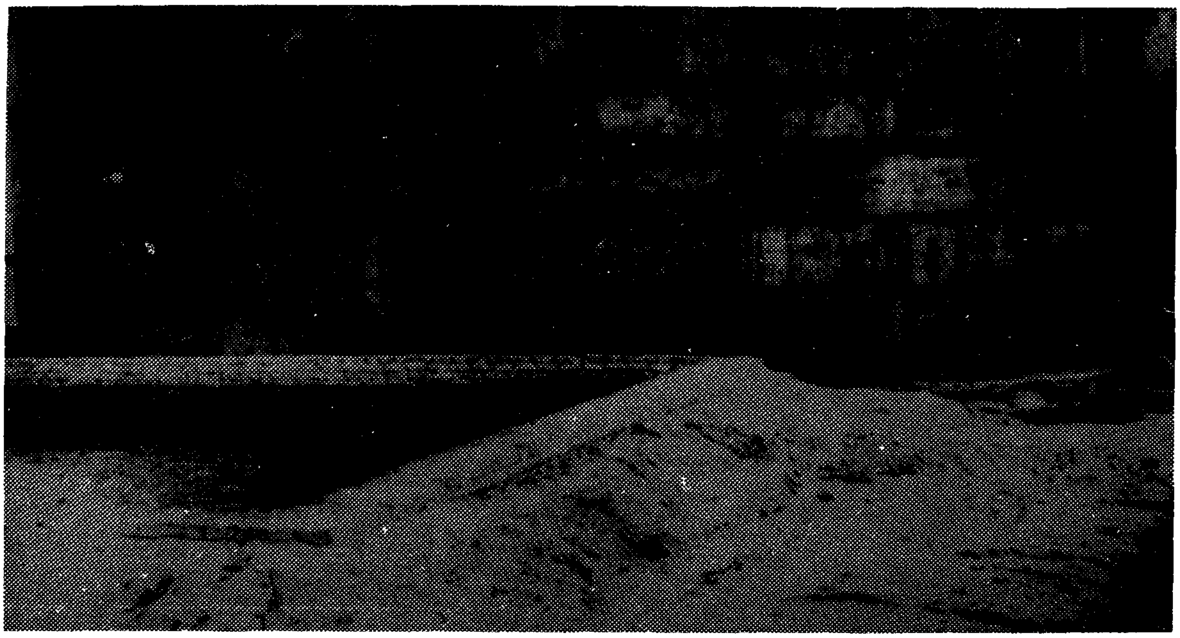


FIG 71. STEEL MOLASSES TANK ALONGWITH KACHA MOLASSE TANK

INFLUENCE OF MOLASSES ON FOUNDATION SUBSOIL

The problem of foundation damage due to the aggresive chemicals present either in the ground or chemicals works or in the waste dumps, was a serious problem due to main difficulty in identifying the full range of deleterious compounds from a limited number of samples. With a view to study the harmful effect of molasses on sub-soil below the foundation, concrete brick and soil samples were collected from 0.5m, 1.0m and 1.5m depth and also from surface Figs.75 and 76. The molasses content and PH values found from the laboratory tests have been presented in Table. 17. It is indicated from the table that the PH values varies between 5.6 to 6.5 These low values were indicative of high percentage of molasses penetration which may have adverse effect on foundation concrete and soil.

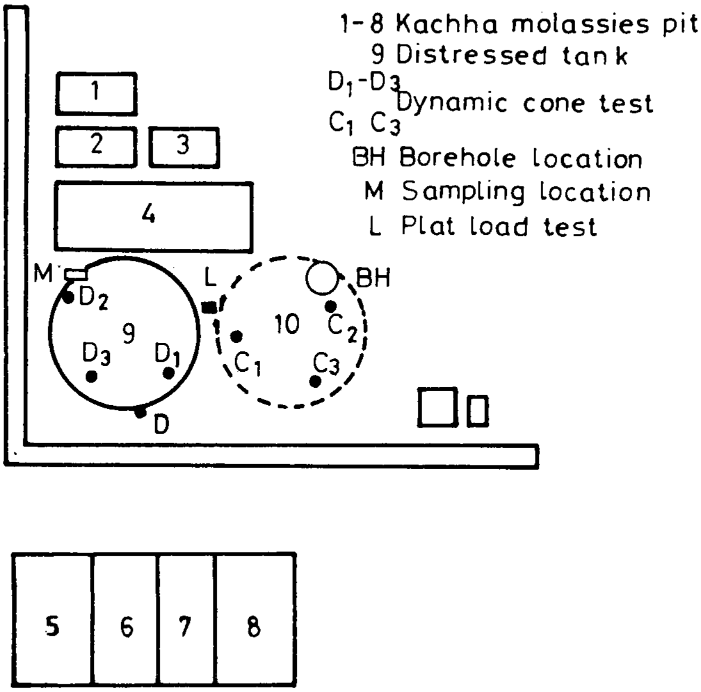


Fig 72 Site Plan Showing Location of Distressed Tank & Test Points

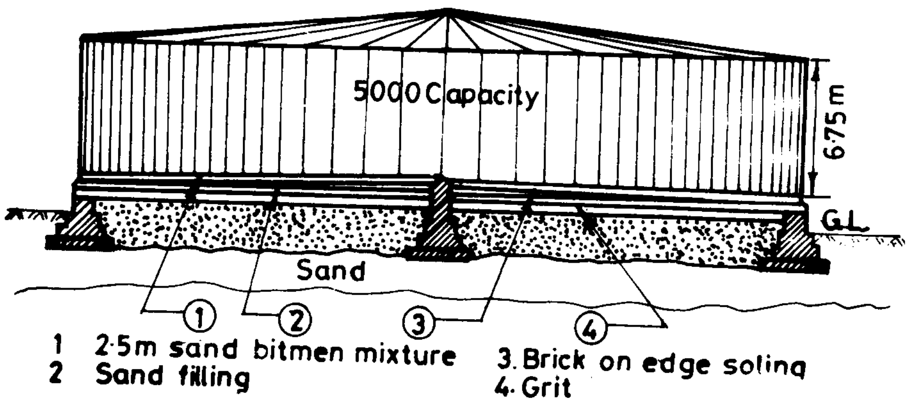


Fig. 73 Distressed Tank Showing Foundation Details

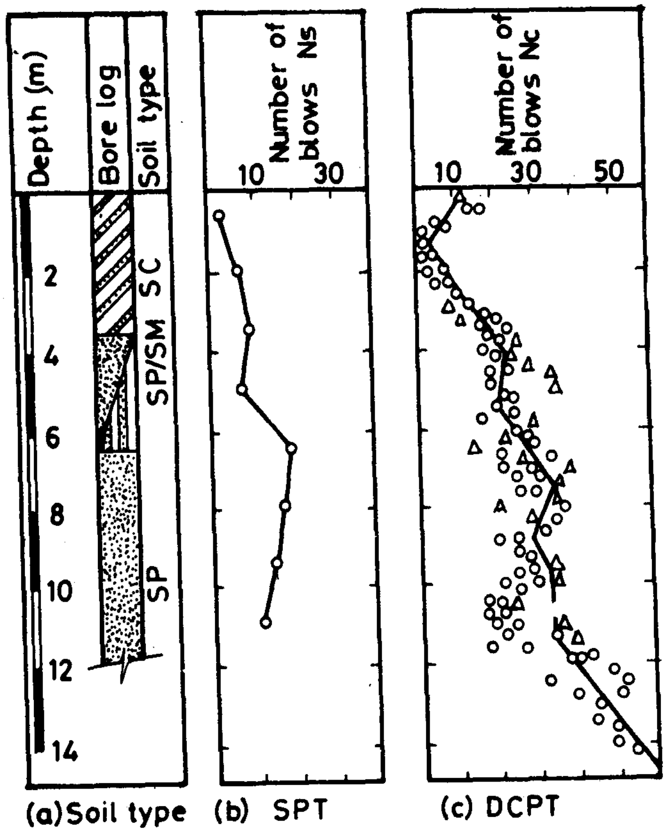


Fig 74 Borelogs & Penetration Test Results

TABLE 17

MOLASSES CONTENT AND PH VALUES

Depth (m)	Molasses Content %	PH Values
Surface	4.2	6.5
0.5	21.0	5.8
1.0	28.0	5.6
1.5	15.0	6.2

EFFECT ON BEARING CAPACITY OF SOIL

Test on soil sample collected from 1.5m depth indicated an increase in natural density from 1.88 g/cc to 2.03 g/cc and Cohesion value of 0.55 kg/cm² and ϕ = 13° from UU test. The corresponding strength values from isotrophically consolidated drained tests, indicated the soil to be noncohesive with ϕ = 21°. However, the virgin soil had ϕ = 28°. The molasses affected soil showed a safe bearing capacity between 1.7 kg/cm²-3.96 kg/cm². Thus, even the lower values of the safe bearing capacity was found to be almost twice the design load intensity of 0.95 kg/cm². These indicated higher strength value due to the effect of molasses on sub-soil.

EFFECT ON CONCRETE, CEMENT MORTAR AND BRICKS

The free lime content in the affected concrete was found to be 0.34 per cent only which is on low side. Thus due to the action of sugar solution on the concrete, it is likely to have low strength due to formation of calcium saccharate. Consequently cement mortar loses its strength resulting in separation of grains of sand and aggregates from the concrete. The Differential Thermal Analysis (DTA) of the concrete sample indicated large exothermic peak at 350-360°C (Fig.77) which may be accounted for decomposition of Calcium saccharate. Howerve, in the case of cement mortar one exothermic peak at 370-380°C and large endothermic peak at 820°C (Fig.78) indicated presence of calcium saccharate and calcium carbonate respectively. The PH value was again found to be 9.5 which is on lower side. these data confirmed the findings narrated above.

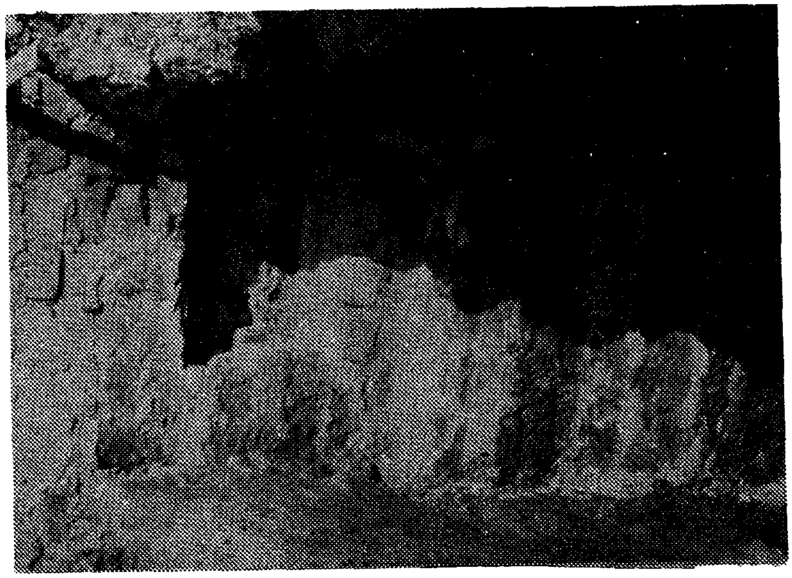


FIG 75. A VIEW OF MASONRY SKIRT WALL

ALLOWABLE SOIL PRESSURE FOR THE EXISTING TANK

The factor of safety against base shear failure for a raft overlaid on sand is always very large hence the danger that the raft may break into a cohesionless soil mass is too remote to require any consideration. More so, the differential settlement is also likely to be smaller than that of a footing foundation designed for the same soil pressure. Hence it is reasonable to permit large allowable soil pressure for raft foundation. The allowable soil pressure, q_a for raft foundation may be found from corrected standard penetration values, N'_s, for a total settlement of 50 mm from Eq. 49 (Peck, Hanson and Thornburn, 1974)

$$q_a = 0.22\, N'_s \qquad \text{....(49)}$$

For N's values less than 5 and greater than 50, use of raft foundation is not recommended. Thus the gross allowable soil pressure q_u is given by Eq. (50).

$$q_b = q_a + \gamma_{bulk}\, D_f \qquad \text{....(50)}$$

where γ_{bulk} is the unit weight and D_f is the depth of foundation. Selecting average corrected comulative N's value as 5, and assuming water table at 2 m below the ground level, the corrected gross allowable soil pressure q_b was found to be 9.53 t/m². Therefore the total load that the sub soil can support for a 24 m diameter raft is 4309 tonnes, against a total design load of 5000 tonnes. Hence the existing foundation was found to be unsafe indicating the necessity for remedial strengthening of the subsoil below the tank foundation upto a minimum depth of 3 to 3.5m.

STRENGTHENING MEASURES

Several factors such as the design requirement suiting the sub-soil conditions and the superstructure, besides the time constraints and the cost were the prime consideration to arrive at a cost effective and speedy method of foundation strengthening measures to be adapted for the distressed tank foundation. To satisfy these parameters various method of strengthening measures were examined and finally installation of granular piles developed at the Central Building Research Institute (Rao, Bhandari and Sharma 1979; Rao, 1982) at a design spacing of 3.5 times the installed pile diameter was favoured and recommended. The depth of piles was kept between 3 to 3.5 metres.

THEORETICAL CONSIDERATIONS

It was essential to determine the safe load capacity of the granular piles, as also the total and differential settlement of the tank base in order to arrive at the safe load that the composite subsoil mass consisting of the granular piles and the ambient loose-medium dense cohesionless deposit upto 3.0m could bear without rupture.

LOAD CARRYING CAPACITY OF GRANULAR PILES

The ultimate load carrying capacity of a single granular pile was computed, using Modified Cavity Expansion Approach (Rao, Bhandari & Sharma, 1979; Rao, 1982).

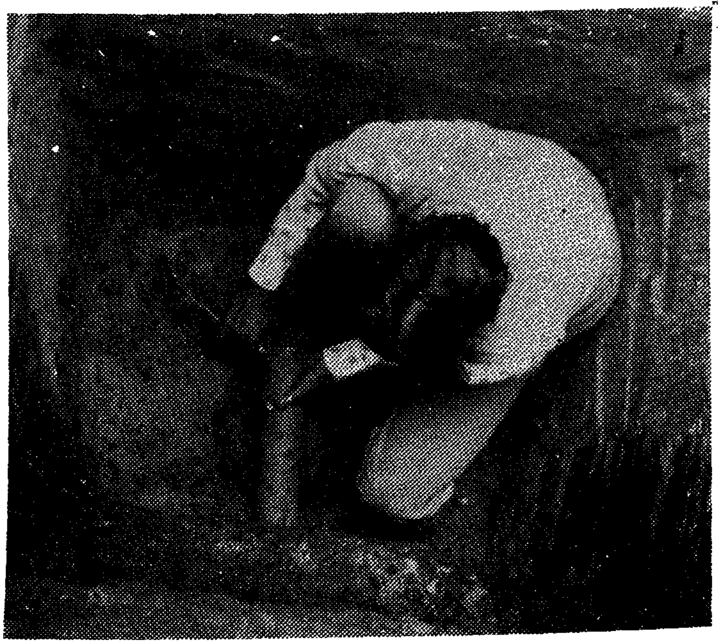


FIG 76. COLLECTION OF MOLASSES AFFECTED SAMPLE IN HORIZONTAL DIRECTION

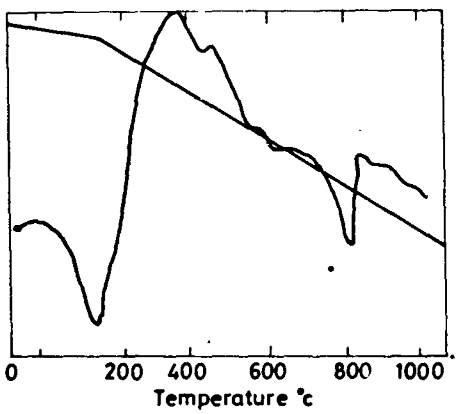


Fig 77 Differential Thermogram Concrete

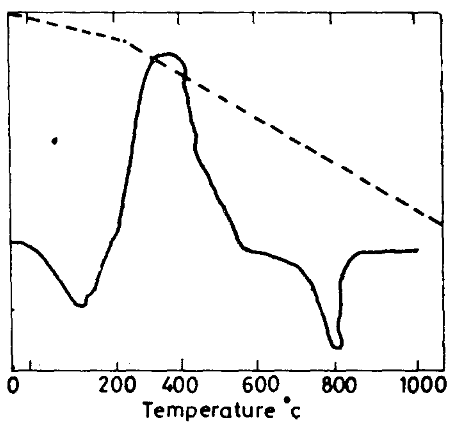


Fig. 78 Differential Thermogram Cement Material

Substituting appropriate values of various parameters found from field and laboratory tests and utilizing Equations 1 and 3. the value of q_{utu} = 75.27 t/m² and q_{utu} 86.44 t/m² were found. Since Ap is the area for the installed pile equal to 0.096 m² for a 0.35m diameter, the ultimate load capacity of single pile was found to be 17.44 tonnes. Further recognising the contribution of collective skirting and using a factor of safety of 2.5, the safe load capacity of a single pile was found to be 21.8 tonnes and hence the total number of piles required for supporting 5,000 tonnes safely was worked out as 230 numbers.

SETTLEMENT PREDICTION

The settlement of the distressed tank foundation was predicted from the method based on Equivalent Coefficient of Volume Compressibility (Rao, 1982; Rao & Ranjan, 1985, 1988), besides empirical approaches (Skempton; 1953 and Vesic, 1977 utilizing insitu load tests on granular piles.

EQUIVALENT COEFFICIENT OF VOLUME COMPRESSIBILITY CONCEPT

Utilizing intensity of stress q, equal to 9.53 t/m² corresponding to a total load of 5000 tonnes and other input data such as total cross sectional area A_p = 22 m², total number of piles =230, total foundation base area A at a depth of 1.2m below the pad, the modulus of the soil and the pile material, E_s = 800 t/m² and E_p = 2300 t/m² the replacement ratio α = 0.0419 and base area A of tank foundation = 524.55 m², the average N_s-SPT value = 8 number of blows and equivalent modulus of composite mass E_{eq} = 8629 t/m² and using equations 10 to 13, the total predicated settlement was found to be 87 mm. The settlement reduction ratio (β) was found to be 91.5%. Here it may be noted that the depth of sub-soil layers below the tank foundation equal to width of foundation = 24m was considered in the settlement computation. The settlement of the tank foundation reinforced with granular piles was also predicted utilizing stress-deformation curves obtained from insitu load test (Fig. 79) and empirical methods proposed by Skempton (1953) and Vesic (1977). These methods were basically proposed for pile foundations (Eq.51 and 52).

$$S = S_p\,[(4B+3)/(B+4)]^2 \quad ....(51)$$

$$S = S_p(B/B_p)^{0.5} \quad ....(52)$$

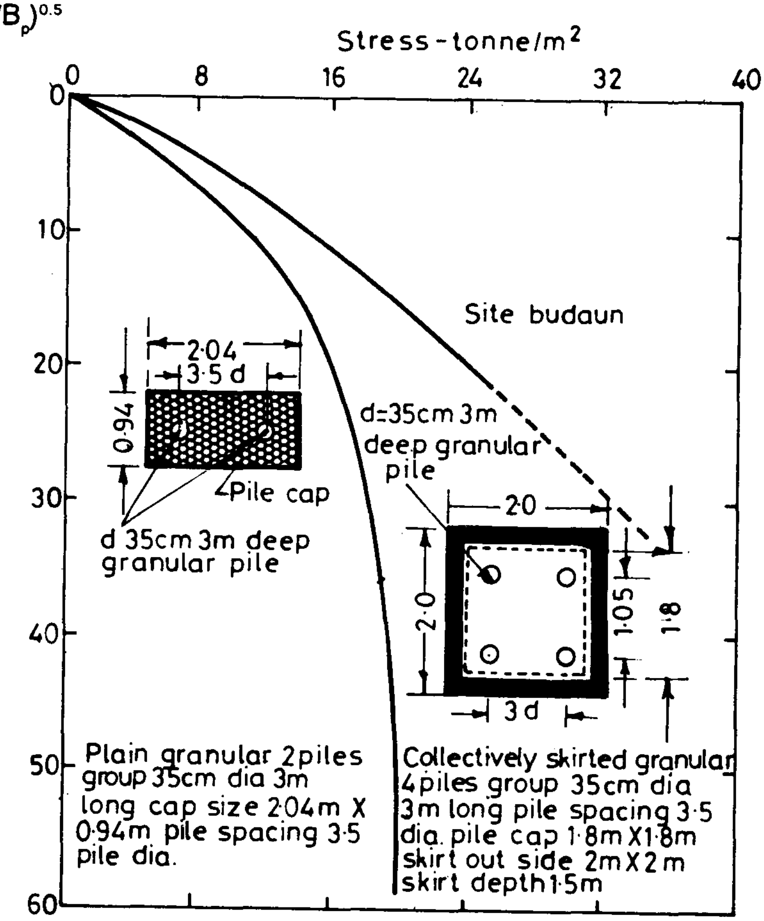


Fig 79 Behaviour of Plain / Skirted Granular Piles Under Load

Where S is the settlement and B is the effective width of foundation reinforced with 230 piles, S_p is the settlement of the test pile group having pile cap width equal to B_p. The input data & computed settlement from Eqs. 51 and 52 have been presented in Table 25 along with Rao (1982) values.

INSITU LOAD TESTS

For the verification of the design assumptions of the foundation subsoil reinforced with granular piles, two groups of piles were installed. In the first group there were two granular piles and in the second there were four piles with a collective skirt. The stress deformation behaviour of these along with other details have been presented in Fig.79. It may be noted that the settlement against the design load of 11 t/m² was found to be 10 mm only. It is further noted from Table. 18 that the predicted settlement from Rao (1982) approach, when compared with two empirical methods based on load tests (Skempton, 1953 and Vesic, 1977) on skirted four pile group, vary between a range of 25.4 mm to 87.0 mm only which is well withing permissible limits (IS : 1904-1978). These settlements were expected to be completed during water load testing itself, being cohesionless soil deposit reinforced with granular piles.

TABLE 18

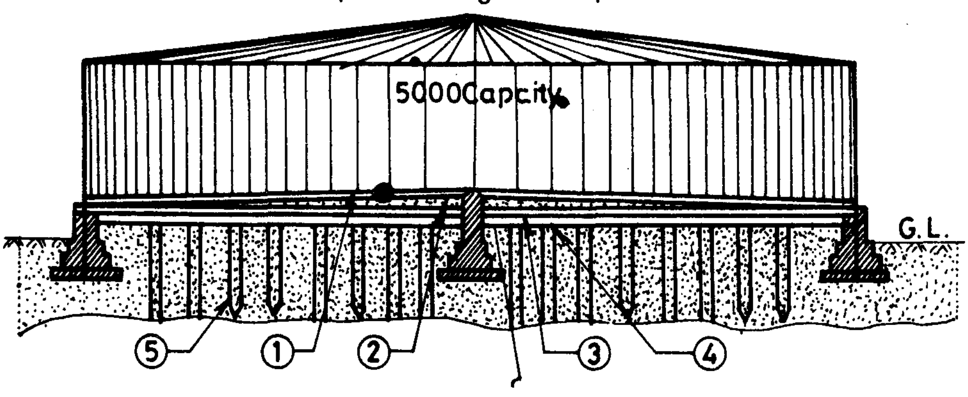
PREDICTED SETTLEMENT OF THE TANK FOUNDATION AFTER STRENGTHENING

	Settlement (MM) utilizing		
Method of Computation	**2 Plain pile group Load test data**	**4 skirted pile group Load test data**	**Equivalent Coefficient of Vol. Compressibility and soil pile properties**
Skempton (1953)	114.30	63.50	-
Vesic (1977)	45.72*	25.40**	-
Rao (1982)	-	-	87.0
Rao & Ranjan (1985)			

Effective width of tank B=25.85m

*Width of pile cap B_p = 0.94m and **B_p = 1.8m and Design load = 9.53 t/m²

Hence no further settlement was anticipated during service period of the tank.



1. 2·5mm sand bitumen
2. Sand filling
3. Brick on edge soling
4. Used as skirt
5. Granular piles

Fig 80 Distressed Tank Foundation Reinforced with Granular Piles

INSTALLATION OF GRANULAR PILES

The granular Piles were installed by simple auger method developed by (Rao Bhandari & Sharma, 1979 and Rao, 1982), in a triangular pattern having spacing equal to 3.5 times the pile diameter. The base plate was punctured by welding at 230 locations as per the layout and auger bore having 30 cm diameter were made. Stone aggregates 20-70mm grading were placed in layers of 30 cm with 15% of clean sand. These layer were compacted using a 125 kg internal operating hammer. The installed pile diameter was found to be as 35 cm. based on the volume of material consumed the bore hole. Subsequently the pile tops were covered by welding steel plates carefully and making the steel tank base water tight (Fig.80).

CONCLUDING REMARKS

Subsequent to the granular pile installation, the water loading test was carried out with a view to check any possible leakage in the steel tank under the full design load, without any settlement. From design predictions no settlement was anticipated during service period of tank. The tank has since been under continuous service since 1980 without any sign of distress due to settlement or any other reasons.

SEARCH FOR INNOVATIVE FOUNDATION TECHNOLOGIES

INTRODUCTION

In the earlier part of the paper the Behavioural prediction and performance of structures of composite sub-soil strata reinforced with granular piles provided with or without a rigid skirting have been demonstrated through full scale insitu load testing on prototype foundations. The study did not conclude at that stage itself and felt satisfied by publishing papers in national and international, journals/conferences symposium. However, efforts were aimed to convince the designers and practising engineers for adopting the new technology in their design as an efficient and cost effective alternatives to existing foundation practices. To accomplish the task, the amount of efforts needed can only be realised and need not be expresset at this occasion. The amount of success in such attempts, is naturally linked with the superiority of the technique, you are selling over other alternatives. This is fully demonstrated by record of performance of few selected case studies out of many, on varities of structures, such as low to high rise buildings, small to large diameter storange tanks oil drilling rigs and underground power houses, besides strengthening of foundations of distressed molasses tank and residential shopping complex.

In author's view, it is not always true that for all problems, only one remedy is the only solution, but several alternatives should be tried and the best is recommended. It is with this view in this part of the paper, attention is chiefly focussed on Innovations in Foundation techniques which is in author's view merit designers preference in most situations, not because of greater latitude in terms of choice but also because of better assurance of performance with due regard to economy in design. Therefore the techniques which need favourable considerations particularly in situations where difficult sub-soil conditions, speed of construction and time, availability of equipment and working space besides cost effectiveness are constraints. The following alternatives deserve favourable consideration.

- Geo Pad
- Self setting soil slurry piles with geofabric reinforced cap
- Soil nailing
- Mini grouted piles
- Socketed Mini grouted piles
- Spliced piles

SELF SETTING SOIL SLURRY PILES

Thixotropic self setting slurries have been used by irrigation engineers in construction of diaphragm walls. Experiments have revealed that the self setting slurry yields high compressive strength without showing much of anisotropy. (Jain et al 1990; Rao et al 1990, 1991, 1992). The strength of such slurries on hardening could increase further when reinforced with organic fibers (Bhandari & Rao, 1988). The author would therefore recommend the above new concept for bored piling replacing the reinforcement and the cement concrete totally by self setting slurry. Such cluster of piles could be provided with geogrid reinforced pile caps to facilitate transfer of loads to ambient ground (Rao, 1990). Slurries placed in bore holes harden in less than a weak and could carry considerable amount of loads. In fact because of lesser modulus of such piles as compared to concrete piles, possibilities of negative drag may also reduce due to better strain compatibility between such piles and the ambient compressible strata.

In practice the process of installing self setting soil slurry pile is effected by drilling a hole in the ground having weak sub-soil using manually operated auger or a pile drilling rig, which is to be improved or transformed in to composite ground. The rate of boring in the weak soil porotion ranges from 0.4 - 0.5 m/hours. The technique effectively eliminates vibration and noise nuisance. This is followed by lowering 3 to 4 mm. thick mild steel casing as steel linear progressively in appropriate length. The drilling fluid is continuously circulated which brings out the cuttings of the soil. Depending upon the sub-soil condition the casing may be replaced by bentonite slurry.

Most of the available soils when mixed with cement, flyash and bentonite produces a new material which qualifies for use in the place of reinforced cement concrete particularly in the construction of bored piles and pad foundations. The basic requirement of such a material is the low order permeability (in the range of 10^{-6} cm/sec-10^{-8} cm/sec), strengh not less than that of the weak soil in which the foundation is laid, resilience to withsatnd without cracking, strain due to sub-soil deformation and resistance to erosion by passage of water through the ground. Test results presented in Table 19 show that out of four categories, soil from location 1 & 2 indicate to be clay with low and medium plasticity whereas the sample from location 3 is silty sand and that of 4 is poorly graded sand SP.

The study therefore indicate the ideal soil slurry from strength and workability point of view could be obtained with a soil having clay 5-10% silt 60 to 70% and sand 20-30% (Jain et al 1990).

Following the procedure described earlier single and in groups of self setting soil slurry piles were cast at site 1 along with Soil Slurry Pile Caps reinforced with Netlon gird. The details of the pile size, depth spacing etc. have been provided in Table 20. The single and groups of piles were in situ tested upto their ultimate capacity in accordance with IS : 2911 Pt IV - 1985.

The stress deformation behaviour of weak soil deposit reinforced with a single 150 mm diameter, 4 m deep self setting soil slurry pile reinforced with and without coconut fibres, with a goefabric reinforced cap is presented in Fig. 81, which clearly indicate that reinforcing of self setting soil slurry piles with coconut fibre does not influence the load settlement behaviour of reinforced weak cohesionless soil. Further study of the same curve (fig. 81) over which the stress deformation behaviour of the composite soil reinforced with minigrouted pile with

Table 19 (a)

SOIL CLASSIFICATION (RAO ET. AL, 1990)

Location	Soil Classification	Grain size Analysis (%)			Atterberg's Limits		
		Sand	Silt	Clay	WL	WP	IP
1.	CL	6	62	32	55	27	28
2.	ML	20	70	10	37	26	11
3.	SM	90	10	-	-	-	-
4.	SP	99	1	-	-	-	-

Table 20

DETAILS OF THE FULL SCALE TESTS AT ST. GABRIEL SITE

Type of pile	Details of pile			
	Dia (mm)	Depth (m)	S/D	*Geofabric Reinforced Pile Cap Size (cm)
(i) Single pile	150	4	-	45 x 45 x 30
(ii) Single pile	150	4	-	45 x 45 x 30
(iii) 4-Pile group	150	4	2.5	100 x 100 x 30
(iv) 4-Pile group	150	4	3.0	100 x 100 x 30
(v) 4-Pile group	150	2	3.0	100 x 100 x 30
(vi)4-Pile group	200	4	3.0	100 x 100 x 30
(vii) 10-pile group	150	4	3.0	60 x 60 x 30

pile cap resting on ground has been superimposed, indicating exactly similar behaviour as the self setting soil slurry piles with and without geofabric reinforced pile cap, showing that self setting soil slurry may be used as a replacement of reinforcement cement concrete in bored piling. Some typical test results from site 1, depicting the influence of depth of reinforcing and also the effect of reinforcing around a foundation footing overlying a group of three soil slurry have been presented in figs. 82 to 84. The test results are self explanatory. Similar behaviour were observed in soft saturated clay deposit (site III) also.

The other innovative Foundation Techniques such as Geo Pad, Soil Nailing, Mini grouted Piles, Socked Mini grouted piles and spliced piles have been discussed in detail else where (Rao 1993 : Ind. Geot. Journal (23), 1, January.).

ACKNOWLEDGEMENT

The work reported in this lecture is based on the author's involvement in research and development activities related to ground improvement and foundations in difficult soil condition, over two and a half decades. The outcome of research has successfully been utilised as cost effective and efficient foundations for several prestigious structures out of which seven case records with their feedback studies have been presented. The content of this lecture is conditioned by the large number of publications, personnel communications the author

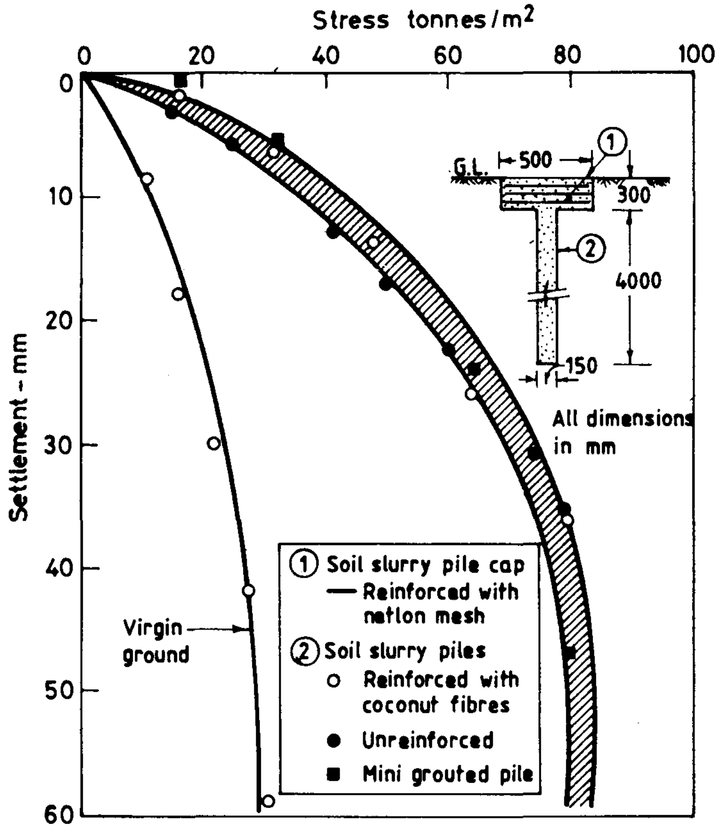


Fig. 81 Stress-deformation behaviour of soil slurry piles with geofabric reinforced pile cap & minigrouted pile

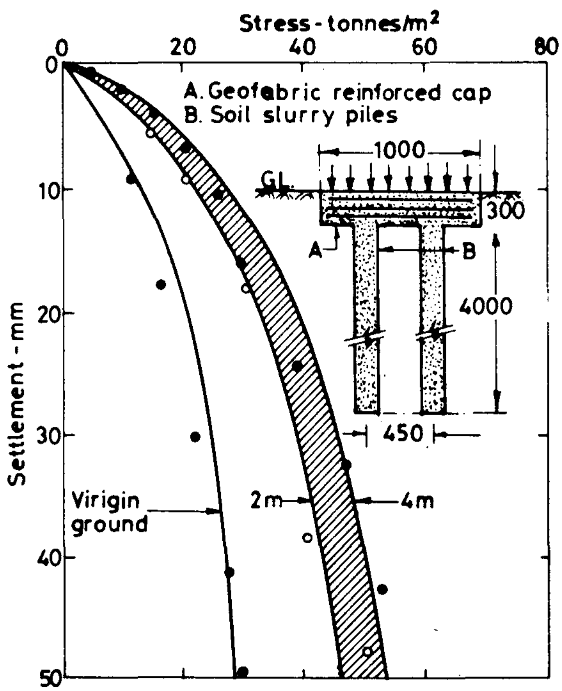


Fig 82 INFLUENCE OF DEPTH OF REINFORCEMENT ON STRESS/DEFORMATION BEHAVIOUR OF COMPOSITE GROUND

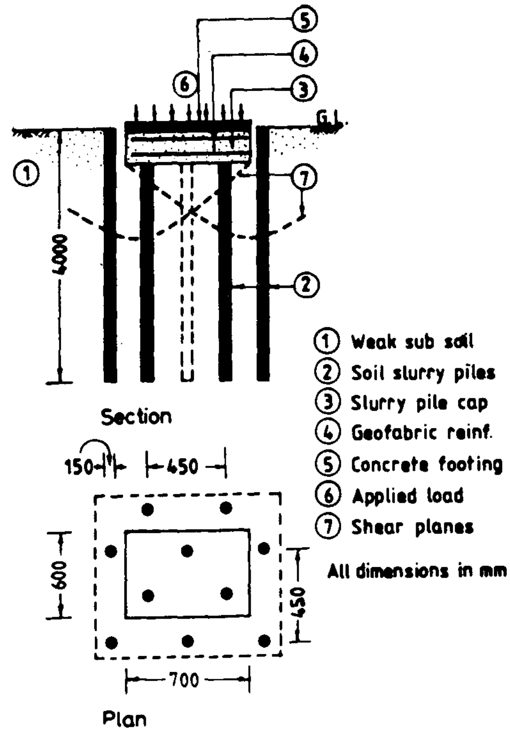


Fig 83 Influence of reinforcing the weak soil immediately below & around the footing by soil slurry piles

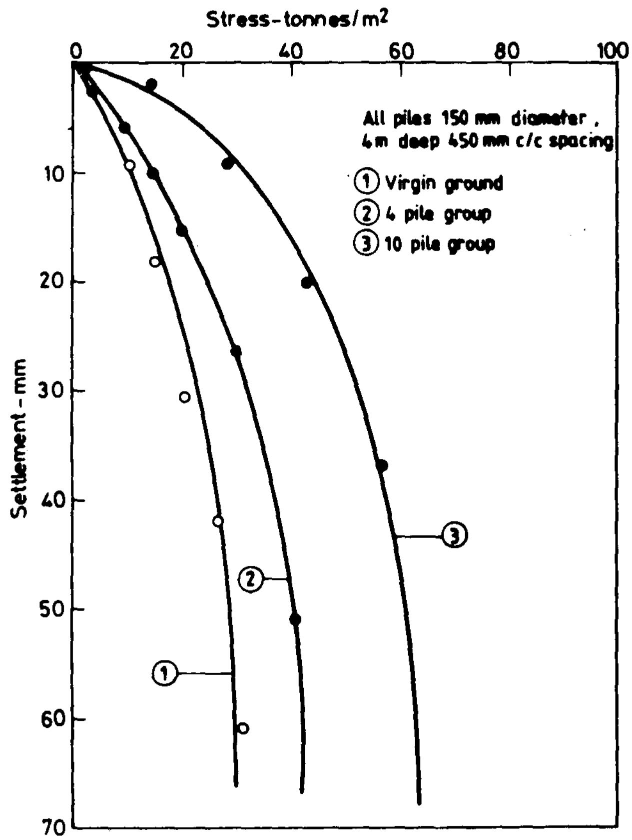


Fig 84 Stress deformation behaviour of weak soil treated with groups of self setting soil slurry piles

had with distinguished researchers on the subject, both in India and abroad.

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