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BEHAVIOUR OF SKIRTED GRANULAR PILE FOUNDATION

A THESIS

submitted in the fulfilment of
the requirements for the award of the degree
of

DOCTOR OF PHILOSOPHY

in

CIVIL ENGINEERING

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ABSTRACT

Designing safe and economical foundations in weak sub-soil stratum is a difficult engineering problem. The problem becomes more complicated when the design loads are high and corresponding settlements are restricted. The common practice is to improve the capacity of ground using drains with a combination of preload. Other means of ground improvements are use of lime piles, woven fabric reinforcement, compaction by explosives and falling heavy weights etc. Stone columns (granular piles) installed through vibrofloatation process or vibro-replacement methods also provide a suitable solution. However, the process necessitates the import of vibroflot and need trained personnel. Despite wide application of granular piles in soft cohesive soils the design methods are semi-empirical and conservative. Also their use in reinforcing loose to medium dense cohesionless soils is not so common.

In the present investigation, a rational analytical design approach using the anology of expansion of cylinderical cavity in an infinite soil mass expanded by uniformly distributed pressure has been modified to arrive at the ultimate load carrying capacity of single plain granular piles. The analysis is also extended to groups of piles individually or collectively skirted.

Utilizing the modulus of compressibility of composit soil mass consisting of granular piles and the virgin soil, method to compute the total settlement has also been developed. Further, analytical solutions have been developed to predict the ultimate uplift capacity of single granular pile when the pull load is applied at the pile toe. This has also been extended for pile groups.

A simple, and speedy construction technique of granular pile based on indigeneous knowhow, of different diameters and depths had also been developed. Using this technique, granular piles have been constructed at five different sites. These have then been load tested.

Full scale in-situ load tests have been carried out at 5 sites, on plain and skirted granular piles (individually and collectively), single and in groups of 2, 3 and 4 with varying diameters of 25cm, 35cm having 3.5m and 8m lengths in loose-medium dense cohesionless soil deposits. On other two sites having clayey silt and soft clay deposits, piles of 60 and 55cm with 9m and 14m lengths have been constructed. Also, collectively skirted 4-pile groups with central unit piles have also been load tested. Tests with skirts made of different materials ranging from r.c.c. to timber piles have been carried out with a view to study their application in actual practice.

Full scale pull out load tests on plain granular piles 25cm diameter, 3.5m deep, single and in groups of 2, 3 and 4 granular pile group with a unit pile cast at the centre have also been carried out. Further horizontal load tests on full scale skirted 4-granular pile group with unit pile cast in the centre has also been carried out.

Based on Analytical and Experimental investigations the following conclusions have been drawn:

(i) The ultimate load carrying capacity of weak sub soil stratum reinforced with plain granular pile under compressive loads, increases considerably and a significant reduction in settlement is noted. The increase in ultimate load and reduction in settlement is further enhanced with the pile group are skirted collectively or individually.

(ii) Also ultimate up-lift capacity of plain granular pile groups are noted to be high with much smaller pull movements.

(iii) Though reinforced cement concrete skirting is found to be best however, other type of skirts using different materials have also shown promise provided, these are rigid enough to confine the reinforced soil plug under applied load.

(iv) The observed ultimate loads and deformations have shown a reasonably good agreement with the predicted values based on proposed analytical approaches.

(v) A 3000 cu.m. capacity molasses tank founded on skirted granular pile group collectively skirted have corresponded well under full water load maintained for 15 days during water load test. The tank since then is in use.

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LIST OF SYMBOLS

Following symbols are generally adopted unless otherwise stated in the test

Symbol	Description
A	Area of the footing/pile cap
A_c	Pile Cross Sectional Area
A_p	Cross Sectional Area of the Granular Pile
A_s	Surface Area of the Pile
A_{sb}	Shaft Area of the Unit Pile
A_{sk}	Area of the interface of the skirt
B	Width of the footing/Pile cap
c_u	Undrained Shear Strength of Clay
D	Depth of the Pile
D_f	Foundation Depth
D_S	Depth of Skirt
D_T	Trench Width
D_r	Relative Density of the non cohesive sub-soil deposit
d_e	Diameter of the Influence Zone of the Pile
E'	Drained Modulus
EI	Modulus of Elasticity and Momentum of Inertia respectively
E_p	Modulus of Deformation of the Granular Pile Material
E_s	Elastic Soil Modulus
$E_{p'}$	Corrected Pile Modulus
E'_s	Corrected Soil Modulus
E_{eq}	Equivalent Elastic Modulus
F	Factor of Safety
F'_c & F'_q	Dimensionless Cavity Expansion Factors
f_s	The Unit Skin Friction

Symbol	Description
G	Shear Modulus
H_c	Critical Height of the Pile
H	Depth of Granular Pile
H_e	Effective Height of the Pile
H_n	Thickness of the nth layer
$H_1, H_2, H_3 .. H_n$	Thicknesses of Various Layers of Un-reinforced soil below Granular Pile Tip
$h_1, h_2 ... h_n$	Thicknesses of Various Layers of Reinforced soil below Skirt Base
$h_{s1}, h_{s2} ... h_{sn}$	Thicknesses of Various Layers of Reinforced soil plug
I_r	Rigidity Index
I'_{rr}	Reduced Rigidity Index
K_o	Coefficient of lateral Earth Pressure
K_p	Coefficient of Passive Pressure for the Pile Material
K_s	Co-efficient of Earth Pressure (lies between K_o and K_p)
K_u	Uplift Coefficient
K'_b	Earth Pressure Coefficient
K'_s	Average Earth Pressure Coefficient
K	Constant (as defined by Hughes & Withers 1974)
L	Length of Granular Pile
L_c	Critical Length
L_f	Length of the Footing
N_S	No. of blows for 30cm Penetration in SPT
N_{SPT}	Standard Penetration Test Value
N_c	Average Dynamic Cone Penetration
n	Numbers of Piles in a Group
n_h	Coefficient of Horizontal sub grade reaction
Q	Total applied load

Symbol	Description
Q_{allow}	Allowable Pile Load
Q_{hg}	Lateral load for a Specified Lateral Displacement
Q_L	Ultimate Uplift Capacity of Unit Pile
Q_o	Ultimate Capacity of Granular Pile in Compression
Q_p	Ultimate Uplift Capacity of Plain Granular Pile
Q_s	Total Skin Friction Between Soil Plug and Skirt
Q_{safe}	Safe Pile Capacity
Q_{sk}	Ultimate Load of Skirted Granular Pile
Q_u	Total Ultimate Uplift Load of Skirted Granular Pile
Q_g	Total Ultimate Uplift Load of Granular Pile Group
q	Surcharge Load per Unit Area
q_1	Stress on Granular Pile
q_2	Stress on Surrounding Clay
q_b	Unit Stress at the Base of Skirt
q_c	Static Cone Penetration Resistance
q_p	Unit Stress Shared by Granular Piles
q_s	Unit Stress Shared by Surrounding Soil
$q_1, q_2, q \ldots, q_n$	Applied Stress in Different Layers in Soil Plug
q_{allow}	Allowable Axial Load
q_{ult}	Ultimate Load Bearing Capacity of the Granular Pile
$(qult)_1$	Ultimate Bearing Capacity of the Granular Pile Below the Plane YY
$(qult)_2$	Ultimate Bearing Capacity of Skirted Footing above the Plane YY
R_p	Radius of Cylindrical Zone of Plastic Equilibrium
R_u	Radius of Cylindrical Zone around the Bulged Pile
S	Total Vertical Settlement
$S_1, S_2, S_3, \ldots S_n$	Settlement in Different Layers
U_o	Initial Excess Hydro-Static more water pressure

Symbol	Description
V	Volume in Deformed State
V_o	Initial Volume in Deformed State
V_F	Initial Velocity
W_F	Weight of the Footing
W_L	Weight of the Unit Pile
W_p	Weight of the Granular Pile
Z	Depth of Compressible Layer
α	Relative Pile Area
α_F	Almansi Strain
$\gamma_{(bulk)}$	Unit Weight of the Soil
$\gamma_{(Sub)}$	Submerge Unit Weight
δ	Permissible Settlement
δ_1	Length of the Element in Deformed State
δ_{1o}	Initial Length of Small Linear Element
μ	Poisson's Ratio
μ'_s	Drained Poisson's Ratio
Δ	Average Volume Strain
Δ_1	Settlement of the Reinforced Clay
Δ_2	Settlement of the Virgin Clay
Δ_S	Settlement in the Skirted Reinforced Soil Plug
Δ_L	Settlement in Reinforced Soil
Δ_H	Settlement in Unreinforced Soil below Granular Pile Tip
$\Delta_{S'}$	Settlement in Reinforced Soil Plug without any Skirting
Δ_V	Change in Volume
θ	Central Angle
ϕ_1	Effective Angle of Shearing Resistance of Pile Material
ϕ_2	Angle of Friction

Symbol	Description
$\epsilon_1, \epsilon_2, \epsilon_3$	Strains in Three Principal Planes
σ_1	Effective Normal Stress
σ	Inc ease in Vertical Stress
σ_{ho}	Initial Lateral Ground Stress
σ_m	Effective Mean Normal Stress
$\sigma_{m'}$	Increased Effective Mean Normal Stress
σ_{rL}	Lateral Limit Stress
σ_v	Vertical Effective Normal Stress
σ_{vz}	Vertical Effective Normal Stress at Depth given by L/D ratio
σ_o	Initial Ground Stress
η	Wedge angle
ρ	Density of Pile Material
ξ	Wedge Angle

CHAPTER I

INTRODUCTION

1.1 General

On many occasions, Geotechnical Engineers are confronted with the problem of designing safe and economical foundations, placed in difficult soil conditions for multitude of civil engineering structures, for example, loose saturated sand deposits with considerable depth and with widely varying densities, coastal areas with soft saturated clay deposits of considerable depths. Also, non-uniform sub-soil deposits consisting of a hard soil crust with limited depth overlaid a weak soil stratum with considerable depth or vice-versa are situations which call call for the attention of the practicing engineers. Similarly, presence of thin layers of sand lenses in soft clay deposits or layers of soft clay deposits with organic matters sandwitched between non cohesive deposits need careful handling. Under these situations, low bearing capacity and excessive total and/or differential settlements warrant especial design treatment. The situation becomes further complicated when the ground water level is high and calls for dewatering operations during construction, excavations leading to significant increase in the cost of the foundation.

It is possible to circumvent with the problem of increasing the bearing capacity and reducing the settlement of soft clays by resorting to different available methods developed in the recent past (Broms, 1979). The applications of these methods depend upon shear strength, compressibility, permeability, depth of the deposit and thickness of the compressible layer, allowable total and differential settlements, environmental factors, economics, available time, equipment

and local experience. Development of sand drain and using it as a method of stabilisation (Moran 1926) was the first step towards improving the strength deformation behaviour of soft soils. The application of sand drains to live problems followed with a combination of preload (Proctor 1936, Moran et.al 1958, Casagrande and Poulos, 1969, Nonveiller 1976, Chalmers and Harris 1981). Sand wicks (Dastidar et.al 1969) rope drains (Mohan et.al 1977) have also been used with preloading as these are not smeared easily as sand drains. Recent developments on the new types of drains such as geo-drains, ali-drains and Colbond etc. and their application to live problems have been highlighted by Broms (1979), Burke and Smucha (1981). The geodrains and ali-drains both have a plastic core which is surrounded by a pervious paper which serves as a filter and prevents clogging of grooves or channels in the plastic core. The width of the drains is 100mm which is the same as those of card board wicks developed by Kjelmann (1948). The functions of these drains have been described elsewhere (Broms 1973, Hansbo & Torstensson 1977, Fukuoka, 1977). These drains are used to reduce the consolidation time of soft clays. The main advantage associated with these drains is that the disturbance caused by installation is small as compared with sand drains due to small size of the drains and efficiency of the drain is not affected by large settlements. However, it is found that the effectiveness of 100mm geodrain is some what less than that of 16cm sand drains (Errikson and Ekstrom 1975). The main disadvantage with these drains is the durability of filter paper around the plastic core which mainly depend upon the bacteriological activity in the sub-soil stratum and ground water level. The maximum life of the filter paper is one to one and a half years under normal conditions (Hansbo and Torstensson 1977).

To increase the shear strength and reduce the compressibility of soft clays, unslaked lime mixed insitu with soft clays to form lime piles have been investigated near Stockholm (Broms & Boman 1975). These piles have also been used in Japan in harbour construction to a depth of 20-60m (Fukuoka 1977). In central part of Sweden lime columns have been used for light structures (Helmet et.al 1981). The observation of lime piles functioning as drains has been substantiated by both field and laboratory experiments. Electro-osmosis to stabilise deep soft deposits has successfully been used (Bjerrum et.al 1957), Chappel and Bruton, 1975, Casagrande et.al 1981. Chemical stabiliaation through electro-injection and salt wells developed by Moum (1968) has not generally been effective except where the soil is not stratified (Broms 1979). Thermal stabilisation (Litvinov, 1960) and freezing of soft clays at low temperatures (Sanger, 1968), electro kinetic stabilisation (Caron, 1971) have also been used. However, these methods have limited applications.

Dynamic consolidation where heavy weights (10-200 tonnes) are dropped from a height of 10-40m has also been used to stabilise soft clays, silts and organic soils when the remoulded soil consolidates (Menard 1959, 1972, West and Solocombe 1976). Several cases of applications both in cohesive and non cohesive soils where dynamic compaction has been used successfully have been discussed by Menard and Broise (1975), Hansbo et.al (1973). One of the major limitation of the method is that it is uneconomical when the area for treatment is less than 1500 sq.m. and it calls for improving the bearing capacity of the area before stabilisation can begin in order to support the heavy tamping machine (60-200 tonnes). The effectiveness of the method in improving the

load carrying capacity of soil at various sites have also been examined by Charles et.al (1981). Stabilisation of soft clays by blasting with the use of explosives have been used by Long and George (1967) and has been successfully tried in USSR (Mueller, 1971). However, the cost of the method is high in comparision to other methods. Fabric reinforcement using woven fabrics have been used for road embankments in Sweden (Broms, 1977). Reinforcing in-situ soil by regular arry of metal rods known as 'Soil Nailing' technique has been developed to be a safe and economic method of soil stabilisation (Glassler and Gudehus, 1981).

Methods of preloading by using earth fills or water loading are commonly used for soil improvements. Rock spoils are commonly deployed for this purpose in Sweden (Broms 1979, Colleselli et.al 1981). Lowering of water table has also been used as preload provided the clay layer is overlaid on a pervious layer like sand lenses and water table is upto the ground surface.

Vibro-replacement methods are also used to improve cohesive soils by deep vibratory methods (Engelhardt and Krisch 1977). The equipment used for this method is the same as that for vibro-compaction method. The hole left by the vibrator in soft cohesive soil is slightly larger than the diameter of the vibrofloat. This borehole is filled with imported coarse gravel 12mm-75mm in size, in layers, vibrated and compacted under the tip. Repetetion of this method forms a stone column or granular piles. In the case of non cohesive loose-medium dense deposits recourse has always been taken to improve the density of the sub-soil stratum by driven piles besides dynamic compaction methods, use of explosives and preloading techniques as discussed earlier.

One of the important methods of improving the strength and reducing the settlement in non-cohesive soil deposits in loose state is by using deep vibratory techniques known as vibro-compaction method (Engelhardt and Krisch 1977). Vibrofloatation, terra probe compaction piles and explosives, have recently been used in spain (Faraco, 1981) for the construction of new strip repair facility. The test results show superiority of all the methods except compaction piles.

The major draw backs with the available methods of ground improvements are that they (a) call for partial or full mechanisation and need special equipment (b) demand trained or skilled personnel at site and (c) are time consuming and (d) are un-economical. These factors induce the practicing engineers particularly in developing country like India to develop method which is economical technically viable and calls for exploiting easily available manual labour efficiently. The method should use indegeneously developed equipments. In addition the time required to complete the improvement of the ground should be minimum and avoid use of scarce material like cement and steel.

Granular piles (stone columns) provide a useful simple solution. These have been used as bi-product of vibro-floatation either singly or in small groups or else installed in a regular pattern to support large rafts or embankment in cohesive soil stratum (Thorbaurn and McViear 1968, Baumann and Bauer 1974, Rathegeb and Kutzner, 1975, Datye and Nagaraju 1975, 1977). For improving the settlement behaviour under static vertical loads, the improved performance of combined mass of soil-granular pile system under seismic loading conditions have also been discussed (Engelhardt and Golding 1975). Despite wide application of granular piles in

soft cohesive soils, the design methods are semi empirical and conservative (Hughes and Withers, 1974, Tanimoto 1971). Their use in reinforcing cohesionless soils is not so common.

These facts clearly demonstrate that the improvement of strength-deformation behaviour of loose-medium dense sub-soil reinforced with granular piles (stone columns) stand promise. The preceding paragraph, thus suggest the need for study of the behaviour of granular pile in weak sub-soil stratum and also to develop a rational design approach. Therefore, in the present investigation the stress-deformation behaviour of granular piles subjected to sustained vertical uplift and lateral loads on single and in groups, under different soil conditions with and without skirt have been studied.

1.2 Scope of the Work

As indicated earlier the present study relates to the behaviour of granular piles installed in different soil conditions. Attempts have been made to develop analytical solutions for single and group of piles and verify the same through in-situ tests. The piles/pile groups under vertical uplift and lateral loads have also been considered in the study.

1.2.1 Analytical

The analytical solutions have been developed by modifying the anology of the expansion of a cylinderical cavity in an infinite soil mass expanded by uniformly distributed internal pressure. These have been modified to arrive at the ultimate load bearing capacity of a single granular pile for two conditions of loadings, (a) when the load is sustained by granular pile top only, termed

as non cap resting condition (b) when the load is shared between granular pile and the surrounding soil termed as cap-resting conditions.

The total settlement of skirted granular pile group in loose-medium dense non cohesive sub-soil deposits have also been investigated in the light of modulus of compressibility of the composite soil mass consisting of granular piles and the virgin soil.

Analytical solutions have also been developed by modifying anology of expansion of cylinderical cavity in an infinite soil mass to arrive at the ultimate uplift capacity of a single granular pile for the condition of loading when the uplift load is applied at the base of the pile. This has also been extended for pile groups collectively skirted with a central unit pile.

1.2.2 Experimental

In the experimental investigations various aspects of the problem have been studied dealing with:

(a) Development of a simple construction technique of granular piles based on indigenous know-how man power oriented technology avoiding the necessity of sophisticated machines and speedy construction method.

(b) Load bearing capacity of single granular pile with varying diameters and depth, without skirt based on in-situ load testing.

(c) Load bearing capacity of single granular pile with varying depth and diameters with collective skirting.

(d) Load testing of granular pile groups with varying dia. and length without skirting.

(e) Load testing of granular pile groups with varying dia. and depth collectively skirted.

(f) Load testing of granular pile group with load bearing element and collectively skirted.

(g) Tests using different materials of skirt. Tests have also been carried out using different material of skirts such as:

(i) Cast-in-situ cement concrete skirt reinforced with mild steel bars.

(ii) Cast-in-situ cement concrete skirt reinforced with 1.6mm thick mild steel sheet.

(iii) Precast brick pannel, sand witched between cast in-situ concrete.

(iv) Timber pile skirting held rigidily at the foundation level and

(v) Prefabricated twin pipe unit skirting of retrievable type.

Test results have been compared to bring out the suitability of a particular type of skirt depending upon the ease of construction, availability of local materials and load-settlement behaviour of granular piles under vertical loads.

(h) Pull out tests:

Pull out load tests have been carried out on plain and skirted granular pile groups with central unit to study the following:

(i) Uplift capacity of single and groups of 2,3,4 plain granular piles.

(ii) Uplift capacity of a unit pile (LBE) cast in the centre of a 4 pile group and collectively skirted.

(iii) Uplift capacity of skirted granular pile group and a centrally located unit pile when pile grpup and the unit pile both are subjected to uplift load.

(i) Lateral Load Tests

Lateral load tests have also been carried out on unit pile and skirted granular pile with central unit pile when the load is applied at the foundation level for:

(i) Unit pile cast in virgin soil.

(ii) Skirted Granular Pile with central unit pile.

(j) Skirted granular piles provided as foundation for the following structures:

(i) 3000 cu.m. capacity molasses steel tank foundation in cohesionless loose subsoil stratum having low bearing capacity.

(ii) Foundation for an oil drilling rig (F-400-4DH) in soft silty sub-soil stratum with high water table.

(iii) A 65000 cu.m. mandatory crude oil tank founded in soft clay deposit with high water table.

CHAPTER II

REVIEW OF LITERATURE

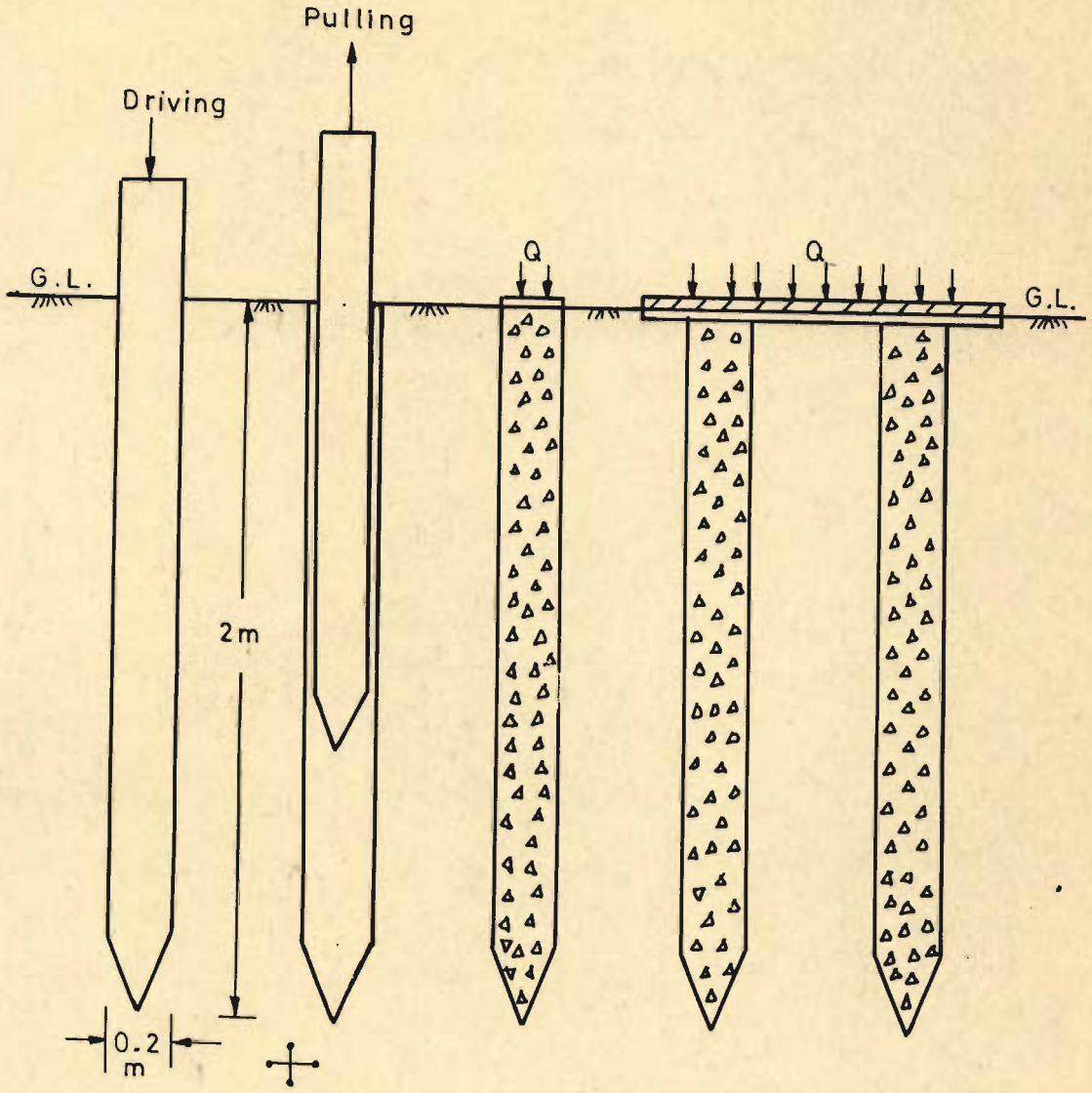
2.1 Introduction

The first attempt to use granular piles (stone columns) were made in France in 1830's (Hughes & Withers, 1974) to support heavy foundations of Iron works at Artillery Arsenal in Bayonne (France). The process of construction of these piles was to drive 20cm dia. 2m long wooden stakes into the ground and withdrawing them. The holes thus formed were filled with stone pieces Fig. 2.1. The load carrying capacity of these piles were taken as 10 KN with settlements reduced to one fourth.

The technique of improving weak sub-soil stratum was rediscovered in 1930 as a bi-product of vibrofloatation used for compacting coarse grained deposits. This procedure is reported to improve the ground by 100 per cent (Steuerman, 1939). Even so the vibrofloatation technique was used till 1960 for constructing granular piles (stone columns) in cohesive soil deposits and is known as vibro replacement method. To-day these can be made up to 20m (Broms, 1979) depth and 0.5m to 1.5m in diameter for a load of 300 KN (30 t) (Hughes and Withers, 1974).

Construction of granular piles in cohesive soil transform the original soil mass into a composit mass of densified granular pile interfaced with original soil. The final result is an overall increase in shear strength and improved stress-deformation behaviour of soil granular pile system.

The laboratory studies and full scale field tests and theoretical models developed by number of authors (Hughes et.al 1975, Baumann et.al 1974) indicate that there are several important factors controlling the load deformation behaviour of granular piles.



Driving of the wooden stake (a)

With drawal of the wooden stake (b)

Filling of the granular material in the hole (c)

Group of 4 piles under on isolated footing (d)

Design load of a single pile = 10 KN (1 Tonne)
Diameter of wooden stake = 20cm
Length = 2m

FIG.2.1 — FORM OF GRANULAR PILES IN 1830'S

These include shear strength of in-situ soil, radial pressure-deformation characteristics of soil, angle of internal friction of granular pile material and length to diameter ratio of pile and spacing in case of groups.

In this chapter the ultimate load carrying capacity of granular piles and methods of computing settlements based on (a) analytical approaches (b) empirical approaches and (c) experimental investigations have been briefly reviewed. Also, various construction practices adopted for granular pile have been reviewed. Several case records where granular piles have been used successfully in soft clays, silt have been discussed. Summery and critical comments have been offered on various aspects of granular piles.

2.2 Ultimate Load Carrying Capacity

Unlike reinforced cement concrete piles, Fig. 2.2(a) granular pile makes efficient use of soil surrounding these piles, Fig. 2.2(b) when a single granular pile is subjected to sustained vertical load, the pile bulges upto the critical length of the pile, thus fails to transmit the load to a deeper stratum. The ultimate load carrying capacity of a granular pile is a function of limit lateral stress provided by the surrounding soil and the angle of shearing resistance of the granular material constituting the pile besides the pile diameter. Thus the ultimate load bearing capacity of a granular pile is reached when the ratio of the applied load to the ratio of the lateral limit stress is maximum

2.2.1 Modes of Failure

In case of a rigid pile negligible lateral displacement is

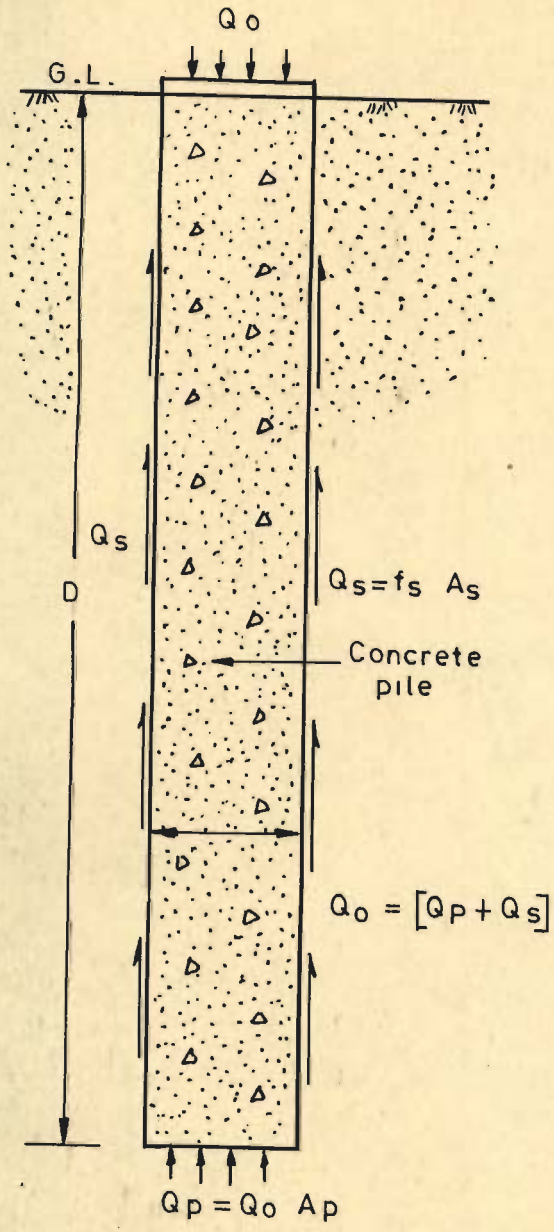


FIG.2.2(a).RIGID R.C.C.PILE

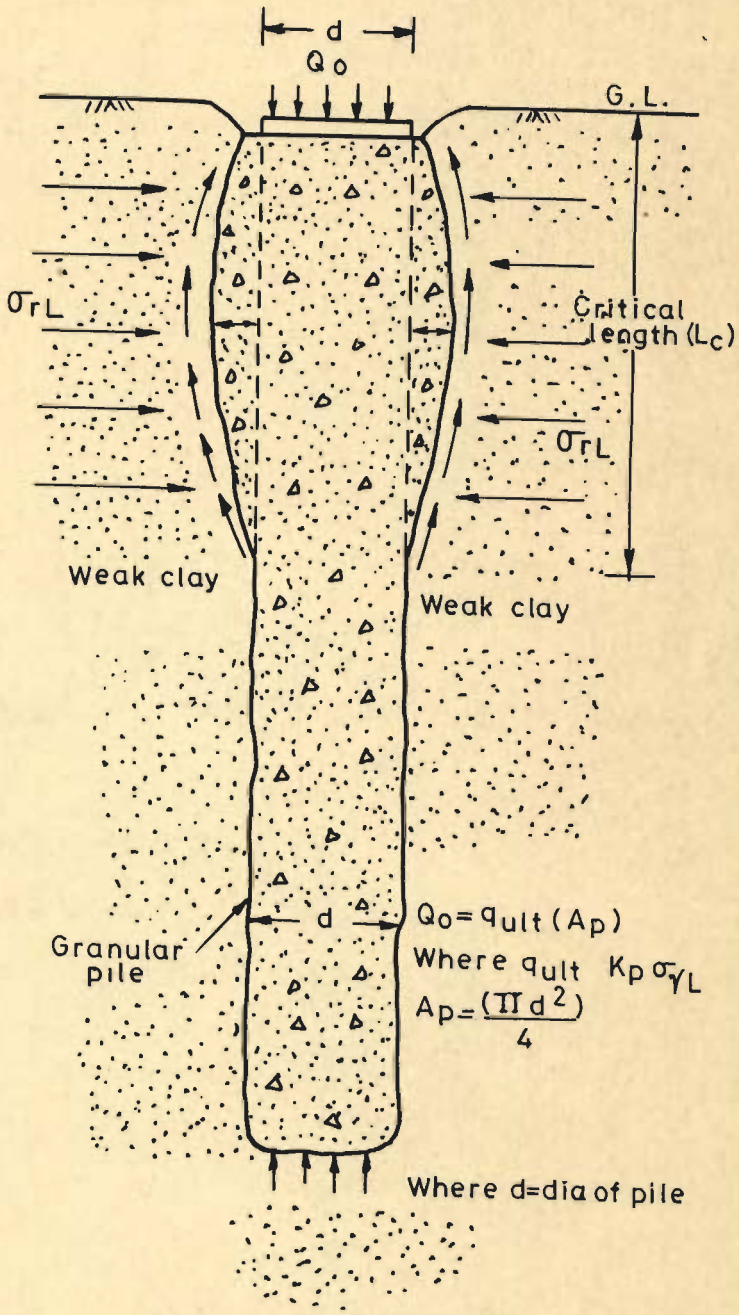


FIG.2.2(b).BULGING FAILURE MODE

postulated, Fig. 2.2(a) . However, in case of granular piles which are seldom as rigid as concrete piles, this type of failure is ruled out. The second mode of failure is the bulging failure Fig 2.2(b), observed in granular piles in soft clays (Hughes and Withers, 1974). The observed deformed shape of the granular pile in one of the model tests (Hughes and Withers, 1974) Fig.2.2(c) and also the actual pile excavated at Convey site. (Hughes et.al 1975) Fig. 2.2(d) suggest that failure shape in upper meter or two is often found to shape like a bucket resting on a cylinderical stem through lower level. The deformed shape of the piles in the laboratory and field were found to be geometrically similar. The observation suggests that the over consolidated dessicated crust was softened a little during pile installation. This was confirmed after excavation at Convey site. The lateral limit strrssσ_{rL} may be estimated by using pressuremeter tests Hughes et.al 1975, or Vesic's cavity expansion factors (Vesic 1972). The ultimate load carrying capacity q_{ult} of a granular pile may be found from the equation:

$$q_{ult} = K_p (\sigma'_{ho} + K C_u) \qquad \text{...(2.1a)}$$

$$= K_p \sigma_{rL} \qquad \text{...(2.1b)}$$

Where K_p is the coefficient of passive earth pressure

σ'_{ho} is the initial lateral ground stress,

K is a constant,

C_u is the undrained shear strength of clay and

σ_{rL} is the lateral limit stress.

The third mode of failure is postulated by Madhav and Vitkar (1978), Fig. 2.2(e) where in a two dimensional plain strain version of a granular pile is taken as a granular trench depending

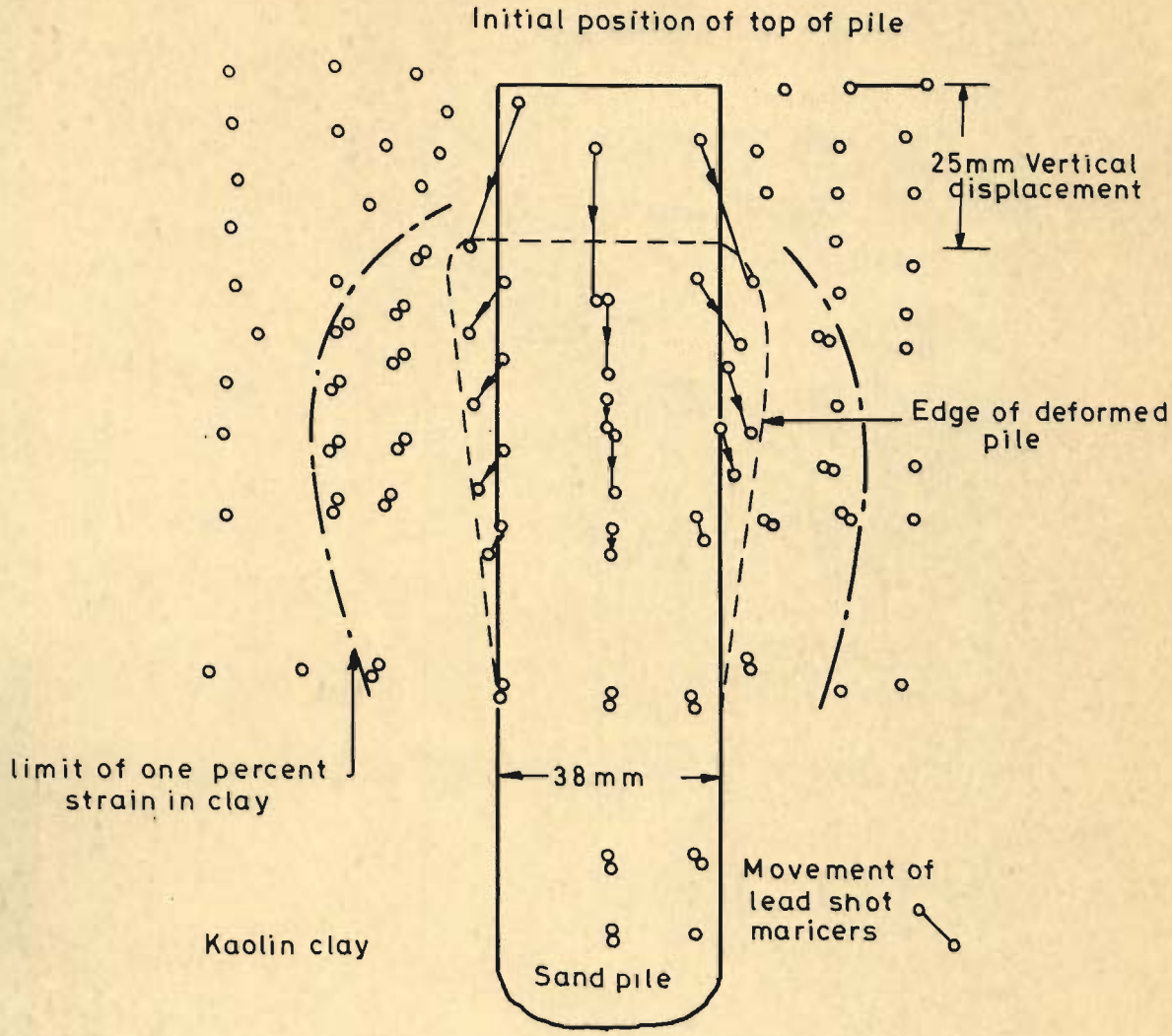


FIG.2.2(c). DEFORMED SHAPE OF A LABORATORY MODEL PILE (AFTER HUGHES AND WITHERS 1974)

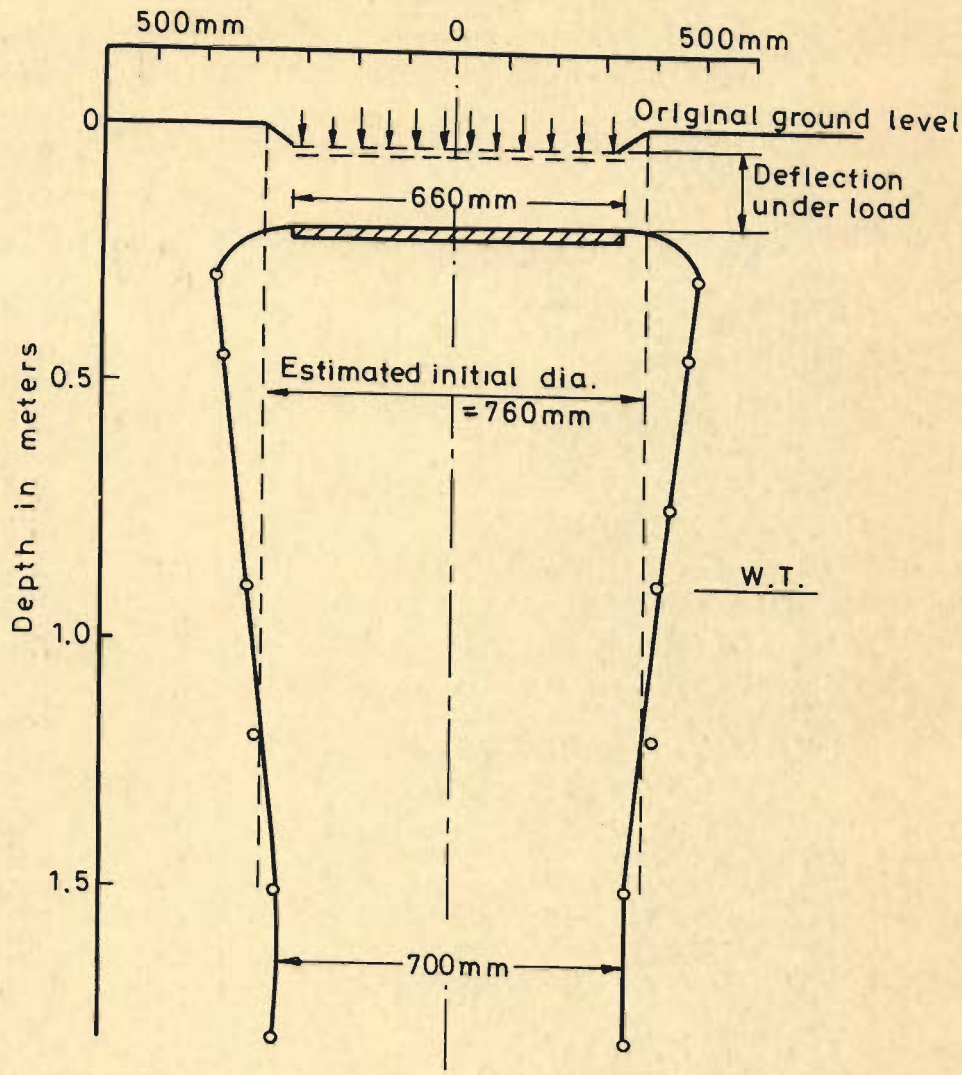


FIG.2.2(d) SHAPE OF A PILE AFTAR EXCAVATIONS
(AFTER HUGHES, WITHERS & GREENWOOD 1975)

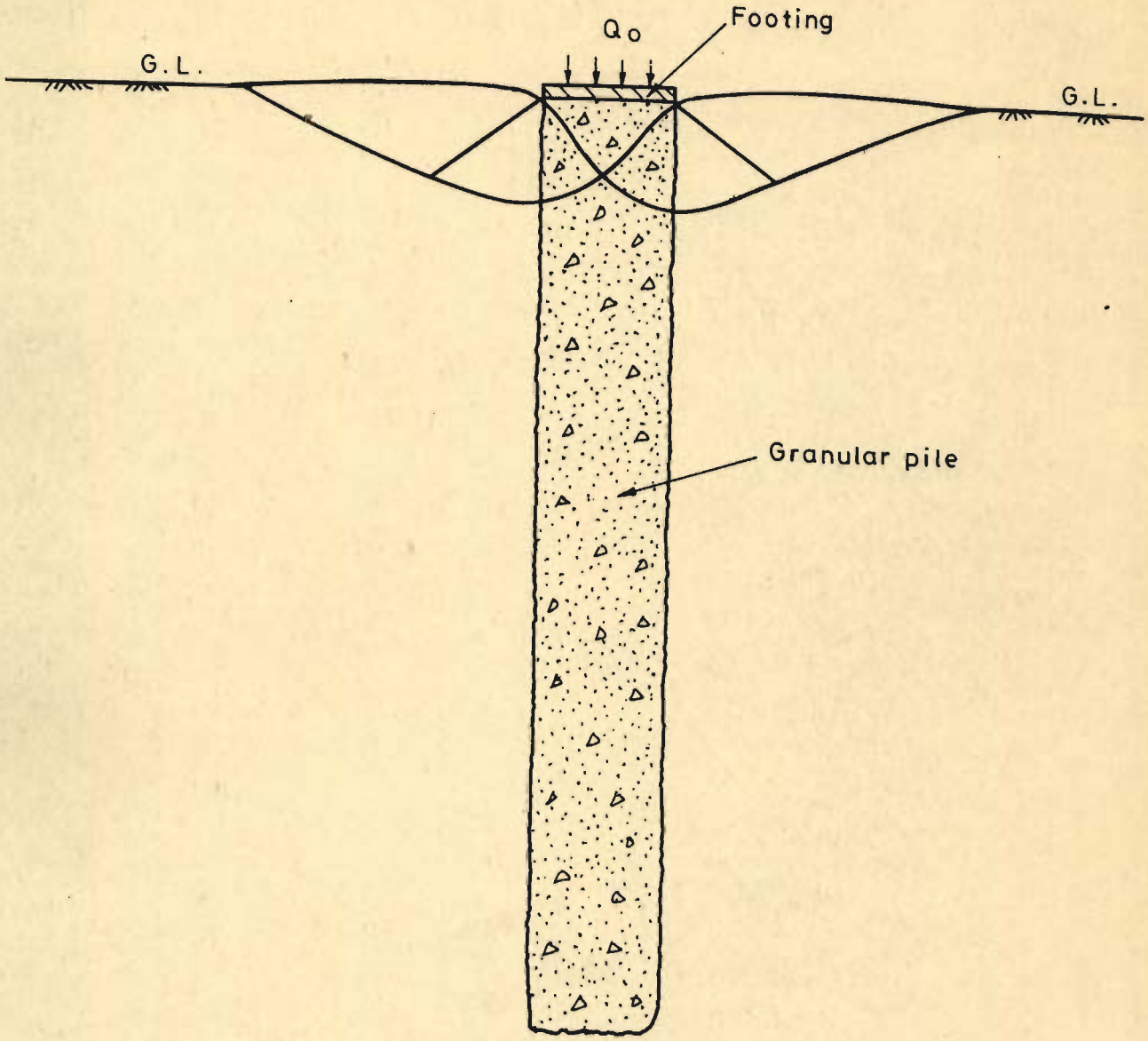


FIG.2.2.(e). GENERAL SHEAR FAILURE MODE
(AFTER MADHAV AND VITKAR 1978)

upon the mode of failure. Madhav and Vitkar (1978) presented solutions for the ultimate bearing capacity of a footing on a granular trench (plain strain case) based on the general shear failure mechanism.

2.2.2 Critical Pile Length

The relation between vertical stress σ_v and L/D ratio is given by Hughes and Withers (1974) (Fig 2.3a)

$$\sigma_{vz} = \sigma_v + \frac{Lc}{D} (\rho D - 4c_u) \qquad \ldots(2.2)$$

Where σ_{vz} = vertical stress at depth given by L/D ratio

ρ = density of the pile material

c_u = Undrained shear strength of clay

(Lc/D) ratio = 4.1

At bulging failure, the stress $_{vz}$ at the top decreases with depth and reaches to a value zero at some depth Fig. 2.3 (a) Beyond a depth (Lc/D = 4.1), Fig. 2.3(b) there is no increase in the strength of the pile. If the depth is reduced, Hughes and Withers (1974) reported that:

(i) the pile will act as an end bearing pile

(ii) if the length is so short that the stresses at the base exceeds the bearing capacity of the clay (9c), then end bearing failure will occur.

(iii) Care must be taken for selecting a suitable value of c.

Hughes et.al (1975) proposed a simple way to find out the critical length of a pile under bulging mode of failure by assuming that the vertical shear stress developed along the pile surface - clay interface is equal to average shear strength of the clay

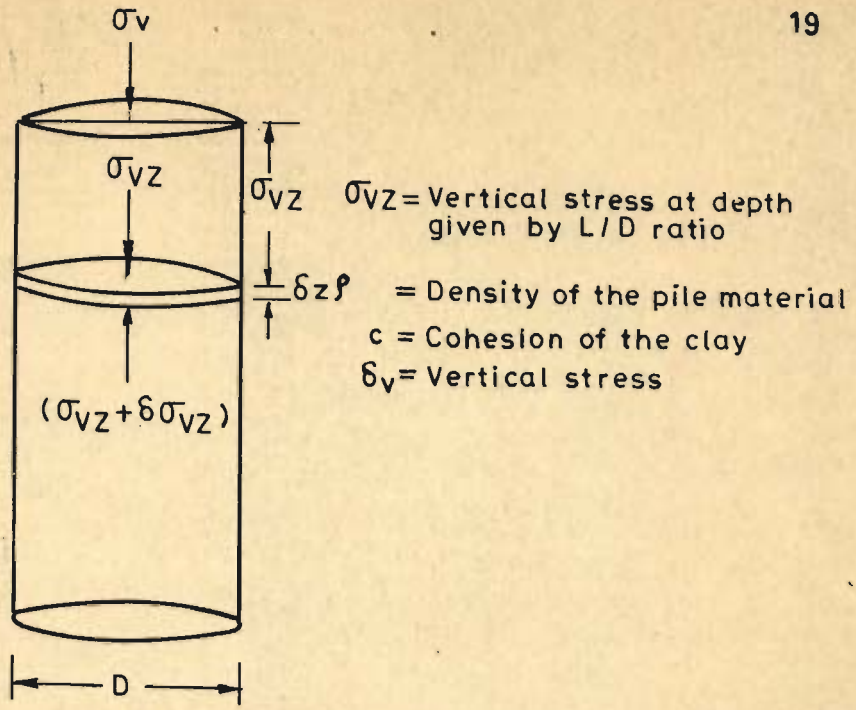


FIG.2.3(a).VARIOUS STRESSES ACTING ON PILE
(AFTER HUGHES AND WITHERS 1974)

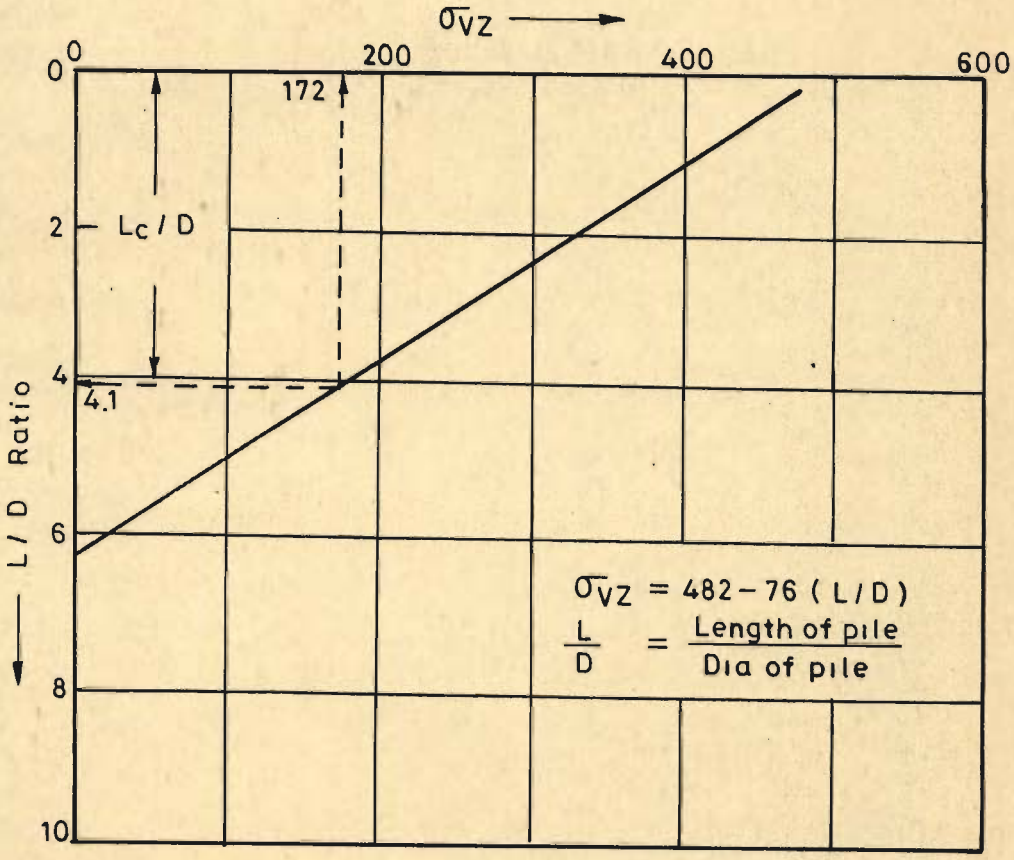


FIG.2.3(b).VARIATION OF VERTICAL STRESS WITH DEPTH IN THE PILE MATERIAL
(AFTER HUGHES AND WITHERS 1974)

when end bearing failure is about to occur. Thus the critical length can be evaluated by equating the boundary forces on the pile, pile load equal to the sum of the shaft resistance and the bearing forces.

$$Q_o = \bar{c} \; A_c + cNc \; A_s \qquad \ldots(2.3)$$

Where Q_o is the ultimate pile load

N_c is the bearing capacity factor taken equal to 9.

A_s is the surface area of the pile equal to (πDLc) having diameter as D.

A_c is the pile cross sectional area and .

$\bar{c}$ and c are the average shaft cohesion and cohesion at the bottom of the critical depth. Hence the value of L_c may be found by knowing the soil properties.

Williams (1969) has reported that the pile under a typical loading condition shall be weakest at a depth equal to 1.9 to 2.33D in weak clay deposit under triaxial stresses with equal alround stress.

2.2.3 Analytical Approaches

(a) Passive Pressure Approach

In the case of passive pressure approach, the load applied through a strip footing (Fig. 2.4) on a granular pile top tends to concentrate on granular pile as the stronger material of the composite foundation soil. The pile material dilates and exerts lateral stresses on the surrounding clay which is resisted by passive earth pressure (Greenwood 1970). Conventional theory of passive pressures implies an increase of pressure with depth.

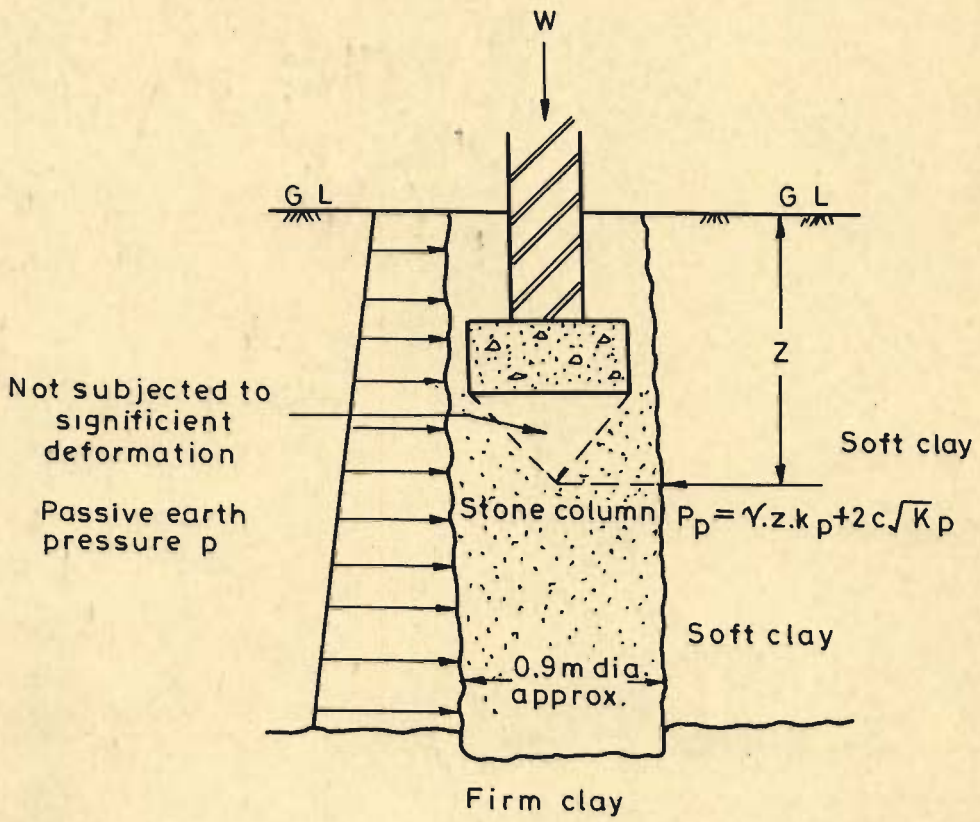


FIG.2.4.GRANULAR PILE UNDER STRIP FOOTING (AFTER GREENWOOD 1970)

There will be a zone of no significant deformation within the pile (shown by dotted lines) under the rigid concrete footing. It was the belief that the ultimate bearing capacity of a single granular pile is equal to the ultimate lateral strength of the soil surrounding the pile (Hughes and Withers 1974). Thus the ultimate bearing capacity of a granular pile is given by the following equation, (Greenwood 1970), as a two dimensional plastic failure case.

$$q_{ult} = P_p = \gamma Z K_p + 2C\sqrt{K_p} \qquad ...2.4$$

Where q_{ult} is the ultimate load bearing capacity of the granular pile in tonnes per sq meter,

γ is the bulk density of the clay in tonnes per meter cube

Z is the total depth of the limit of bulge of the pile &

K_p is the coefficient of passive earth pressure.

The total depth of bulge Z, is equal to the depth of the footing from the ground level plus the depth of the bulge of the pile which is the critical pile length (Fig. 2.4). In case of a clay of essentially uniform strength, the passive restraint just below the dotted line (Fig. 2.4), the granular pile will be weakest where the lateral support is least (which is about 1.75m to 2m below ground level) (Greenwood 1970). This critical length is found to be equal to 2 times the pile dia. (Williams 1969) as reported by Greenwood (1970). However, in the case of bulging failure mode in clay, the critical length is found to be 4 times the pile diameter (Hughes and Withers 1974). The equation 2.4 as proposed by Greenwood (1970) gives the all round passive pressure which is taken as equal to ultimate bearing capacity of the pile, which gives a conservative estimate of the granular

pile capacity. The total pile capacity Q_o (tonnes) is given by:

$$Q_o = q_{ult} A_p \qquad ...2.5$$

Where A_p is the cross sectional area of the granular pile.

The lateral passive restraint on the pile away from the edge of loaded area under the wide spread footing is much larger due to the equal alround pressure influence due to surcharge load. Generally the effect of surcharge load 'q' to the restraining pressure is much larger than the strength of the surrounding soil and its density. Thus the total carrying capacity of the granular pile increases untill the local shear failure in clay (due to contact stresses with the individual pile material back fill particles) or the end bearing failure of the pile occurs first. It is determined by the strength of the surrounding soil, Fig. 2.5, (Greenwood 1970). The ultimate bearing capacity of the pile (q_{ult}) depends on its diameter and is given by the following equation:

$$q_{ult} = \gamma Z K_p + 2C\sqrt{K_p} + q K_p \qquad ...2.6$$

where q is the surcharge load per unit area.

It has also been demonstrated that the pile near the edge of the loades area (indicated by dotted lines)will not have the same bearing capacity as those under the centre. This is not critical under the embankment loading where gradual decrease of loading occurs, however, it becomes critical when the load terminates abruptly Fig. 2.5. Granular piles near the edge, therefore, must be spaced closely than in the centre and should not carry

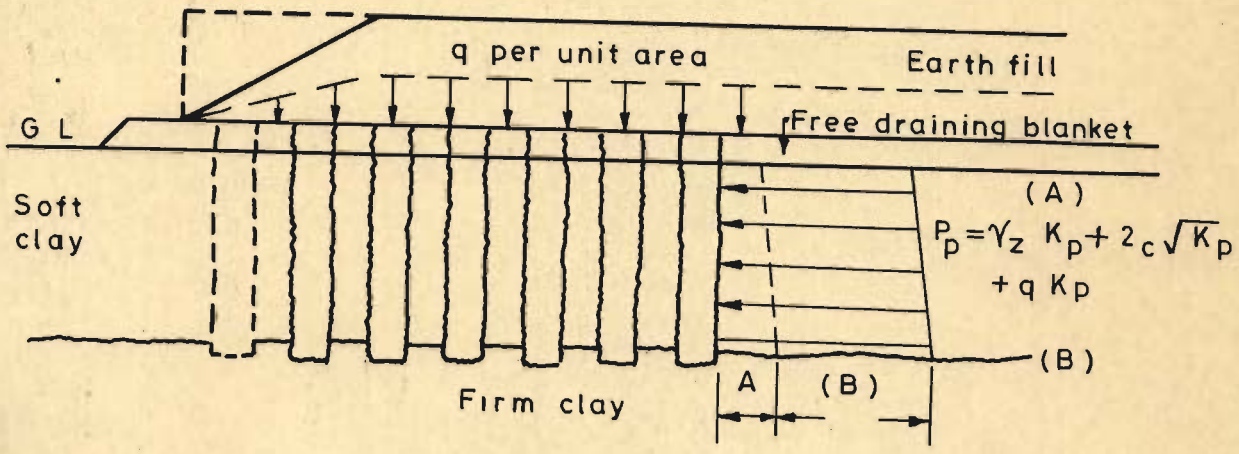


FIG.2.5. GRANULAR PILES UNDER WIDE SPREAD LOAD
(AFTER GREENWOOD 1970)

more loads than those for isolated footings.

(b) General Shear Failure Approach

Madhav and Vitakar (1978) have stipulated the plain strain version of a granular pile as a granular trench and have postulated a failure mechanism for the granular trench Fig. 2.6. Thus utilising the upper bound technique of limit analysis approach analytical solution for the bearing capacity factors have been presented in the graphical forms for a strip footing on a general c-ϕ-γ soil, reinforced with granular trench.

Using the upper bound theorem, the work equation is formed by equating the external rate of work done due to (i) external applied load, (ii) soil weight and (iii) soil surcharge to the internal energy dissipated in the plastically determined region, for which Coulombs yield criteria is valid. The approach of analysis follows closely Chen (1975) theory.

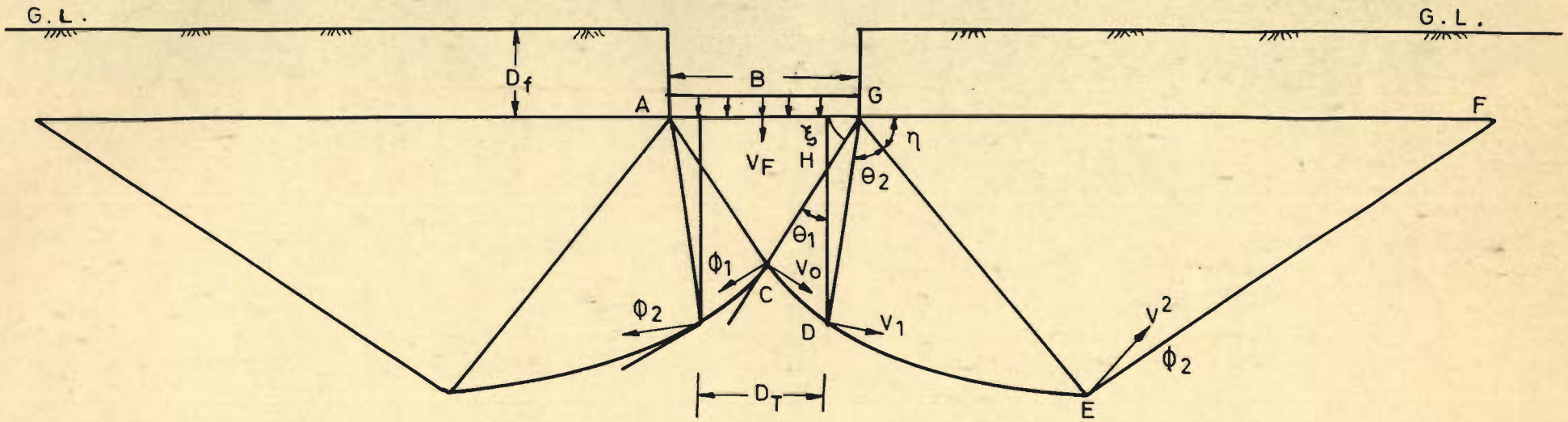
The general shear failure mechanism is postulated for two cases (i) $D_T/B \leqslant 1$ and (ii) $D_T/B \geqslant 1$ as shown in Fig. 2.6(a), 2.6(b). D_T is the trench width and B is the width of the strip footing resting on soil trench system with the foundation at a depth D_f. The mechanism under consideration is 'Prandtl Mechanism' for homogeneous soils consisting of different zones such as Fig. 2.6a. The different zones are:

(i) An active Rankine zone AGC with wedge angle & ξ

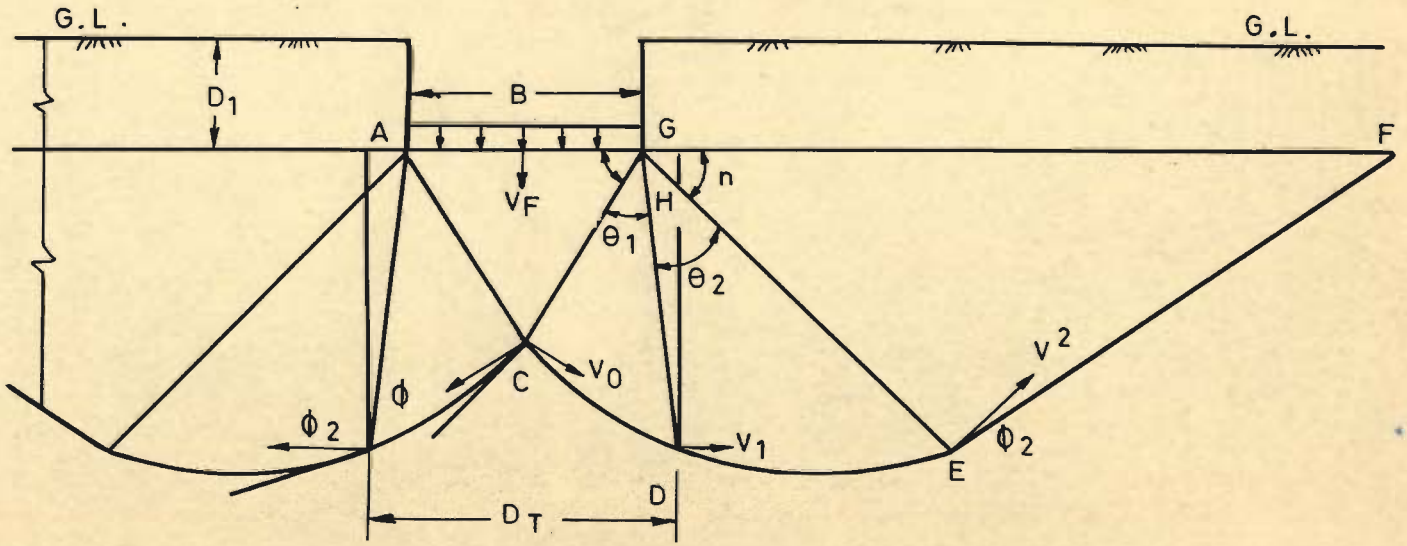
(ii) Mixed transition zone GCD with central angle θ_1 bounded by log spiral based on frictional angle ϕ_1 of the trench material

$$r = r_o \exp(\theta \tan \phi_1) \qquad \ldots 2.7$$

where $r_o = GC = \dfrac{B}{2\cos(\xi)}$



(a) Failure mechanism for $D_T / B \leq 1$



(b) Failure mechanism ($D_T / B \geq 1$)

FIG.2.6.FAILURE MECHANISM
(AFTER MADHAV AND VITKAR 1978)

(iii) A transition zone GDE with central angle θ_2 bounded by log spiral based on friction angle $\emptyset_2$ of the natural soft soil having the equation

$$r = r_o \exp(\theta \tan \emptyset_2) \qquad ...2.8$$

where $r_o = GD = r_o \exp(\theta_1 \tan \emptyset_1)$

(iv) Passive Rankine Zone GEF with wedge angle η

The wedge AGC moves vertically down as a rigid body with the same initial velocity V_F of the footing. The downward movement of the footing and wedge AGC is accommodated by the lateral movement of the adjacent soil. The central angles θ_1 and θ_2 of the transition zone depend upon the wedge angles ξ and η, the ratio D_T/B and the angle of friction $\emptyset_1$ of the trench material. The properties of the granular trench material considered are c_1, $\emptyset_1$ and γ_1 which are cohesion, angle of friction and density respectively. Cohesion c_1 of the trench material could be zero, however, the theory is developed for most general case of c-$\emptyset$-γ soil. The properties of natural soil are cohesion c_2, angle of friction $\emptyset_2$ = and density γ_2.

From the geometry of the failure mechanism Fig. 2.6a, 6b the lengths and velocities at various discontinuities are found.

The rate at which the work is done by soil weight is found by multiplying the area of each rigid body by γ times the vertical component of the velocity of the rigid body. The velocity component of the zones AGC, GCD, GDE and GEF are considered to act in the same direction at that of the force V_F, while that of surcharge in the opposite direction. This convention is based on whether the work is done against V_F or in the same direction of V_F.

The rate of external work for the various zones are (i) work done due to unit weight of the soil (ii) work done on account of surcharge and (iii) work done by external load. The work equation is formulated by equating total rate at which the work is done by (i) external load on the foundation (ii) soil weight in motion and (iii) soil surcharge, to total rate of internal energy dissipation along the lines of discontinuities.

Equating the work done by the external load (q_{ult}) to the energies dissipated by cohesion, and work done on account of soil weight and surcharge, the final equation is given as:

$$q_{ult} = c_2 N_c + (\gamma_2 \frac{B}{2}) N\gamma + \gamma_2 D_f N_q \qquad ...2.9$$

$$\text{where } N_c = (\frac{C_1}{C_2}) N_{c1} + N_{c2} \qquad ...2.10$$

$$\text{and } N\gamma = (\frac{\gamma 1}{\gamma_2}) N\gamma_1 + N\gamma_2 \qquad ...2.11$$

N_{c1}, N_{c2} $N\gamma_1$ and $N\gamma_2$ and N_q are the dimensionless factors which depend on the properties of trench material and soil material and the ratio D_T/B. C_2 $_2$ and C_1 $_1$ are cohesion and unit weight of soil and trench material respectively D_f is the depth of foundation.

The bearing capacity factors have been evaluated for the ranges of parameters listed in Table 1 and are presented in Figs.2.7 through 2.9.

Table 2.1 Ranges of Parameters

Parameters	Ranges
A/B	0 - 2
ϕ_1	20 - 50°
ϕ_2	0°
c_1/c_2	0 - 1
γ_1/γ_2	1.0

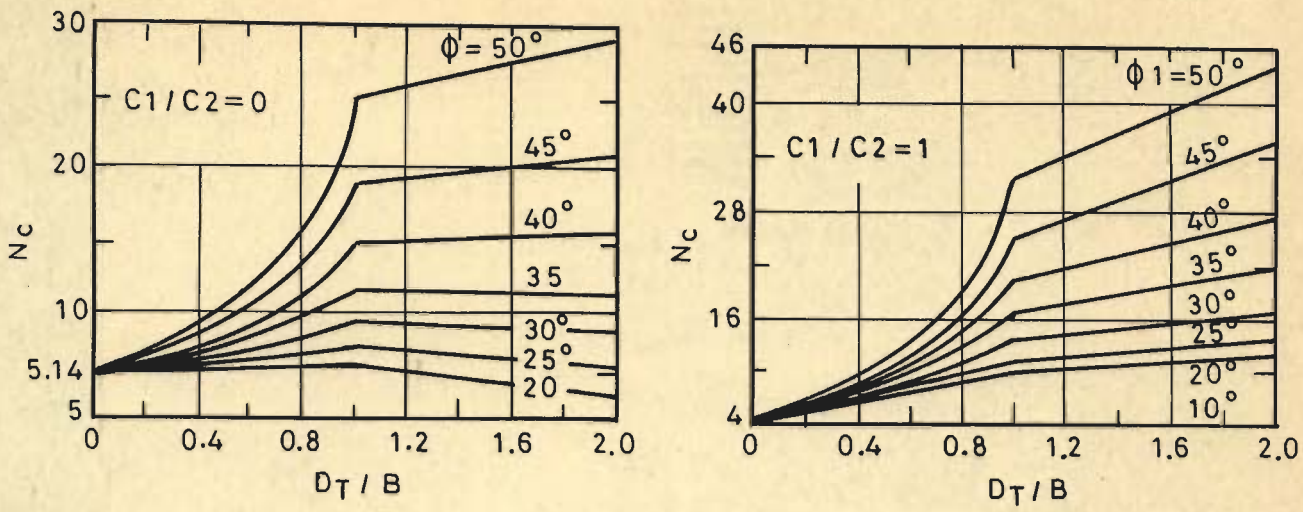


FIG.2.7. BEARING CAPACITY FACTORS N_c

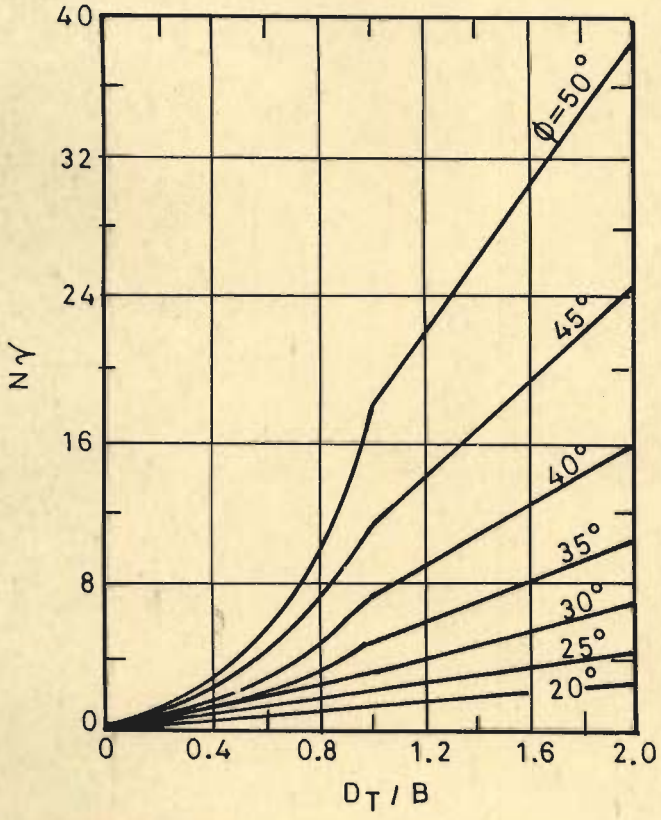


FIG.2.8. BEARING CAPACITY FACTOR $N\gamma$
(AFTER MADHAV AND VITKAR 1978)

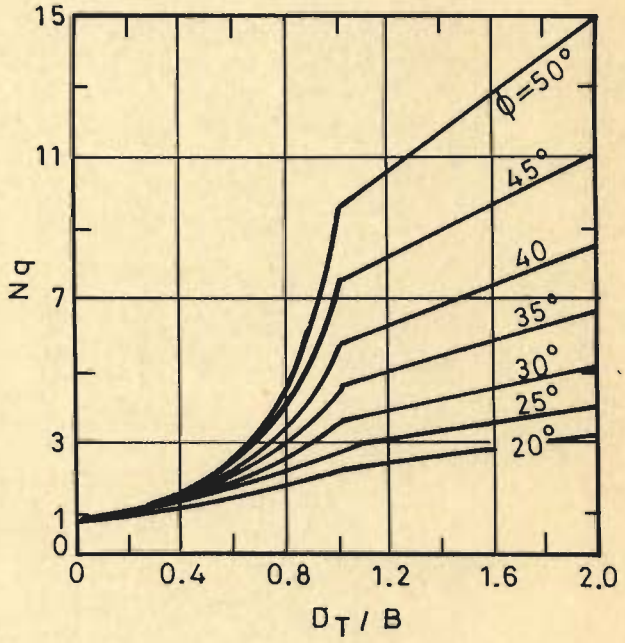


FIG.2.9. BEARING CAPACITY FACTOR N_q
(AFTER MADHAV AND VITKAR 1978)

When $c_1 = c_2$ and $\emptyset_1 = \emptyset_2$, the soil trench system reduces to a homogeneous medium and the values of the bearing capacity factors are the same as of Chen (1975). For a axisymmetric case (grannular pile) in soft soil the values of shape factors may be taken from Vesic (1975).

N_c values vary significantly with the ratio A/B $\emptyset_1$/ and the ratio c_1/c_2, when A/B = 0, the problem reduces to the homogeneous case with soft soil having $\emptyset_2$=0 and the N_c value coincide with the Prandtl value of 5.14 (Fig. 2.7)

The rate of increase of N_c values is higher for $\frac{A}{B}$ in the range of 0-1 than in the range of 1-2. This rate depends on the ratio c_1/c_2. For A/B$\rightarrow\infty$ N_c value should coincide with Prandtl values corresponding to $\emptyset_1$. As in the homogeneous case, N_c value increase with increase in $\emptyset_1$ values. For an intermediate ratio of c_1/c_2, the N_c value can be interpolated.

The N_γ values are ploted agains A/B for different $\emptyset_1$ and the ratios of γ_1/γ_2 =1. Fig 2.8. The value of $N\gamma$ also increase with A/B and $\emptyset_1$ values. The increase is significant because more and more trench material will be contributing to the work done by soil weight, as the ratio A/B increases N_γ values for $\frac{\gamma_1}{\gamma_2}$ =1.5 are only 10 per cent more than the values for $\frac{\gamma_1}{\gamma_2}$ =1. Hence N_γ values are given only for γ_1/γ_2 =1. Variation of N_q is similar to those of N_c and N_γ , (Fig. 2.9).

(c) Cavity Expansion Approach

The problem of expansion of a cylinderical cavity in a homogeneous, infinite isotropric soil mass expanded by a uniformly distributed internal pressure is considered analogous to the bulging

of a single granular pile in a soil media and is subjected to a sustained vertical load on the pile top only. Because of the internal pressure the initial radius of the cavity increases resulting into increase in the plastic zone around the cavity. Beyond this plastic zone the soil remains in the elastic state of equilibrium. To arrive at the lateral limit stress, the soil in the plastic zone behaves as compressible plastic solid defined by Coulomb-Mohr shear strength parameters c and Ø, and average volumetric strain Δ . Beyond the plastic zone the soil is assumed to behave as linearly deformable isotropic solids defined by modulus of deformation E_s, and the Poissions ratio μ. It is further assumed that prior to the load application the entire soil mass has an isotropic effective stress σ_m and that the body forces within the plastic zone are negligible when considered with existing and newly applied stresses. Based on above assumptions the lateral limit stress is computed as proposed by Vesic (1972).

Datye and Nagaraju (1977) used cavity expansion approach proposed by Vesic (1972) and computed the limit lateral stress of a granular pile in soft saturated clay having 0.75m nominal diameter. The soft saturated clay was having cohesion c as one tonne per sq.m. submerged unit weight γ_{sub} as 0.9 tonne per cu.m. and the area of the installed pile was taken as 0.7082 sq.m. The computations have been made for the two extreme cases of soft saturated clay (rigidity index I_r=10) and stiff clay (I_r = 300). The main assumptions made in computing the ultimate bearing capacity of the granular pile are (a) the depth of the cylinderical cavity is taken arbitrarily as 4m below the ground level (b) the stability of the reinforced clay is based on assumed load sharing ratio between the pile and the surrounding soil based on strain compatibility phenomenon and (c) the ultimate vertical stress

is taken as six times the limit lateral stress computed from cavity expansion approach. No rational justification for these assumptions has, however, been made. Also the comparision between the observed and computed pile capacity has not been made as the piles were subjected to preloading before load testing though the comparision in a earlier case is claimed to be satisfactory (Datye and Nagaraju 1975).

Similarly in the case of non-cohesive soil (c=0) Datye and Nagaraju (1981) have computed the ultimate bearing capacity of the granular piles for a range of compressibilities (loose sand to dense deposits) using different values of rigidity index and have indicated the adequacy of such an analysis even if the ratio of the ultimate pile capacity and the limit lateral stress is reduced due to dilation of the pile under vertical loading associated with the enlargement of the pile diameter.

2.2.4 Empirical Methods

The load bearing capacity of a single granular pile is a complex problem involving interaction of the granular pile material constituting the pile and the soil surrounding the pile. As such no exact mathematical solution is available to estimate the ultimate bearing capacity and settlement of granular piles.

Granular piles can not be considered as completely rigid elements, although they can perhaps be thought of as piles with low factor of stiffness. Further these are not capable of transferring high stresses to the lower deep load bearing stratum. Under the actual loading conditions, the applied load is distributed between

the granular pile and the surrounding in-situ soil. For this reason, the experimental structures and model tests, where vertical loads are directly applied to the granular pile, while in-situ soil remain in its original state of stress, do not correspond with actual conditions. In these circumstances a different state of stress exists between the granular pile and the surrounding soil. The existing construction practices utilise: vibro-replacement method for the construction of granular piles in soft clays. Thus the ultimate bearing capacity of a granular pile is dependent on its diameter. With a wet technique of construction, the back fill consumption is usually 0.65 to 0.85 cu.m per linear meter resulting into a diameter of 0.9m to 1.04m. However, in the case of dry processes of construction the corresponding figure of back fill consumption is 0.28 to 0.7 cu.m. per linear meter of the pile which gives a diameter between 0.6 to 0.7m (Greenwood 1970). On the basis of actual load tests, the following empirical approaches have been suggested:

(a) Thorburn and McVicar Chart (1968)

Based on their experience and results of the actual load tests on granular piles, Thorburn and Mc Vicar (1968) proposed an empirical relationship (Fig 2.10) between allowable load on a granular pile and the undrained shear strength 'c_u' of the cohesive soil mass surrounding the pile. It was, however, recommended that the allowable load thus obtained, should invariably be verified by in-situ load tests. Also the consolidation settlement of the cohesive soil reinforced with granular piles should also be calculated and compared with structural requirements, though method to compute settlement in such a case has

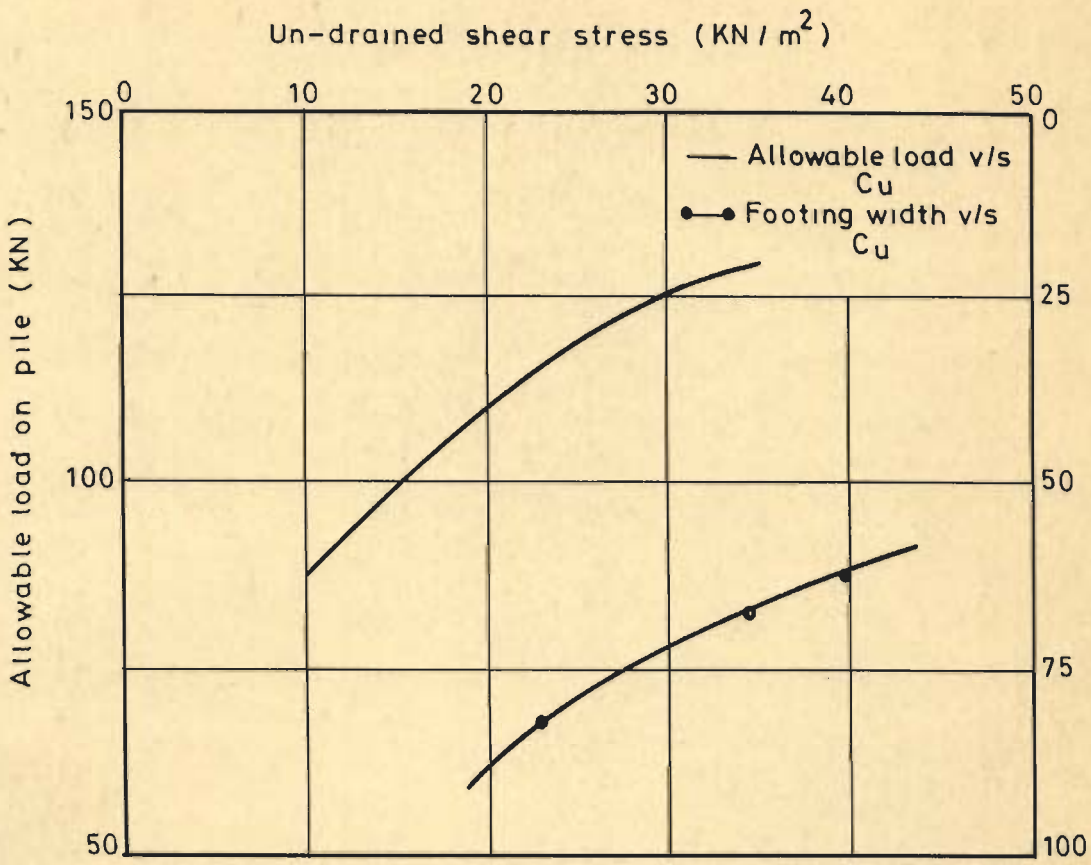


FIG.2.10. RELATION BETWEEN ALLOWABLE LOAD ON PILE AND UNDRAINED SHEAR STRESS
(AFTER THORBURN AND Mc VICAR 1968)

not been suggested. While arriving at the empirical design chart, it is assumed that the total design load is supported directly by granular piles and based on this assumption the total number of piles should be calculated. Further it is claimed that such a design approach is likely to have an adequate factor of safety, both initially under rapid loading conditions and finally when the excess pore water pressure is fully dissipated. This will ensure safety against bearing capacity failure and provide ground for considerable stiffness.

Further it has been suggested that in case of strip resting on cohesive soils reinforced with granular piles, footing width should be such that it covers the granular piles. The recommended width in relation to un-drained shear strength of the cohesive soil is also shown in Fig. 2.10 (Thorburn and McVicar 1968). Also, it is pointed out that the cohesive soils having shear strength less than 0.191 kg/cm^2 (19.2 KN/m^2) can not be satisfactorily reinforced with granular piles due to their low passive resistance and difficulties of forming sound piles. The relation between un-drained shear strength (KN/m^2) working pile load as proposed by Thorburn (1975) is shown in Table 2.2

Table 2.2 - Relation between undrained shear strength (c_u) and working pile load

c_u (KN/m^2)	Working pile load (KN)
19.2	88.5
24.0	100.83
28.8	110.67
33.6	118.00
38.4	125.43
43.2	127.89
48.0	132.81

(b) Charts proposed by Thorburn (1975) and Hughes & Withers (1974)

Within the normal range of un-drained shear strength of cohesive soils which can be reinforced with granular piles for preliminary design purposes, Thornburn (1975) proposed an empirical relationship between the allowable working load and the un-drained shear strength c_u, Fig. 2.11. The allowable working load of granular pile was arrived at by using passive earth pressure approach and the effective diameter were obtained from field measurements, which is also marked on Fig. 2.11. A similar relationship between allowable working load and 'c_u' as proposed by Hughes and Withers (1974) is marked on Fig. 2.12 by dotted lines. The field measurement of effective diameters of granular piles are related to piles formed by powerful cementation and Keller Vibrofloats which were used for the construction of granular piles. It is seen from Fig. 2.12 that the non linear relationship between allowable working load and undrained shear strength c_u, proposed by Thorburn (1975) shows a good correspondance with those suggested by Hughes and Withers (1974). Similar comparision is also shown in Fig. 2.13a, where empirical relations suggested by Hughes and Withers (1974) and Mokashi et.al(1976)are marked. These also compare well with each other.

(c) Charts proposed by Smoltzyk (1979)

A modified design chart based on undrained shear strength and settlement has been proposed by Smoltczyk (1979) Fig. 2.13b. It is observed that the lateral earth resistance is not sufficient for granular pile when undrained shear strength c_u is less than 0.15 kg/cm^2 (15KN/m^2). However, an economical limit for the method is given in cases of stiff cohesive soils, with c_u values

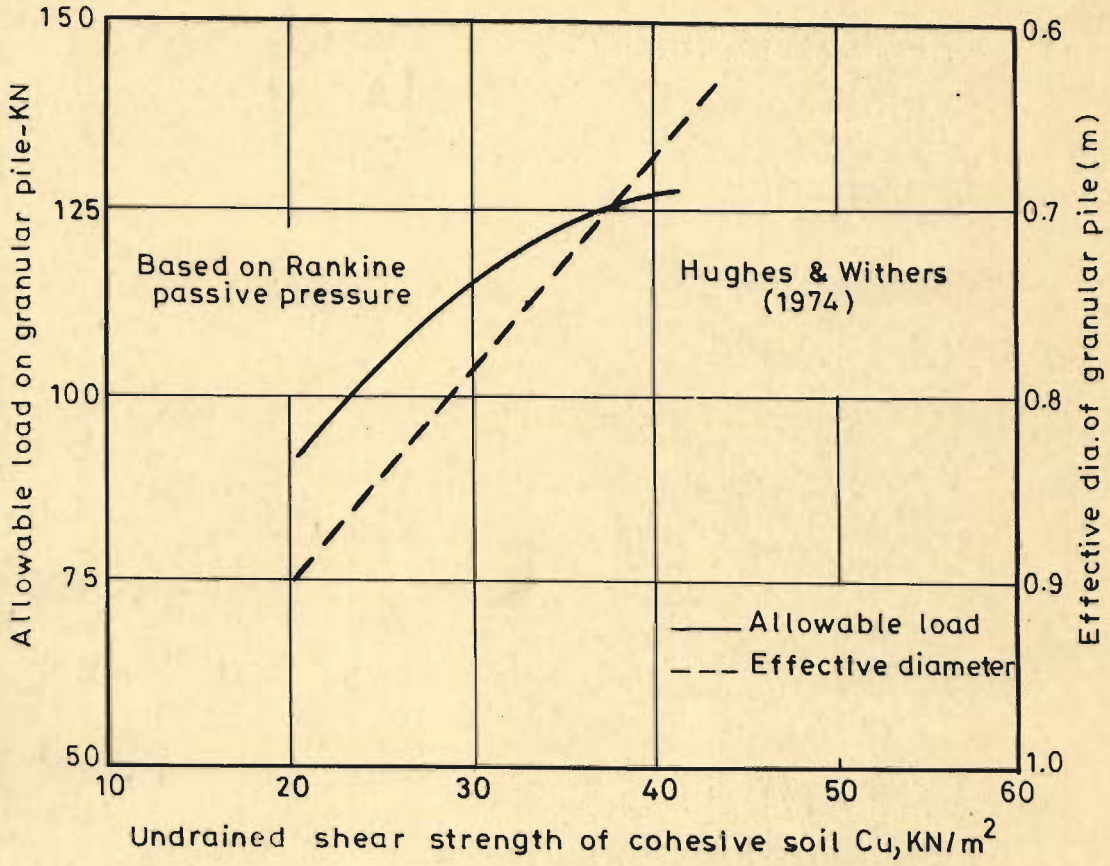


FIG.2.11. RELATIONSHIP BETWEEN ALLOWABLE WORKING LOAD ON GRANULAR PILE AND UNDRAINED SHEAR STRENGTH OF COHESIVE SOIL AT POINT OF MAXIMUM RADIAL RESISTANCE (AFTER THORBURN 1975)

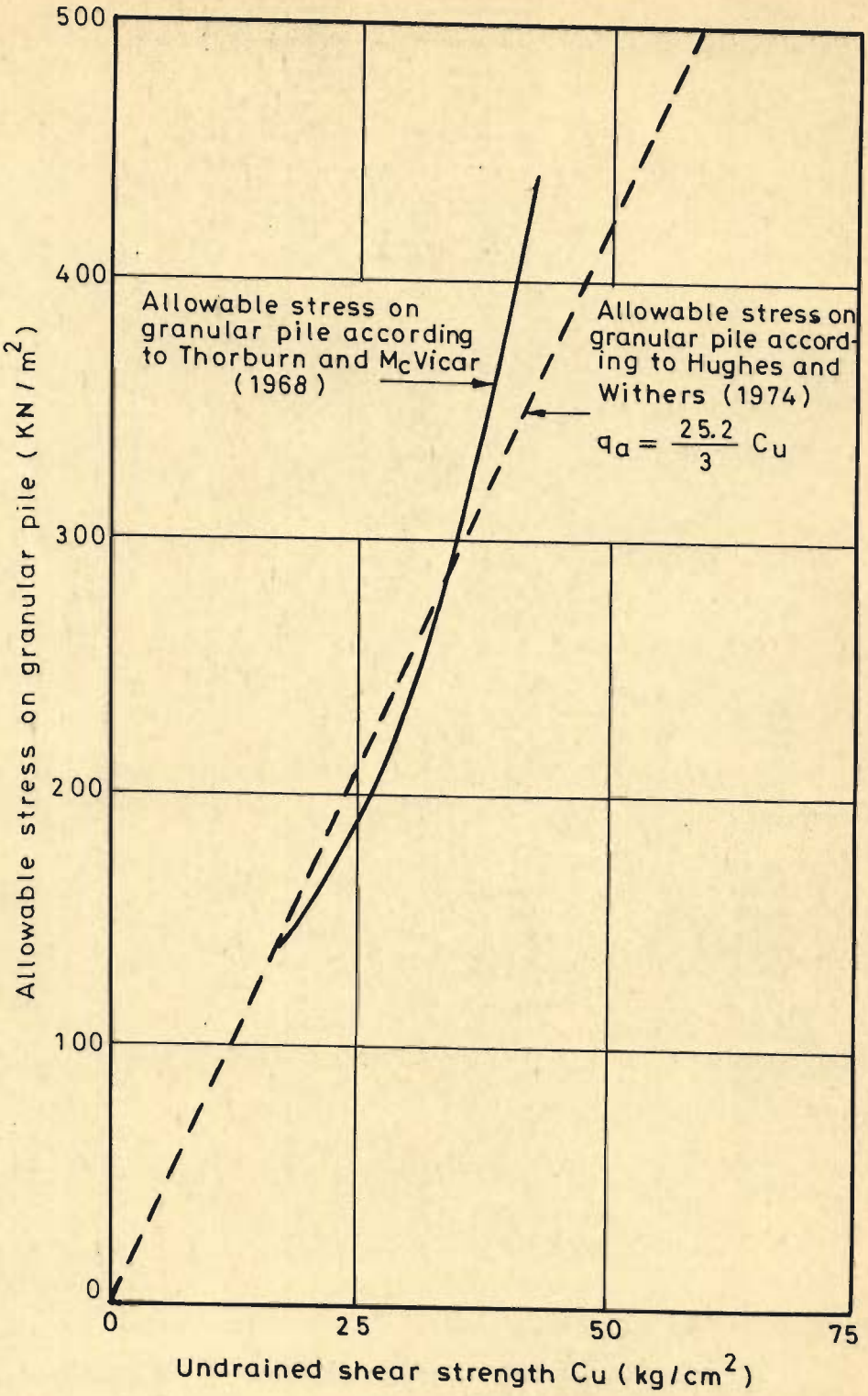
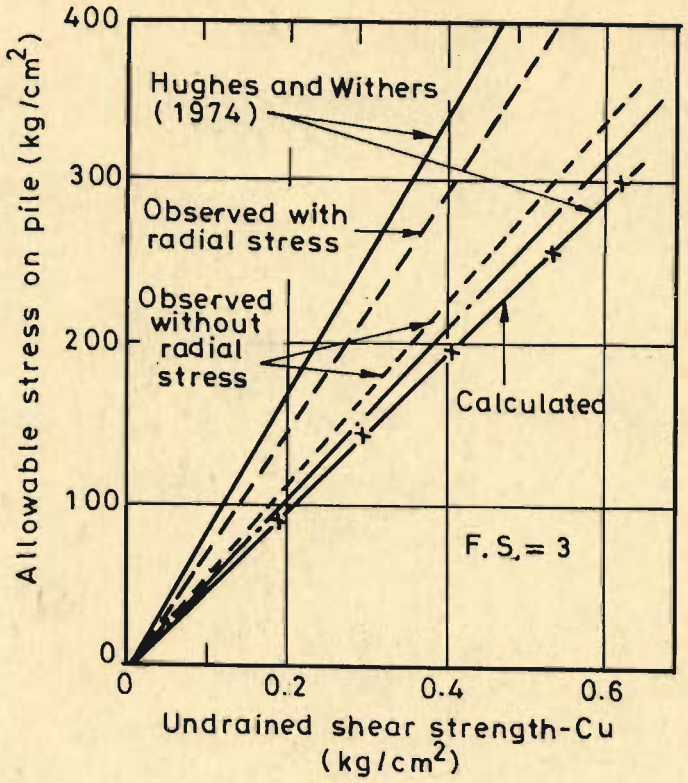


FIG.2.12. RELATIONSHIP BETWEEN ALLOWABLE VERTICAL STRESS ON GRANULAR PILES AND UNDRAINED SHEAR STRENGTH COHESIVE ARTIFICIAL DEPOSIT
(AFTER THORBURN 1975)



Calculated with radial stress ——— - —
Observed with radial stress — — — —
Calculated without radial stress ——x——
] Hughes and Withers (1974)

Observed without radial stress ——— - —
Observed without radial stress — — — —
But with pre-compressed layers
] Mokashi et al (1976)

FIG.2.13(a).RELATIONSHIP BETWEEN-C_u (kg/cm^2) AND ALLOWABLE PILE STRESS (kg/cm^2)
(AFTER MOKASHI et.al.1976)

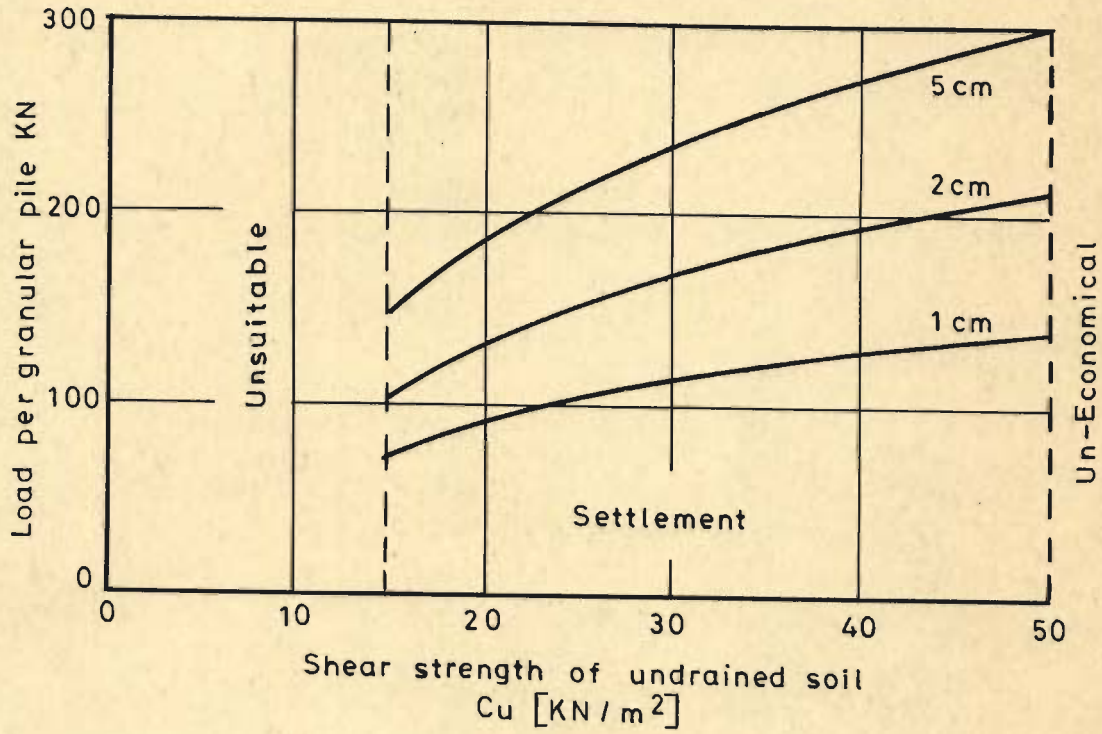


FIG.2.13(b). PROPOSAL FOR DESIGNING BEARING CAPACITY OF GRANULAR PILES (AFTER SMOLTZYK 1979)

equal to or greater than 0.5 kg/cm^2 (50 KN/m^2)

2.2.5 SEMI EMPIRICAL APPROACH

(Pressuremeter Theory)

(a) Soils with Cohesion (c) and friction (Ø)

The ficticious case of expansion of the cavity from a zero initial radius was first demonstrated by Bishop, Hill and Mott (1945). But Ladanyi (1961) was responsible for developing it to its full potential. Baguelin Jezequel and Shield (1978) have proposed the relationship between limit pressure σ_{rL} and soil strength c and Ø based on elasto-plastic models. These are presented by following equations

$$\sigma_{rL} = (\sigma_{h_o} + c \cot \emptyset)(1-\sin \emptyset)\left(\frac{1}{2\alpha_F}\right) \frac{1-K_a}{2 - c \cot \emptyset} \quad ..2.12$$

and for purely cohesive soils

$$\sigma_{rL} = \sigma_{ho} + c_u \left(1 + \mathrm{Ln} \frac{1}{2\alpha_F}\right) \quad ..2.13$$

Where

σ_{rL} is the theoretical limit pressure at finite expansion of the cavity.

σ_{ho} is the total initial ground stress.

α_F is the ALMANSI STRAIN at failure.

The ALMANSI strain is given by:

$$\alpha = \frac{1}{2} \frac{\delta l^2 - \delta l_o^2}{\delta l^2} \quad ..2.14$$

where δl_o = initial length of the small linear element

δl = length of this element in deformed state.

The value of the circumferential strains at the wall of the cavity are related to volume of the measuring cell in the pressuremeter. If v_o is the initial volume, V is the volume in the deformed state and Δv is the change in volume then ALMANSI strain is expressed as

$$\alpha_F = \tfrac{1}{2}\left[\frac{V - V_o}{V}\right] = \tfrac{1}{2}\,\frac{\Delta V}{v} \qquad ..2.15$$

It is noted that ALMANSI strain is directly proportion to change in volume (ΔV)

Soils with (Ø) values:

There is no theoretical approach at present to relate limit pressure σ_{rL} and effective angle of shearing resistance (Ø') in case of free draining soils. Empirical relationship between limit pressure and (Ø') for sands are expressed below:

$$\sigma_{rL} = 2.5x\,[2]\,\frac{Ø'-24}{4} \qquad ..2.16$$

where σ_{rL} is expressed, in KPa (0.15 psi). The same relationship is quoted by Muller (1970)

$$\sigma_{rL} = b\,[\,2\,]\,\frac{Ø'-24}{4} \qquad ..2.17$$

where b = 1.8 for homogeneous wet soils

= 3.5 for dry hetrogeneous soils

= 2.5 (average)

The values of 13 pressure meter tests in free draining soils demonstrate that the values of lateral limit stress σ_{rL} computed from eqn. 2.16 are more closer to the independently measured values (Winter and Rodriguez 1975 according to Vesic 1972).

(b) Cohesive soils with 'c' values

The method of using the limit pressure σ_{rL} for estimating undrained shear strength c_u from pressure meter has received much more attention in recent past. Three theoretical relationships with different volume change properties are available

Bishop, Hill and Mott (1945)

$$\sigma_{rL} = c_u\left[1 + \text{Ln}\,\frac{E_s}{2(1+\mu)\,c_u}\right] \qquad ..2.18$$

Hill (1950)

$$\sigma_{rL} = c_u\left[1 + \text{Ln}\,\frac{E_s}{c_u\,(5-4\mu)}\right] \qquad ..2.19$$

Salencon (1966)

$$\sigma_{rL} = c_u\left[1 + \text{Ln}\,\frac{E_s}{4c_u\,(1-\mu^2)}\right] \qquad ..2.20$$

where σ_{rL}=limit pressure in a pressure meter,

E_s= the elastic soil modulus,

c_u= undrained shear strength of the clay deposit and

μ= the Possions ratio

If μ = 0.5, all the above equation reduce to

$$\sigma_{rL} = c_u\left[1 + \text{Ln}\,\frac{E_s}{3\,c_u}\right] \qquad ..2.21$$

The equation is re-written as

$$\sigma_{rL} = k.c_u \qquad ..2.22$$

$$\text{where } k = \left[1 + \text{Ln}\,\frac{E_s}{3c_u}\right] \qquad ..2.23$$

Typical values of the ratios E_s/c_u range between 200 to 2000 (D'Appolonia, Poulos and Ladd 1971). Substituting these values in the Equation 2.21, the corresponding values of k vary between 5.2 to 7.5. The theoretical values of k would be increased by 1/3 if the pressure meter tube is too short and a spherical cavity is expanded in the ground instead of cylinderical cavity (Bishop et.al 1945). Also higher values of E_s will lead to much higher values of k. It has been concluded that none of the three variables (σ_{rL} , k and c_u) that make up the Equation 2.23 can be determined with precesion (Baguelin, Jezequel and Shields 1978). However Cassan (1972) has recommended k equal to 5.5 for low values of σ_{rL} and also according to Amar and Jezequel (1972) the limit pressure σ_{rL} is less than 3 kg/cm^2 (300 kPa). For the intermediate range of σ_{rL} (Limit pressure) the value of k is proposed as 8 and for high limit pressure this value is increased to 15 (Cassan 1972).

Amar and Jezequel (1972) suggest instead an equation relating limit pressure σ_{rL} and undrained shear strength c_u as

$$c_u = \frac{\sigma_{rL}}{10} + 25 \text{ kPa} \qquad ..2.24$$

where c_u and σ_{rL} are in kPa (0.01 kg/cm^2)

Hughes and Withers (1974), Hughes, Withers and Greenwood (1975) have proposed a value of k equal to 4 in equation. However, Mori (1979) and Broms (1979) assign a value of k as 5 in their design. Mokashi et.al (1976) found k for clay (c_u = 0.19 kg/cm^2) as 3 and for Wax (c_u = 0.88 kg/cm^2)

as 3.8 in the model tests in laboratory.

Lukas and LeClere De Bussy (1976) recommend k equal to 5.1 for cohesive fills and hard pan in Chicago region. If elastic soil modulus from pressure meter (Em) is used then Menard (1967) suggest the following equation:

$$C_u = \frac{\sigma_{rL}}{1+Ln\left(\frac{E_m/\alpha}{3c_u}\right)} \qquad ..2.25$$

where α is the rheological coefficient which is of the order of ½ to 1/3 Menard (1970) recommends k equal to 5.5 to compute the ultimate or residual strength. This value of k is supposed to give precise value of c_u (residual)

(c) Limit pressure

(i) Gibson and Anderson (1961)

Gibson and Anderson (1961), after incorporating the effect of total initial lateral stress (σ_{ho}) in an elasto plastic material, based on in situ measurement of limit pressure (σ_{rL}) in a pressure meter modified the Bishop Hill and Mott (1945) equation in the form given by:

$$\sigma_{rL} = \sigma_{ho} + c_u\left[1 + Ln\frac{E_s}{2c_u(1+\mu)}\right] \qquad ..2.26$$

$$\text{or} = \sigma_{ho} + k\,c_u \qquad ..2.27$$

(ii) Hughes and Withers (1974)

Based on the results of quick pressure meter tests Hughes and Withers (1974) demonstrated that the Limit Pressure σ_{rL} may be approximated to an acceptable solution replacing

rigorous - analytical solution by following equation:

$$\sigma_{rL} = \sigma_{ho} + k\, c_u + U_o \qquad ..2.28$$

where U_o is the initial excess hydro-static pore water pressure and other terms are as defined earlier.

Based on the field records and the fact that the granular piles, as in the case of triaxial tests where the cell pressure is limited, the limit stress σ_{rL} is approximated by equation 2.28. If a loaded granular pile behave as a pressuremeter, (Hughes et.al 1975) and the granular material constituting the granular pile approaches shear failure with an angle of internal friction Ø' and the bulging occurs near the top of the pile, the ultimate bearing capacity q_{ult} of the granular pile is given by eqn 2.29 (taking U_o as zero by allowing full drainage in the granular pile).

$$q_{ult} = k_p(\sigma_{ho} + k\, c_u) \qquad ..2.29$$

$$= k_p\, \sigma_{rL}$$

Where σ_{ho} is the initial total lateral stress at a depth z, c_u is the undrained shear strength of clay and k is a coefficient defined earlier and is equal to 4 in accordance with findings of Hughes and Withers (1974). The coefficient of passive earth pressure K_p is defined as:

$$K_p = \tan^2 (45 + \tfrac{1}{2}\, Ø') \qquad ..2.30$$

where Ø' is the angle of internal friction of the granular pile material (stone aggregate). The values of K_p depends on Ø'. The values of Ø' as 36° to 45° have been used (Broms 1979). At Ø' = 45° , value of K_p = 5.89.

If the initial total lateral stress (σ_{ho}) due to over burden at depth z is neglected, then $q_{ult} = K_p.k\ c_u$ ..2.31

The allowable load q_{allow}, if a factor of safety F with respect to ultimate bearing capacity is used, will be equal to

$$q_{allow} = (K_p.k\ c_u)/F \qquad ..2.32$$

At $K_p = 5, k=5$ and F = 3 (Broms 1979)

$$q_{allow} = 8.3\ c_u \qquad ..2.33$$

According to Hughes and Withers (1974), at

$\sigma_{ho} = 54\ KN/m^2$ (0.55 kg/cm^2), $\emptyset' = 35^o$

$K_p = 3.69$, k = 4 and F = 3

$$q_{allow} = \frac{25.22}{3}\ c_u = 8.4\ c_u \qquad ..2.34$$

The ultimate axial load q_{ult} was found to be as 482 KN/m^2 when $c_u = 19.1\ KN/m^2$ which agreed well with the observed values.

(iii) Mori (1979)

Based on field experience Mori (1979) suggested a modified Hughes and Withers (1974) equation , for the ultimate bearing capacity of granular pile (Equation 2.35).

$$q_{ult} = k_p\ \left(0.5\gamma' h + 5c_u\right) \qquad ..2.35$$

The only difference with Hughes and Withers (1974) equation lies in selecting the value of total initial lateral stress σ_{ho} and k values which are taken as ($0.5\gamma' h$) and 5, where $\gamma' h$ is the overburden pressure. Taking $\emptyset'=38^o$ and $c_u=0.2\ kg/cm^2$, $\gamma'=0.7t/m^3$ and h=5m, Mori (1979) found the value of q_{ult} as $50t/m^2$ which agrees well with Hughes and Withers (1974).

Taking Ø' 38° and c_u = 0.2 kg/cm^2, ' = 0.7t/m^3 and h = 5m, Mori (1979) found the value of q_{ult} as 50t/m^2 which agrees well with Hughes and Withers (1974).

The equation 2.29, is re-written as:

$$q_{ult}/c_u = K_p.k \qquad ..2.36$$

When effect of initial total lateral stress (σ_{ho}) is neglected. While considering the effect of σ_{ho} Hughes and Withers (1974) found the value of q_{ult}/c_u = 25.2. This value reduced to 14.7 when (σ_{ho}) is neglected (Mokashi et.al 1976) though their actual tests gave a value of q_{ult}/c_u = 22 or (K_p.k)=22. The values of Ø' as reported by Mokashi et.al (1976) range between 45° to 55°. Thus the value of (K_p.k) ranges from 25 to 30. The details of soil parameters and values of k and K_p k is shown in Table 2.3

Table 2.3 Undrained shear strength and ultimate pile capacity

(c_u) kg/cm^2	Material	Values of k	Ratio q_{ult}/c_u or (K_p.k)	Reference
0.194	clay	4	25.2	Hughes & Withers(1974)
0.19	clay	3	15.8-17.8	Mokashi et.al (1976)
0.88	Wax	3.8	-	-do-
0.20	soft clay	5	20	Mori (1979)
-	clay	5	25	Broms (1979)

2.2.6 Experimental Studies

A series of model tests were carried out at Cambridge University

using radio graphic technique to determine the actual behaviour of single granular piles in normally consolidated clay (Hughes and Witbers 1974).

The un-drained shear strength of the clay used in the test was 19.1 KN/m^2 (0.1947 kg/cm^2). The angle of shearing resistance of the granular pile material was 35°. The initial lateral stress in the clay was 54 KN/m^2 (0.55 kg/cm^2) which is found to be as 2.84 times the value of undrained shear strength of clay. The test results indicated that the ratio of the applied stress and the undrained shear strength of the footing on granular pile was more than 23, while this ratio was far less for the footing on clay without granular pile. It was further observed that the extent of vertical movement in the granular pile was limited to 4 times the diameter of the pile. It was also indicated that the granular pile failed in end bearing if the length of the pile was less than 4 pile diameter before the bulging would take place. The limiting load was predicted by results of plasticity theory.

Hughes, Withers and Greenwood (1975) have presented interesting data on the behaviour of a single granular pile subjected to vertical load upto its ultimate capacity. The granular test pile 66cm in dia. and 10m deep was constructed at convey Island (UK) in soft clay, having undrained shear strength as 0.22 kg/cm^2 (22 KN/m^2), the angle of internal friction for the pile material was $\emptyset' = 38^o$ and the limit pressure σ_{rL} was 120 KN/m^2. The piles were formed by vibro jetting a vibro-float into the ground and compacting an imprated gravel into the resulting hole. The ultimate pile capacity was predicted by Hughes and Withers (1974) equation

as given below:

$$q_{ult} = K_p (\sigma_{ho} + 4C_u) \qquad ...2.37$$

where

q_{ult}, σ_{ho} & C_u are all expressed in t/m^2.

The total axial stress q_{ult} was found to be 500 KN/m^2 ($50t/m^2$). This was equivalent to a total load of 171 KN (17.1 tonnes) on a 66cm dia. granular pile. The back fill material was uniformly graded 20mm-40mm gravel. However from the excavation of the piles at Convey site (out of 1000 piles), the average diameter of the constructed piles was 73cms i.e. an increase of about 10 per cent over the original pile diameter.

Taking average pile diameter as 73cm, C_u = 22KN/m^2 (0.22 kg/cm^2), $\emptyset' = 38^o$, the ultimate granular pile capacity have been predicted and compared with the observed pile capacity Table 2.4 and Fig 2.14.

Table 2.4 Computed and Observed Granular Pile Capacity (after Hughes et.al 1975)

Investigation methods	Computed ultimate pile load (t) (KN)	Observed pile load (tonnes) (KN)
Cambridge Pressuremeter	20.9 (209)	Tending to 22.0 (220)
Menard Pressuremeter	34.6 (346)	
Calculation from site Investigation	22.8 (228)	
Cementation approach	12.3 (123)	

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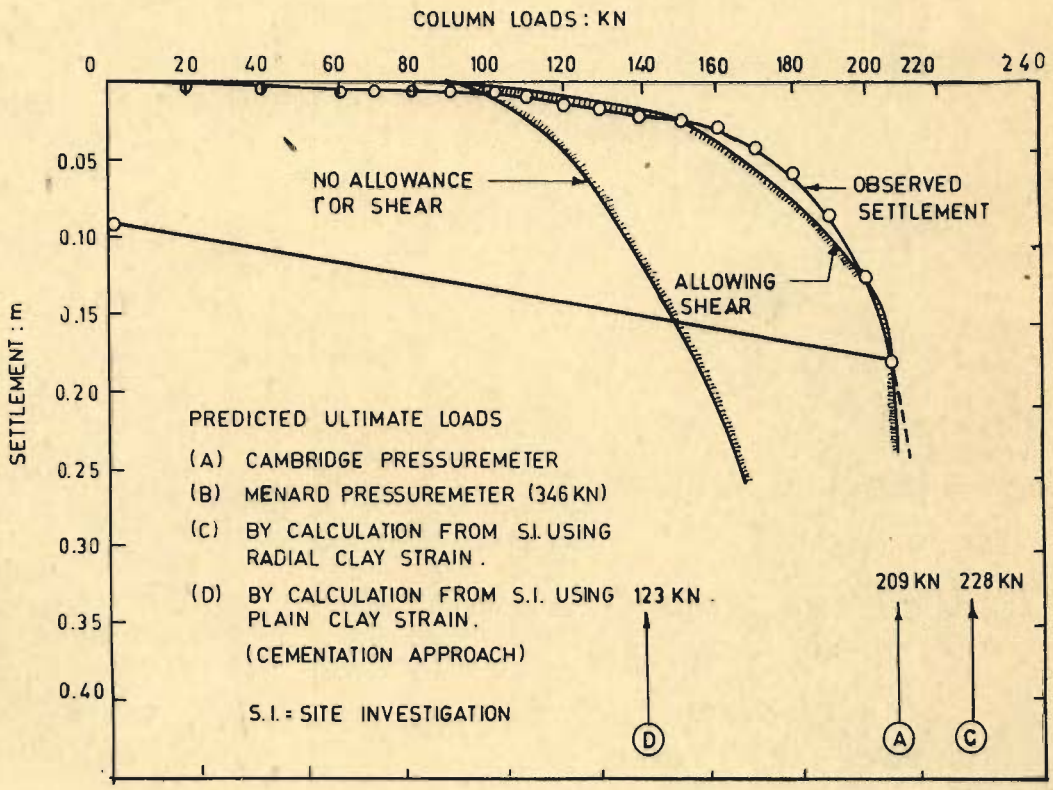
The shear along pile-soil interface may be found by substracting total shear on the boundary from the applied load above the horizon.



FIG·2·14. PREDICTED LOAD-SETTLEMENT CHARACTERISTIC FOR 730 mm DIAMETER PILE COMPARED WITH OBSERVED RESULT.
(AFTER HUGHES, WITHERS AND GREENWOOD 1970)

The authors recommend the consideration of shear along the pile-soil interface. A comparision of the load-deformation curves after making allowance for the shear and without is shown in Fig. 2.14

Baumann and Bauer (1974) collected data from the literature of case histories as shown in Table 2.5

Table 2.5 - Load Capacity of a Single Granular Pile

Soil type	Compacted footing area (sq.m)	Contact pressure t/m^2 (KN/m^2)	Total load tonnes (KN)
Granular soil	2.0-2.5	40-50 (400-500)	100 (1000)
Cohesive soils	1.5	20.0 (200)	30 (300)

Model experiments have been reported by Mokashi, Paliwal and Bapaye (1976) with a view to study the performance of a single column (granular pile) in soft cohesive matrix with and without surcharge on surrounding cohesive soils.

In the initial stages of testing, bees wax ($\emptyset$=0) simulating the behaviour of soft saturated clays under undrained conditions was used to check the suitability of experimental set up. The properties of the granular pile material and the clay are shown in Table 2.6

The test data confirmed the view that the soft clays could be strengthened by granular piles effectively and the bearing capacity increases from 2.5 to 4 times over the virgin clay stratum.

Table 2.6 - Properties of Clay and Pile Material

Marine Clay			Local Sand	
Consistancy limit percent	Undrained shear strength kg/cm^2	Particle size percent	Particle size	Angle of shearing resistance degrees
LL 61	0.192	Silt and clay 71.8	1.2mm to 4.76mm	41^0
PI 19		Very fine sand (0.02mm-0.2mm) 25.4 Sand 2.8 (greater than 0.2mm)		

Effect of Precompression

The vertical deformation of the granular pile under a given axial loading is directly related to the magnitude of the radial displacement of the surrounding clay. This radial deformation of the clay could be reduced significantly by preinducing each layer by precompression so that non-recoverable deformation takes place and only elastic deformation will occur at the time of the re-stressing the pile. Based on this idea, Mokashi et.al (1976) conducted model tests on granular piles in cohesive soil in the laboratory by precompressing each layer to a stress of 75 per cent of the ultimate anticipated pile load capacity. The results exhibited that there was reduction in initial settlement by 50 percent and 15 percent increase in ultimate load carrying capacity, when compared with tests in unprecompressed clay deposits.

Effect of Partially Penetrating Piles

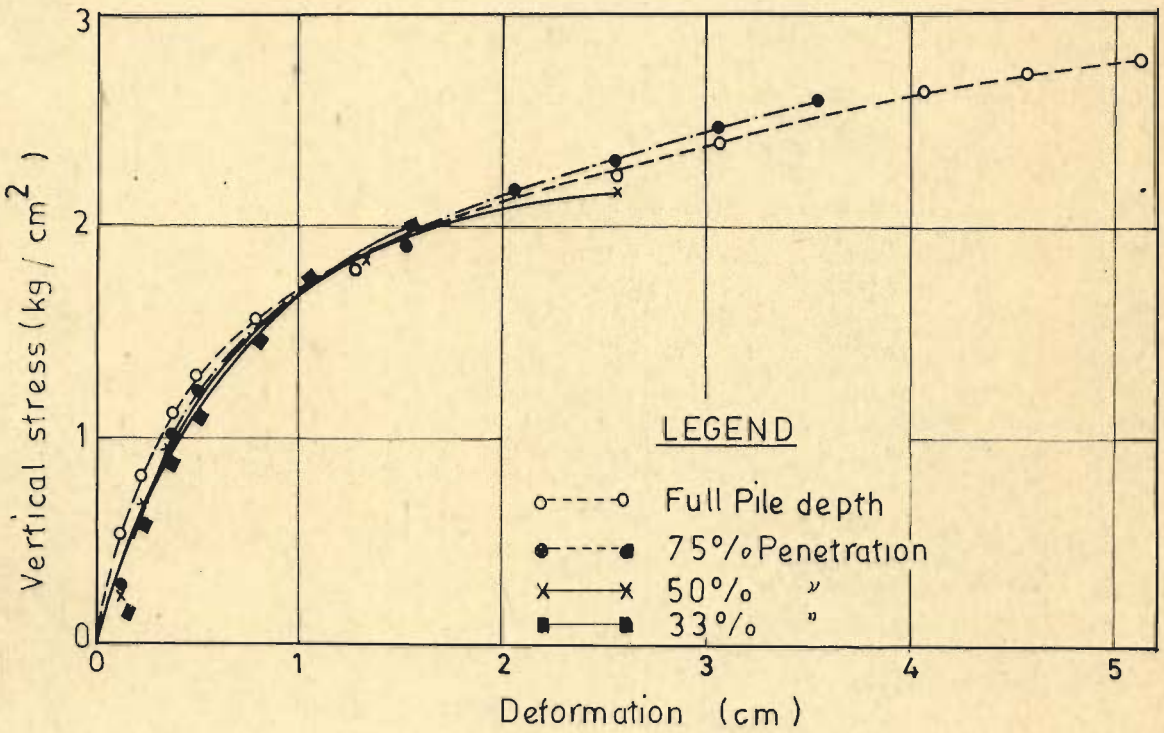
Since the bulging of the pile was observed to be limited to 100mm

from the top of the pile ($L/D = \frac{100}{35} = 2.85$) which demonstrated that full length of the pile was not stressed at failure. Tests with depth of piles equal to 33,50,75 and 100 percent of the full pile length, Fig. 2.15 indicated that the stress deformation behaviour of pile in cohesive soils were independent of pile depth, (Mokashi et.al 1976)

These tests indicate that the critical pile length was less (2.85 dia) in comparison to 4 Dia. as observed by Hughes and Withers (1974). Also the length of the pile does not seem to have any effect on granular pile capacity.

Rao and Bhandari (1979) reported a field study on the stress deformation behaviour of loose to medium dense non-cohesive soil deposit reinforced with granular piles. Benefits of reinforcement are determined by in-situ load testing of soil granular pile systems having piles cast singly or in groups of two, three and four piles, and comparing the results with those relating to the ground without granular piles. The field study has been extended to examine and quantify the added advantage of skirting the granular pile (by providing a mild steel pipe around the granular pile upto a depth of 4 to 5 pile dia). Sub-soil investigations revealed a soil deposit of SM-ML group upto 3m depth, with an average standard penetration test value of 8. The stratum was underlain by a ML-CL group upto 6m depth, penetration resistance remaining practically unchanged. Between 6m and 12m depth the soil of SC group was encountered. The penetration resistance in this layer abruptly increased from 20, below 6m depth to 40 at 12m depth. The ground water table was 1.5m below the average ground surface.

The granular piles 15cm dia. and 3m long, were cast using 18-20mm



FIG·2.15. STRESS DEFORMATION CURVES FOR PARTIALLY PENETRATING COLUMNS (AFTER MOKASHI et.al 1976)

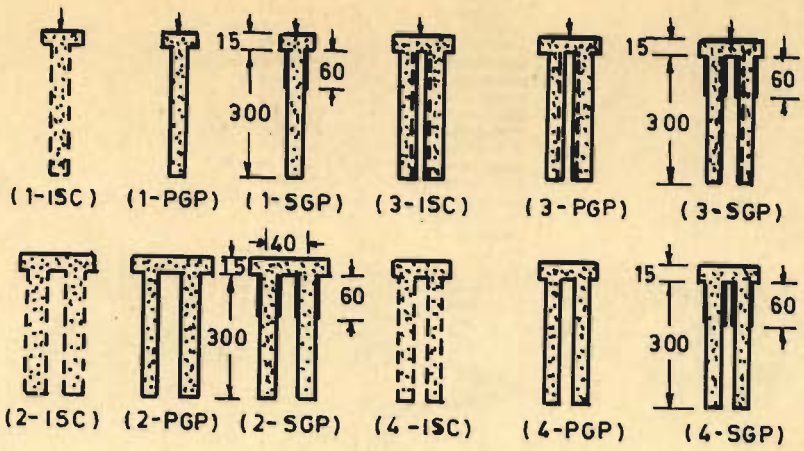


FIG. 2.16. SUMMARY OF INSITU TESTS
(AFTER RAO AND BHANDARI 1979)

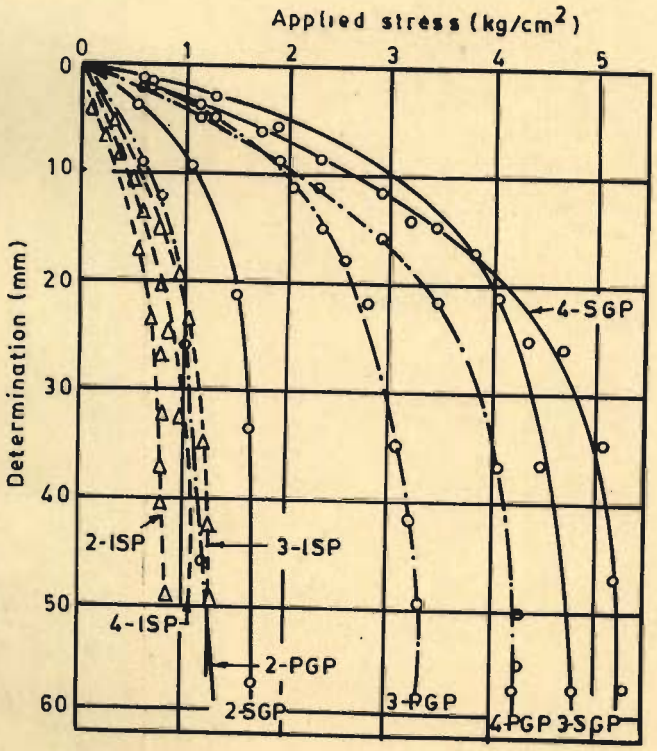


FIG. 2.17. STRESS DEFORMATION CURVES FOR SINGLE AND GROUPS OF PILES, PLAIN SKIRTED. (AFTER RAO AND BHANDARI 1979)

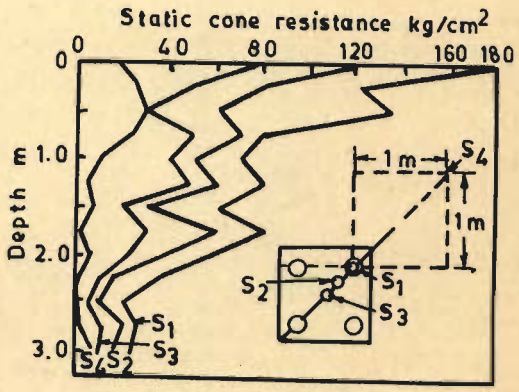


FIG. 2.18. STATIC CONE TEST RESULTS AFTER CASTING OF GRANULAR PILES
(AFTER RAO AND BHANDARI 1979)

stone aggregate and coarse sand compacted with the impact of 125 kg internal hammers in layers of 30cm thickness. While casting the groups, the granular pile spacing was maintained at 3 diameters constant stress method was used to load test the soil-granular pile system. The installed pile diameter was found to increase from 15cm to 20cm, an increase of about 30 percent. A summary of the tests conducted is shown in Fig. 2.16. The stress deformation behaviour of soil as well as soil-granular pile system is shown in Fig. 2.17. The improvement in strength of soil surrounding the pile is shown in Fig. 2.18. The test results showed that the ultimate bearing capacity of the virgin ground, when reinforced with granular piles plain or skirted showed an improvement of 110 percent for single skirted granular pile, 280 percent for group of three and 375 percent for group of four. The corresponding values for plain piles varied from 60 to 280 percent. The deformation against the ultimate loads for various tests were found to vary between 19 to 28 percent of the pile diameter.

The stress deformation behaviour of the soil granular pile system changed from an elastic state to elasto plastic state corresponding to a deformation of 7.5 percent of the installed pile diameter and appears to be independent of number of piles in a group. The skirting was found to augment the peak loads from 25 to 44 percent. The efficacy of granular piles systems in non-cohesive soils as a technically viable alternative to other ground strengthening methods, was pointed out.

Madhav (1981) conducted model tests in the laboratory on single granular pile with varying length (L), diameter of the pile (d_p) ratios. Tests were carried out in standard CBR mould.

Locally available silt was used in the tests. The properties of material are given in table 2.7

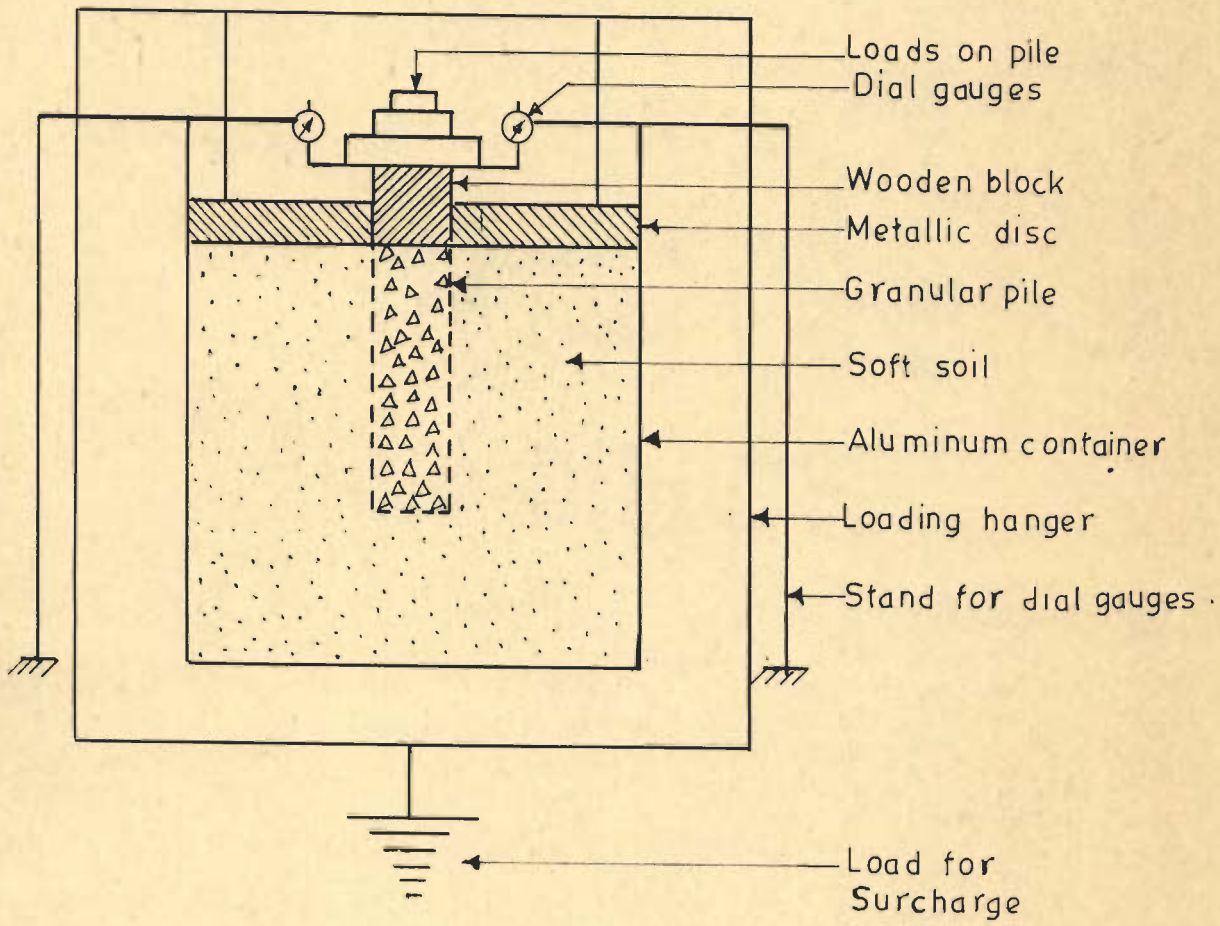
Table 2.7 - Properties of Materials (Soils & Pile)

Kanpur silt		(Percent)	Footing dia d_f (cm)	Pile properties		
				Length (cm)	Diameter (cm)	Material
LL 34	Silt	71.6	2.5	10	2.5	Well graded
PL 16	Clay	17.0	and 5.0	15	2.5	Sand gravel mixture
PI 10	Sand	11.4		20	2.5	

The oven dry soil was powdered thoroughly mixed and consolidated in layers or rammed through some number of blows. The container was then soaked in water for a week. Then the granular pile was installed by driving a 2.5cm dia. thin wall pipe, the clay inside the pipe was scooped. This was followed by filling the hole by granular material while the tube was pulled out gradually.

To obviate the effect of container from a small CBR mould tests were also carried out in circular drum. The loading arrangement is shown in Fig. 2.19. The tests were carried out on footings 2.5cm and 5cm dia. on un-reinforced soil and reinforced soil. In the CBR mould, the load was applied through constant rate of penetration method and the load was measured for a particular deformation as in triaxial testing device. However, in the large aluminium container the loads were applied through static loading giving incremental load method.

The test results are summarised in Table 2.8.



FIG·2·19 . SCHEMATIC DIAGRAM OF THE SET-UP.
(AFTER MADHAV 1981)

Table 2.8 - Effect of L/d_p, d_p/d_f on Load Capacity of Piles

L/d_p	$d_p/d_f=1$*	Increase in load due to pile (kg)	d_p/d_f =0.5**	Increase in load due to pile (kg)	Increase due to large footing (kg)
(1)	(2)	(3)	(4)	(5)	(6)
0	1.2	-	4.2	-	-
4	2.2	1.0	7.0	2.8	1.8
6	3.0	1.8	7.8	3.6	1.8
8	3.2	2.0	8.0	3.8	1.8

*d_p = 2.5cm, d_f = 2.5cm ** d_p = 2.5cm, d_p = 5 cm

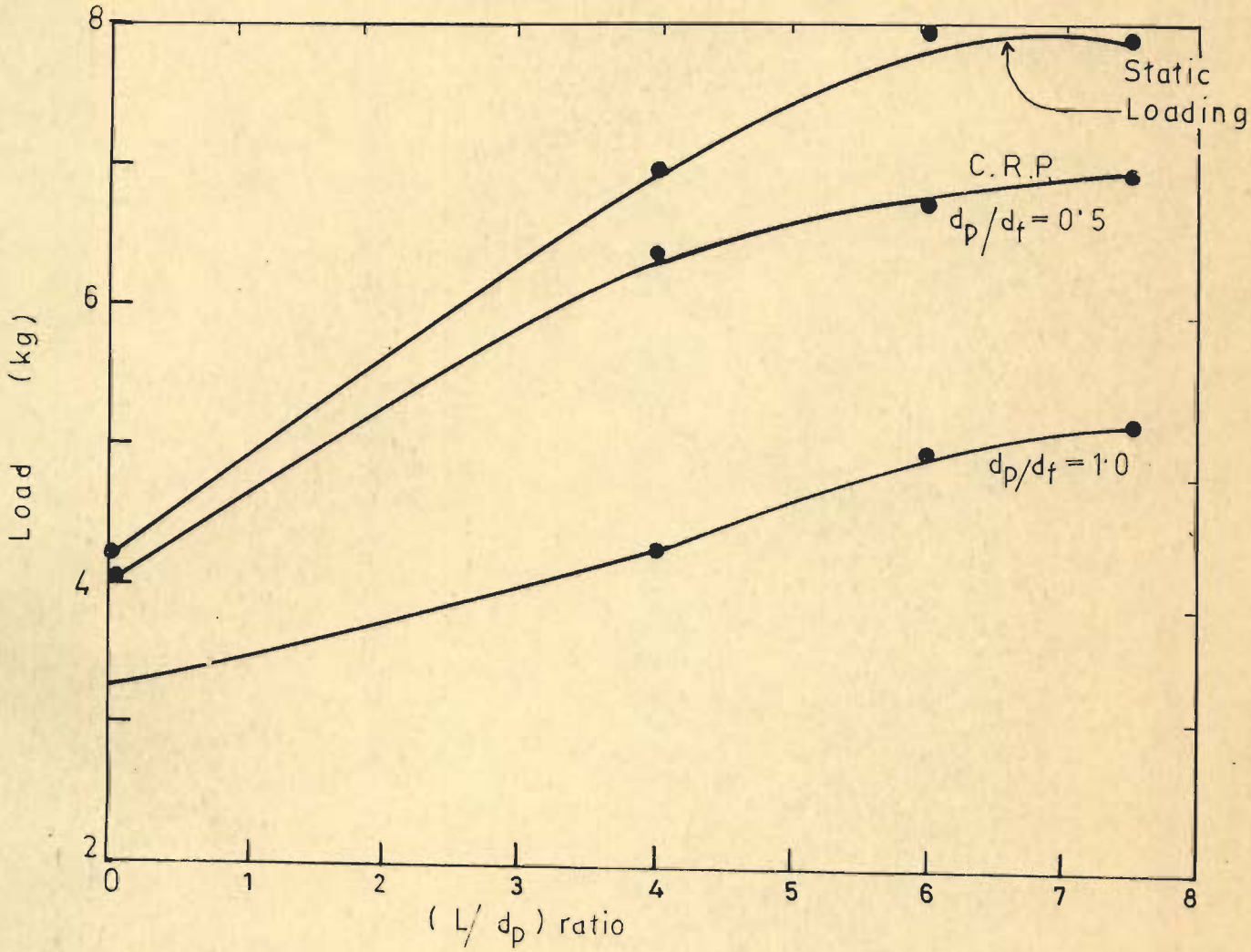
The test data revealed that:

(a) The granular pile in soft soil increase the bearing capacity from 60 percent to 155 percent as compared to un-reinforced soil.

(b) The surcharge placed around the pile, on the surrounding clay, increases the load carrying capacity of the pile by 25 percent for a surcharge of 0.8 kg/cm^2.

(c) If the soil is precompressed before pile installation and the same precompression load is maintained as a surcharge load, the load carrying capacity of granular pile further increases.

(d) The rate of loading was also found to influence the load carrying capacity of the reinforced soil. Slower is the rate of loading, higher is the load carrying capacity of the pile, Fig. 2.20.



FIG·2·20. EFFECT OF PILE LENGTH-DIA RATIO ON PILE CAPACITY.
(AFTER MADHAV 1981)

(e) The consolidation behaviour of partially penetrating pile reinforced soil, using Terzaghi Renaulic theory, and adopting a finite difference techniques, the dissipation of a pore pressure in granular pile reinforced soil has also been studied. The average degree of the consolidation is estimated for four different boundary conditions.

Greenwood (1970) carried out footing tests on treated (reinforced) and untreated (un-reinforced) ground at three sites, Port Talbot, Ardeer and Dunham Park (UK) having poorly sand deposits. The standard penetration value for Dunham Park site and one inch dynamic cone penetration resistance for Dunham Park site and Ardeer sites are shown in Fig. 2.21. Footing sizes were 1.3m and 1.5m square in shape and were placed 0.6m and 0.9m below ground level. Typical load test results for Dunham Park site are shown in Fig. 2.22 showing marked difference between treated and untreated ground. Similar results are shown at Port Talbot site also, Fig. 2.23.

In situ load test results have been reported by Datye and Nagaraju (1981) at Raoli and Kandla sites. The granular piles were constructed by ramming process using deep hammer or heavy diesel hammer. The filling of stone aggregates was carried out by pouring through a hopper into the cased borehole and casing lifted in stages known as rammed piles while in other case the granular material was pre-assembled into a dispensible sheath, fabricated out of bamboo vertical strips and horizontal ties with organic fibres. Thus the dispensible sheath filled with granular material was lowered into the borehole and rammed, this was known as preassembled process. The diameter of the installed piles at Raoli site was 62.5cm by ramming process and 75cm by preassembled process, whereas at Kandla site the respective

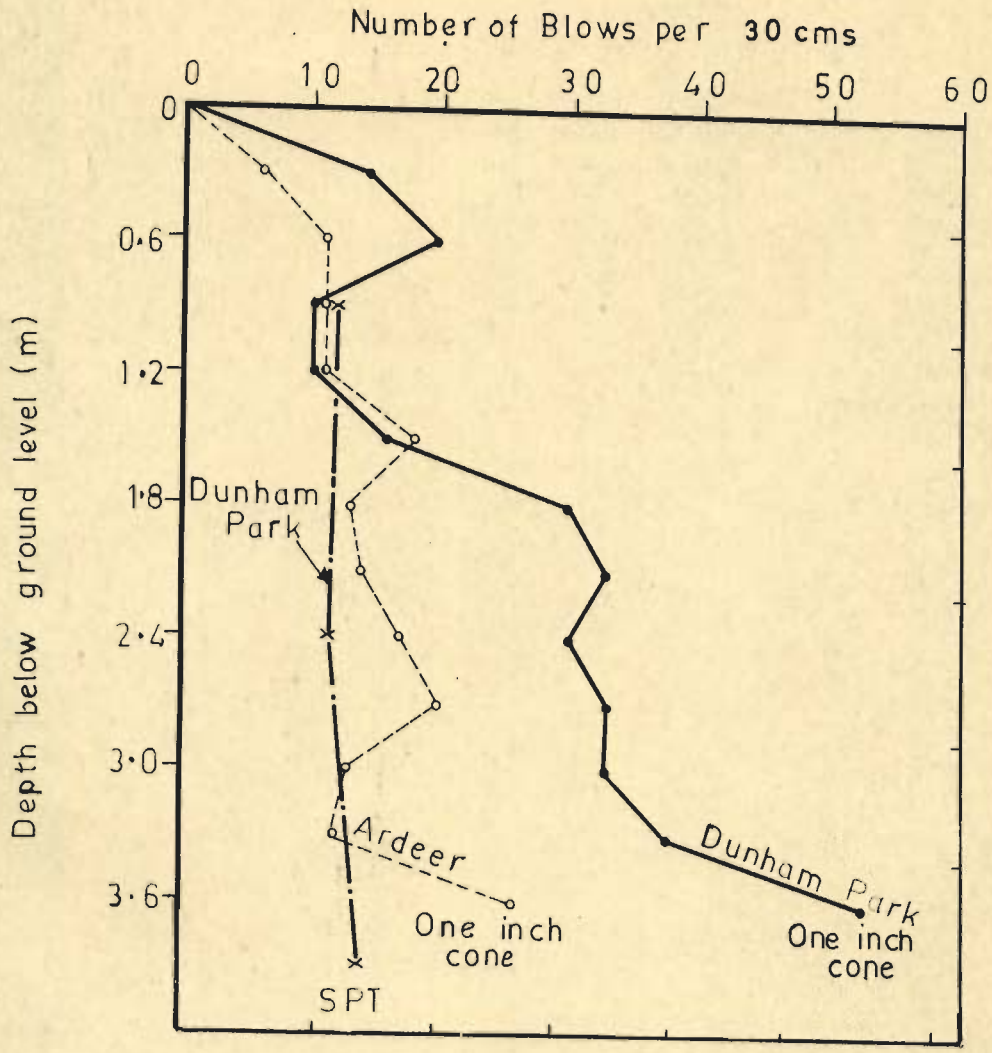


FIG. 2.21. PENETRATION RESISTANCE VALUES AT ARDEER AND DUNHAM PARK SITES
(AFTER GREENWOOD 1970)

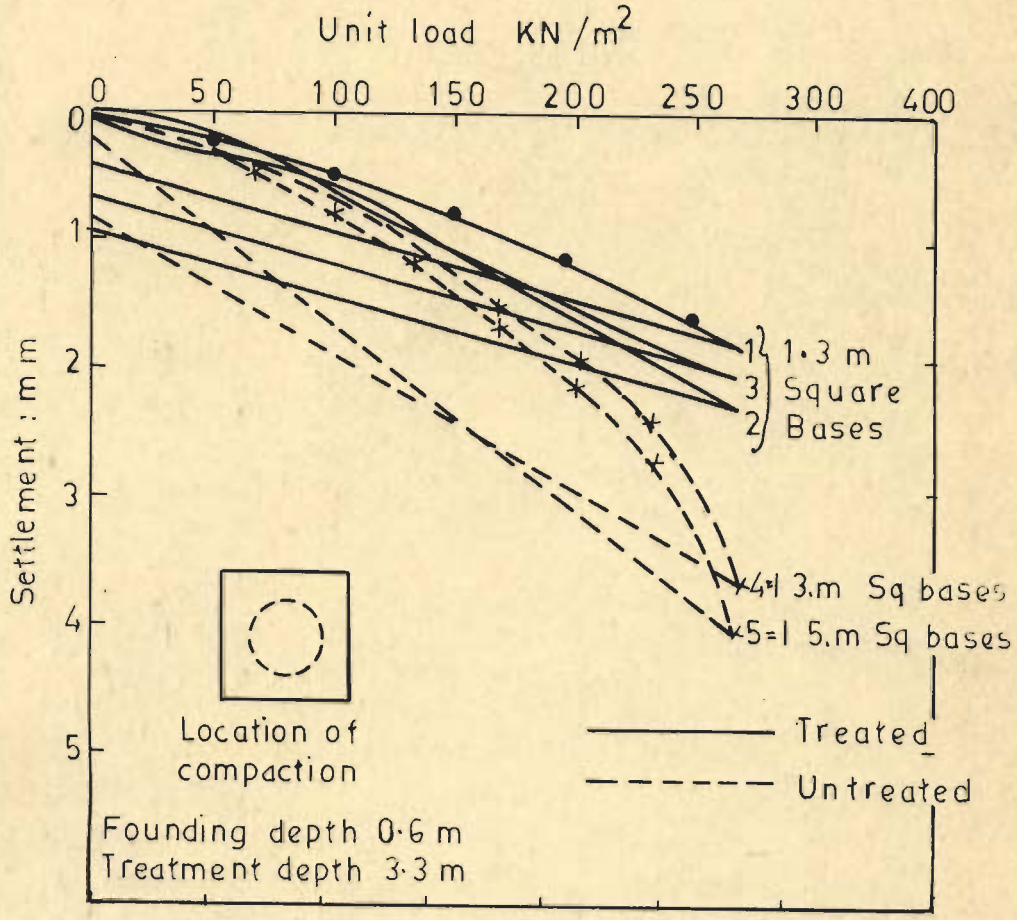


FIG:2:22. DUNHAM PARK: LOADING TEST RESULTS ON TREATED AND UNTREATED GROUND.
(AFTER GREENWOOD 1970)

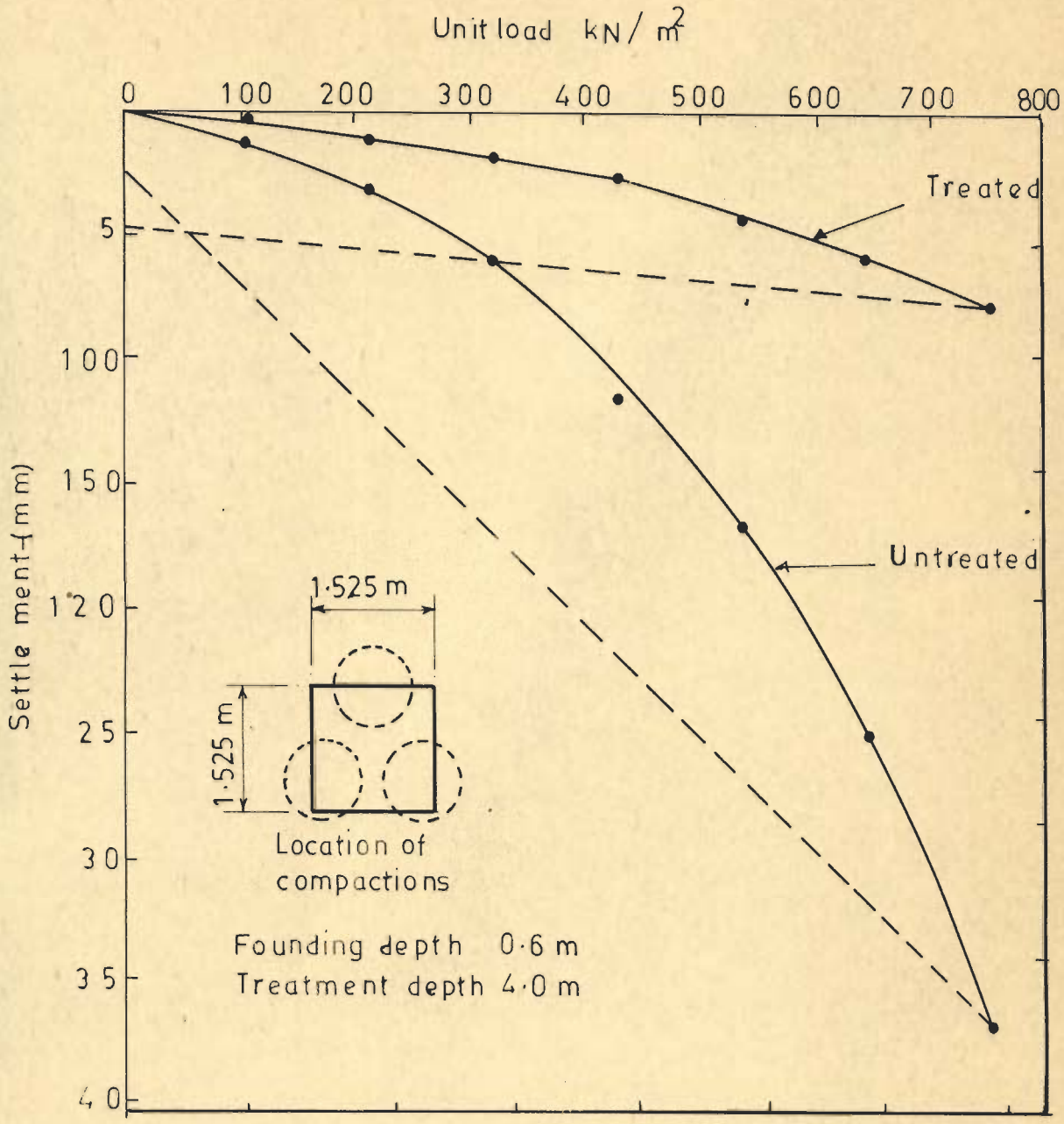


FIG ·2·23. PORT TALBOT LOADING TEST RESULTS ON TREATED AND UNTREATED GROUND (AFTER GREENWOOD 1970)

diameters were 40 and 60cm. The length of the piles were 10m compaction was done at 4,4 and 2m. A set of 2cm for 8 blows were applied at each stage.

2.3 SETTLEMENT OF GRANULAR PILES

One of the requirements for the satisfactory behaviour of foundations is settlement which should be within certain permissible limits. In view of this the computation of settlement of granular pile foundations is of paramount importance. The various methods available for computing settlement of granular piles have been reviewed in the following sections.

2.3.1 Analytical Approaches

Finite Element Method

Finite element and finite difference methods have been employed for theoretical prediction of magnitude and rate of settlement of soft clays reinforced with granular piles installed over a large area in a regular pattern (Balaam, Poulos and Brown 1977). In this case except near the edge of the loaded area, the behaviour of all the pile - soil units is virtually the same. These units are subjected to uniform vertical pressure such as might be imposed by a flexible raft or an embankment. The analysis has been developed for a single granular pile in which a pile-soil interface strength is specified and both, the pile and soil has been treated as elasto-plastic material Balaam etal (1976). A regular pattern of vertical granular piles may be analysed by considering a typical pile-soil unit. In a triangular arrangement, each pile may be considered as having a hexagonal zone of influence Fig.2.24 and an equal area of influence is given by a circular zone of diameter $d_e = 1.05S$, where S is the pile spacing.

● Granular piles
S Pile spacing
— Zone of influence

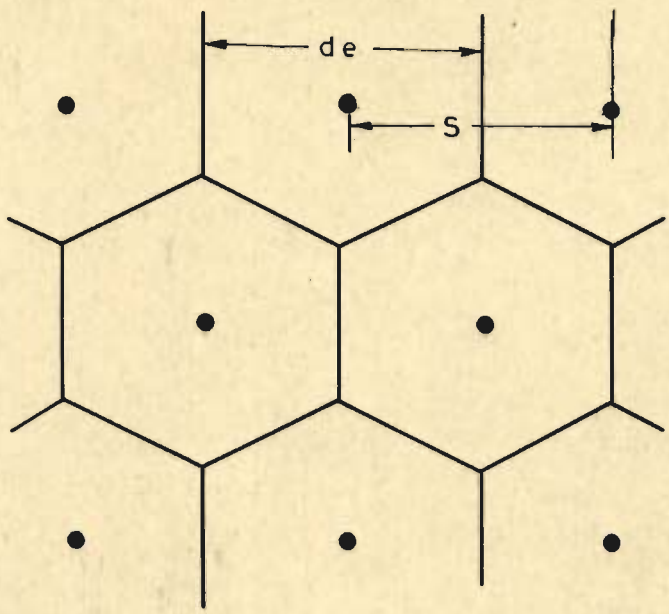


FIG : 2.24. PLAN OF REGULAR TRIANGULAR PATTERN OF GRANULAR PILE USED FOR FINITE ELEMENT ANALYSIS
(AFTER BALAAM et.al 1977)

For a square pattern d_e = 1.14S. The pile-soil units analysed are cylinderical with diameter as d_e and are divided into finite elements.

The main assumptions made in the analysis are:

(a) The drained Poission's ratio of the pile, material μ'_s and soil μ'_s is 0.3

(b) At any point the drained and undrained elastic modulii of the soil E_s and E'_s are related by the Equation 2.38

$$E_s = \frac{3}{2} \frac{E'_s}{(1+\mu'_s)} \qquad ...2.38$$

(c) At any depth the ratio E'_p/E_s is taken as 20 unless other wise specified

(d) Any variation of modulus with depth is taken as linear

(e) The form of loading is considered as uniform vertical pressure such that as might be imposed by a flexible raft or embankment.

A weak sub-soil stratum under drained conditions and reinforced with granular pile is subjected to a uniform stress (q) applied over a large area considering the various variables such as shear modulus of clay (G_o), depth of the compressible layer (h), length of the granular pile (L), permissible settlement (δ), radius of zone of influence r and radius of the pile (r_w). The non-dimensional term ($\delta G_o/hq$) is related with (r/rw).

As the loading pressure (q) is increased the mechanism of load transfer is assumed as local yield in the form of contained plastic flow will develop within the pile and or the soil. The

extent to which this causes settlement compared with the settlement from an elastic analysis under drained conditions is shown in Fig. 2.25.

The material properties for h/d = 5, de/d = 2, L/h=0.5 are shown in Table 2.9.

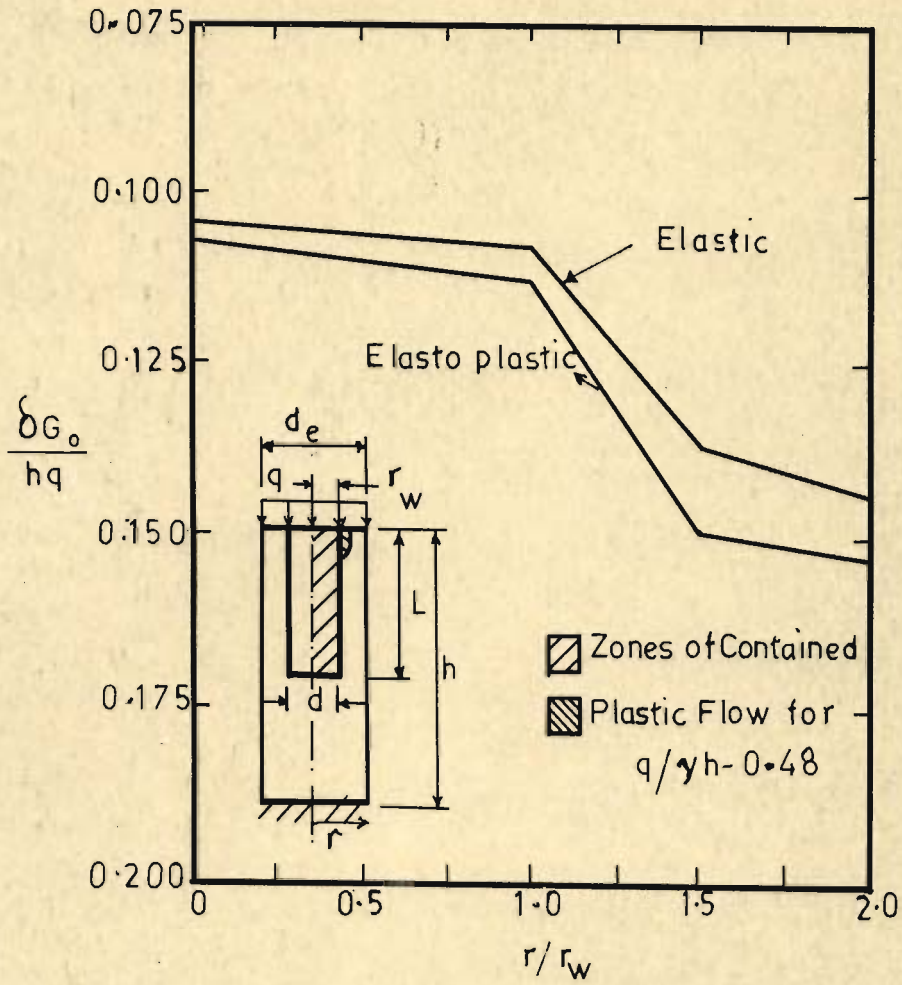
The results of the analysis are shown in Figs 2.26 through 2.28 based on elastic analysis for drained settlements as ratio of the settlement occuring when there is a pile in pile-soil unit to the settlement which would occur if there were no pile.

Table 2.9 - Material Properties

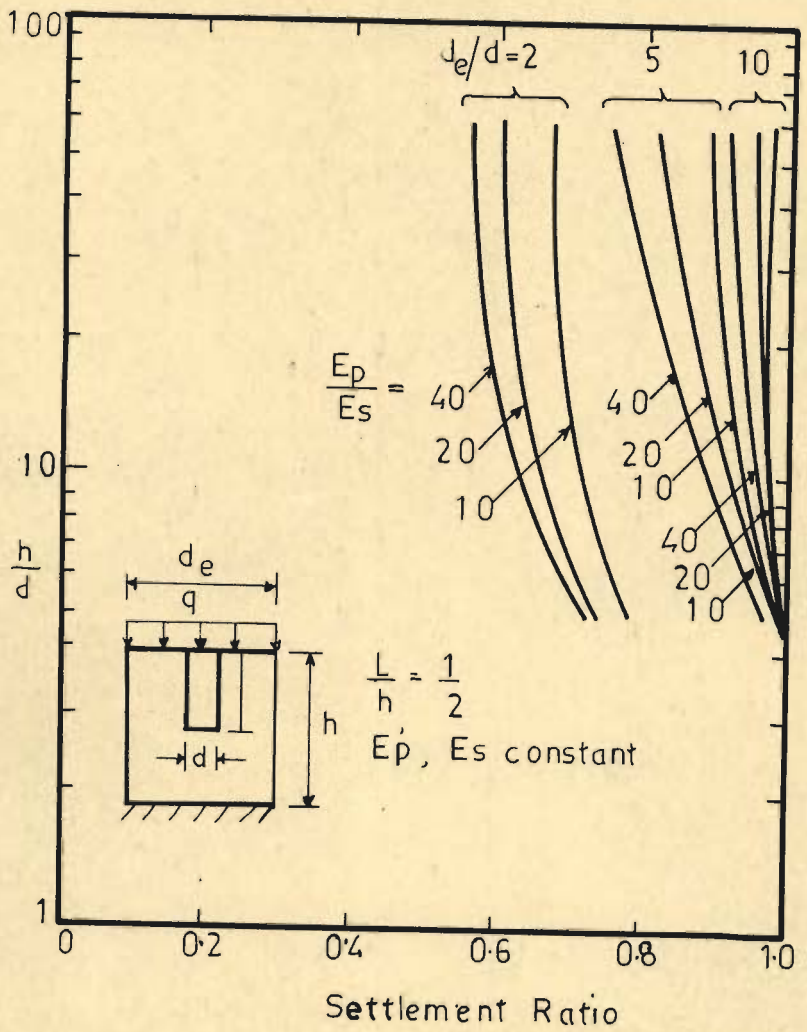
Property	Pile material	Soil
Coefficient of horizontal effective stress K_o at rest	1.0	1.0
Drained cohesion C'	0	0
Drained angle of internal friction Ø'	40°	30°
Angle of dilatancy	20°	10°
Ratio of surface young modulus E(0) to modulus at depth E(h)	0.5	0.5

Effect of Ratio of Pile/Soil Modulii on Settlement

Fig 2.26 shows the effect on soil settlement with varying ratios of pile and soil modulii Ep'/E_s when E'_p and E_s are constant with depth and L/h (pile penetration ratio is 0.5) while the values of ratios E'_p/E_s has some effect on the results but are not of major significance. The subsequent results are presented for E'_p/E_s = 20 which is likely to be representative of the ratio's encountered in the practice.



FIG·2·25. EFFECT OF LOCAL YIELD ON DRAINED SURFACE PROFILE (AFTER BALAAM et al 1977)



FIG·2.26. SETTLEMENT VARIATION WITH CHANGES IN E_p'/E_s RATIO. (AFTER BALAAM et al 1977)

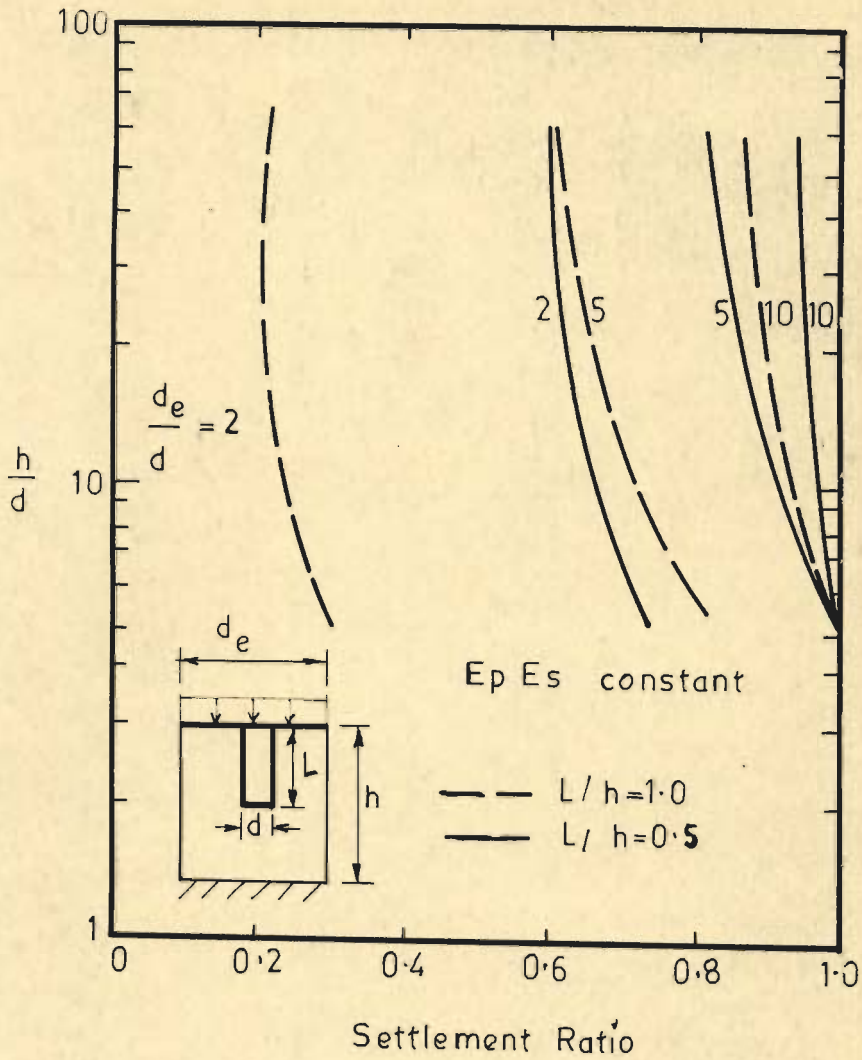


FIG. 2.27. SETTLEMENT VARIATION WITH CHANGES IN DEGREE OF PENETRATION OF THE PILES.
(AFTER BALAM et al 1977)

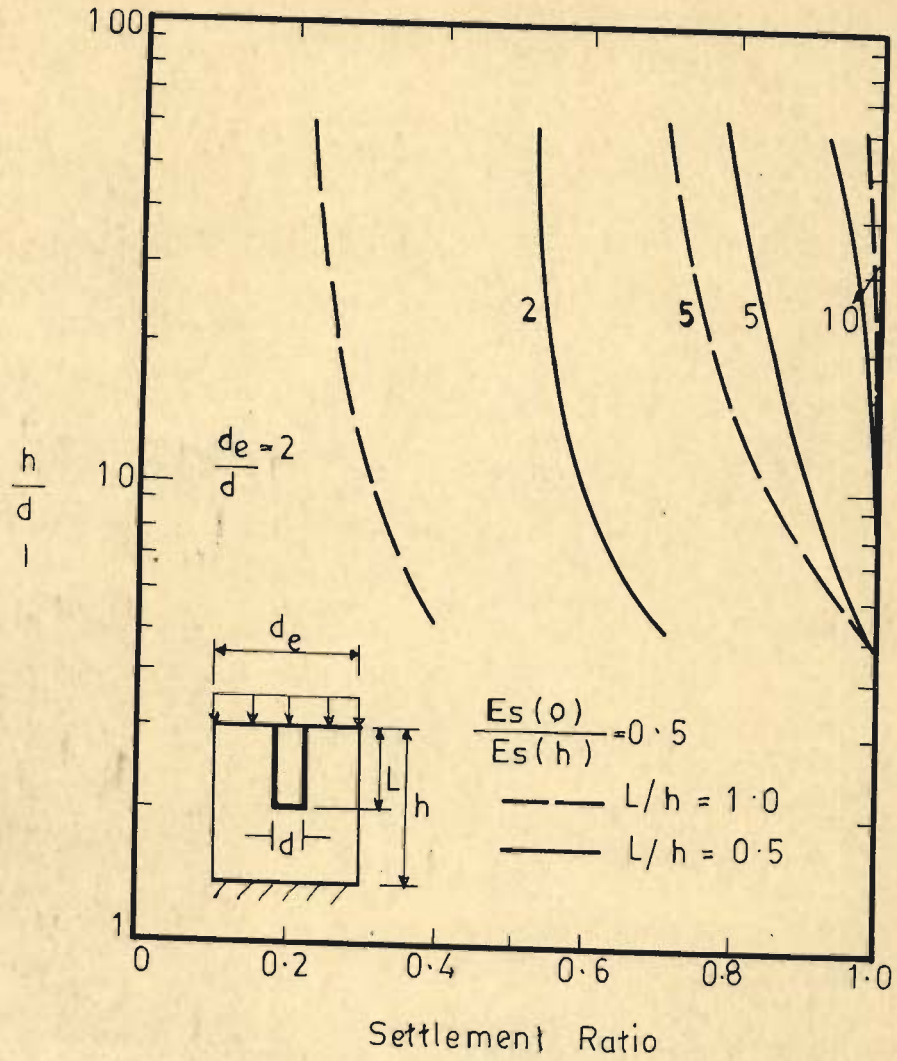


FIG.2.28. SETTLEMENT VARIATION FOR LINEARLY VARYING MODULUS IN PILE AND SOIL.
(AFTER BALAAM et. al 1977)

Effect of Pile Penetration on Settlement

Fig 2.27 shows the effect of reinforcement on soil settlement of increasing the effect of pile penetration (L/h) from 0.5 to 1, when E'_p and E_s are constant with depth.

Effect of Increase of Pile Penetration on Settlement

Fig. 2.28 shows the effect on soil settlement when L/h increases from 0.5 to 1, E'_p and E_s increase linearly with depth. This increase is 50 percent at half the layer depth and 100 percent at the layer base.

In case of L/h = 0.25, even if piles are closely spaced, no significant reduction in this is observed.

On the basis of the above analysis, it has been concluded that (a) when the granular piles are installed in a regular pattern over a large area significant reduction in settlement occur only if the piles are closely spaced ($\frac{S}{d}$ 5) and usually if the piles are installed to the full depth of the consolidating layer (b) the effectiveness of the granular piles in increasing the rate of consolidation increases significantly by simultaneous increase in pile penetration and decrease in pile spacing (c) estimates of pptimal spacing, diameter and degree of penetration of the pile can readily be obtained from the results presented.

2.3.2 Experimental Methods:

(a) Settlement Based on Pressuremeter Test

Hughes et.al (1975) demonstrated that the granular piles constructed at convey sites (UK) were 10m deep and penetrated through the soft clay into sand or silt strata. The piles were

considered to be fixed at 7m depth, where the strata changed. Since the critical length (Lc) of the pile as discussed in Art. 2.2.2 was less than 7m depth hence the end bearing failure was ruled out.

The main assumption (Hughes and Withers 1974) in computing the settlement of the granular piles based on total stress versus radial strain relationship as obtained from pressuremeter tests at various depths is that the volume of the granular pile remain constant during vertical deformation and the pile expands radially. The pile is divided into layers of thicknesses $H_1, H_2 \ldots\ldots H_7$. The total vertical settlement S is given by the following equation:

$$S = S_1 + S_2 + S_3 + \ldots\ldots\ldots + \ldots S_n \qquad \ldots 2.39$$

where settlement at any layer (S_n) is given as:

$$S_n = 2\, H_n \frac{\delta ro}{r} \qquad \ldots 2.40$$

where $\delta r_o / r$ is the radial strain obtained from pressurmeter tests. H_n is the thickness of the layer considered.

(b) Settlement of Pile Group

In practice, granular piles are always used in groups. The load is applied on pile and soil both. Under the design load, the clay in between the granular piles consolidates, which results in increasing the radial stiffness of the clay. While studying the settlement of granular pile group under a raft Hughes and Withers (1974) assumed that the behaviour of a typical pile within the group is the same as that of an isolated pile. It is also indicated that the increase in the radial stress in the clay between the piles does not have a marked influence on

the ultimate capacity of the pile and thus consolidation of the clay is ignored.

Granular piles having 1m diameter, 4m length at a spacing of 2.5m under a raft is shown in Fig 2.29 on a four meter square grid. The settlement characteristics are shown in Fig. 2.30 which indicate the settlement pattern of the raft subjected to a stress intensity q. The settlement of the reinforced clay is denoted as Δ_1 and that of virgin clay is Δ_2.

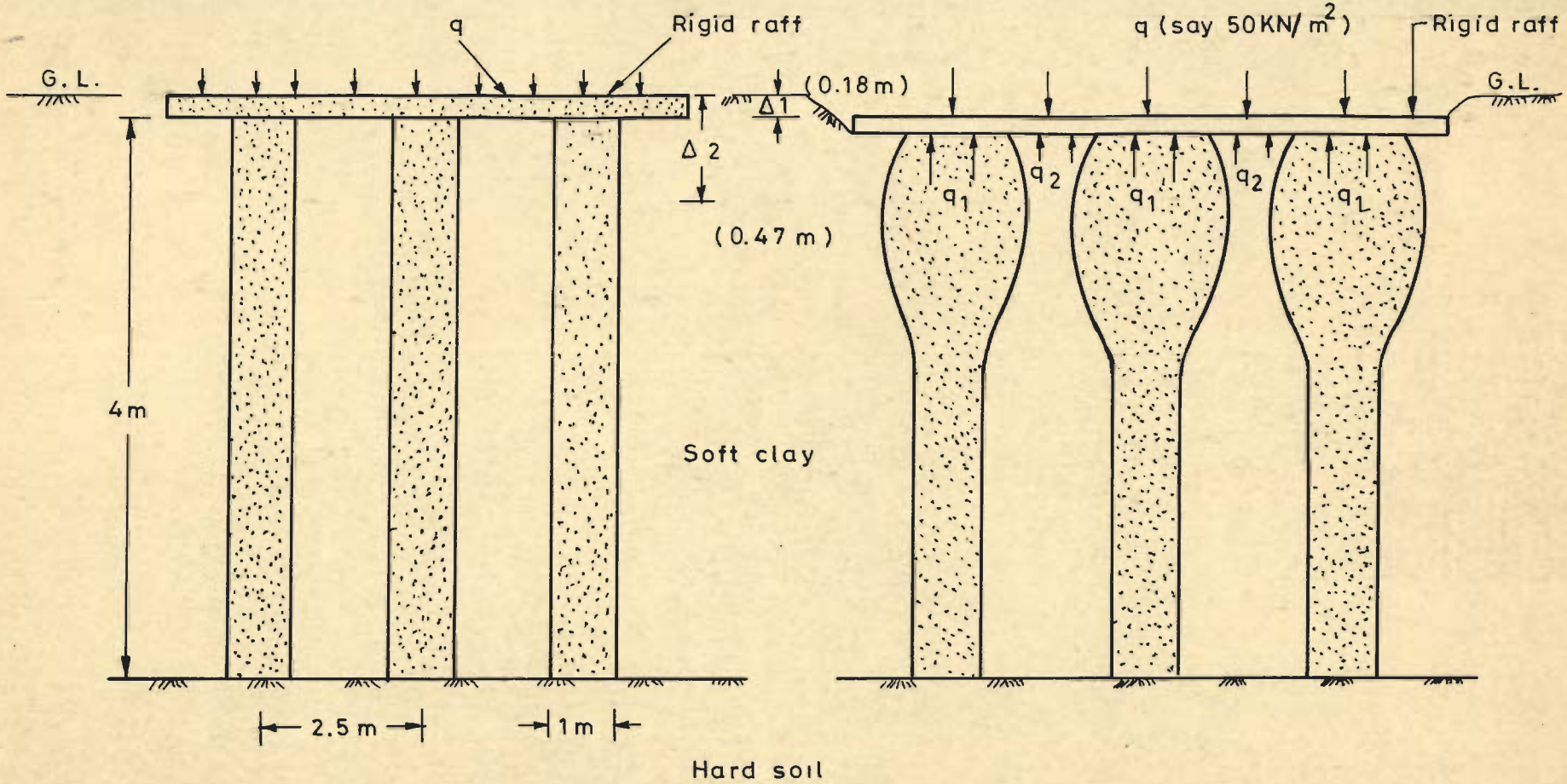
If the raft is subjected to a stress q equal to 50KN/m^2 (0.5 kg/cm^2), the corresponding settlement of the reinforced ground Δ_1 is found from tests to be as 0.18m Fig. 2.30b (point A). For the same intensity of stress the settlement Δ_2 of the virgin clay deposit is observed as 0.47m. Fig. 2.30a (point D) The corresponding stress (q_1) on the granular pile and on the clay between the piles (q_2) were observed to be equal to 320 KN/m^2 (3.2 kg/cm^2) point B and 10KN/m^2 (0.1 kg/cm^2) point D in Fig. 2.30a It was concluded that the settlement of reinforced ground is reduced by 90 percent and at higher stresses 75 KN/m^2 (0.75 kg/cm^2), granular piles are not effective.

2.3.3 Empirical Methods

Hughes and Withers (1974) reported that the settlement of the ground reinforced with granular piles 20cm in dia. and 2m deep were reduced to one fourth. It is mentioned that this fact was known to French Engineers even in nineteenth century. Though the details regarding pile spacing etc. have not been provided.

(a) Greenwood's Settlement Chart

Greenwood (1970) recommended that in view of the complexity of



(a) GRANULAR PILES UNDER A LARGE RAFF

(b) GRANULAR PILES UNDER A LARGE RAFF DURING LOADING

FIG:2.29. BEHAVIOUR OF GRANULAR PILES IN GROUP
(AFTER HUGHES AND WITHERS 1974)

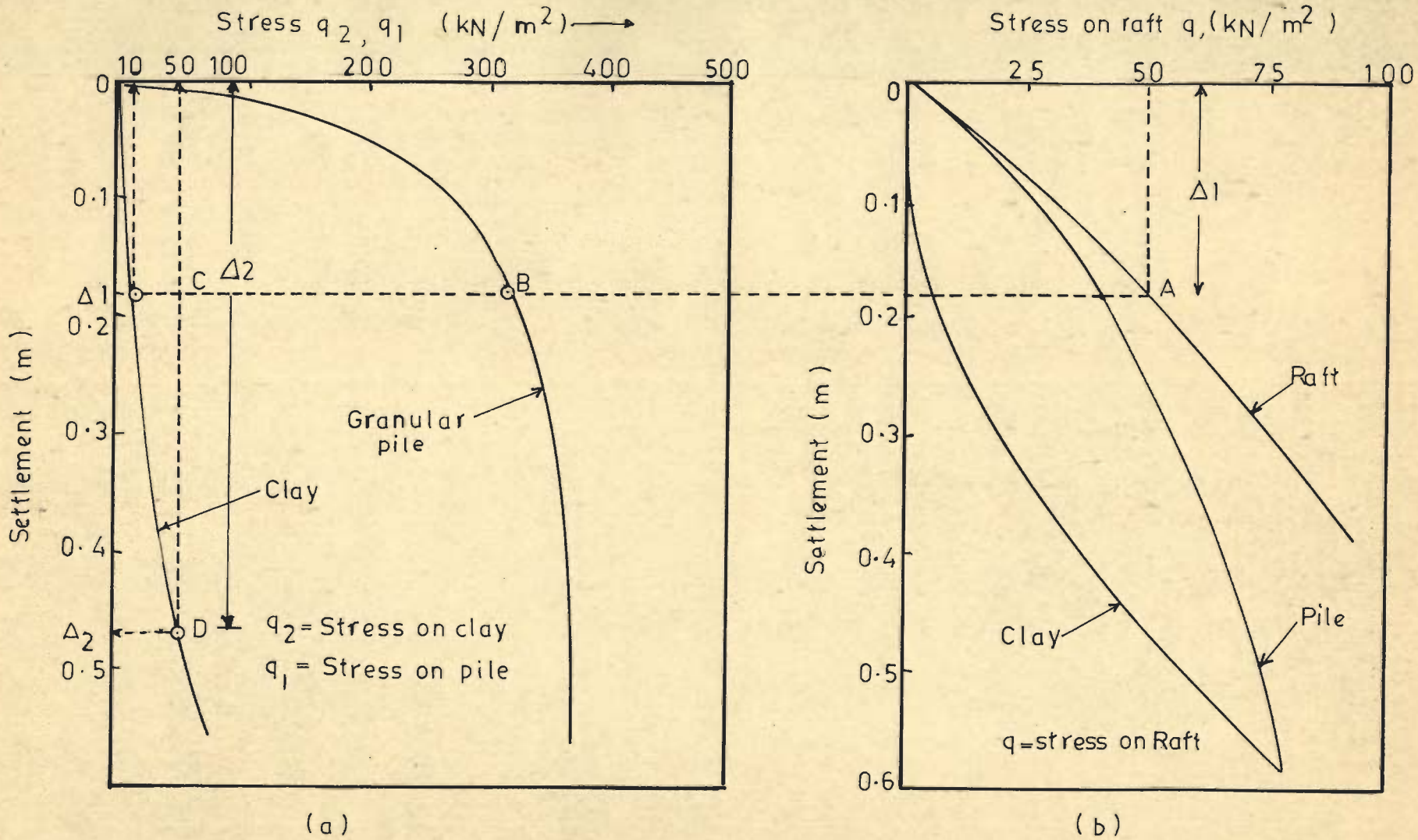


FIG. 2.30. SETTLEMENT CHARACTERISTICS OF GRANULAR PILE GROUP.
(AFTER HUGHES AND WITHERS 1974)

the problem and non-availability of a precise mathematical solution ensuring compatibility of the pile and clay deformation, the settlement of the ground reinforced by granular piles be estimated by empirical methods only. Settlement of soft clay reinforced with granular piles may be found from Fig. 2.31 for wide spread footings.

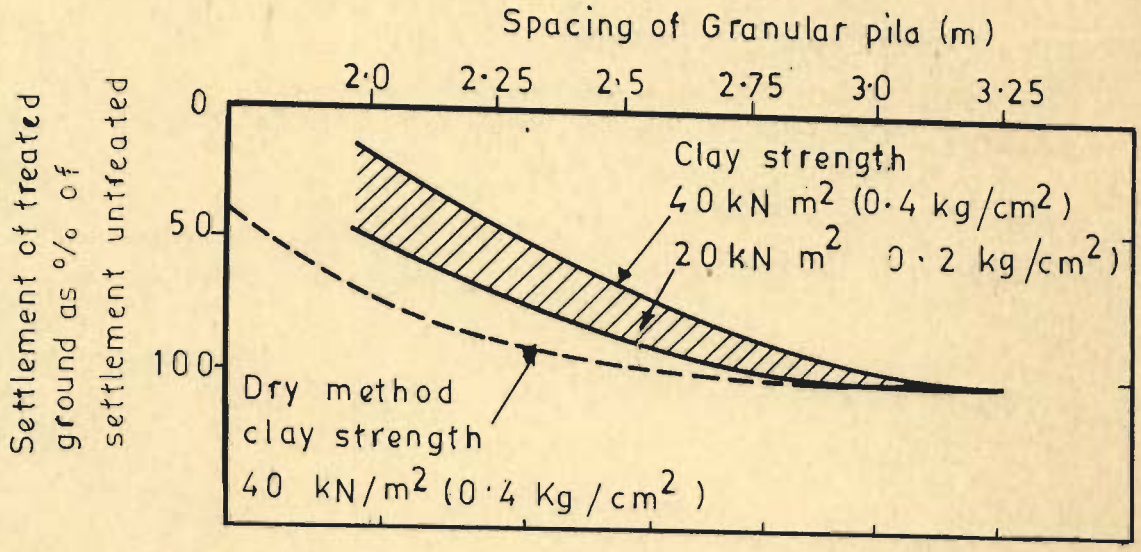
A soft silty clay strata, 4m thick overlying sand was reinforced with granular piles and a comparision between settlement of tanks with and without granular piles is shown in Table 2.10 (Penman 1977). Though the reduction in settlement is found to be 50 percent such a comparision, however, could be meaningful provided thickness of the compressible stratum rate and magnitude of tank loading etc. are identical.

Table 2.10 - Comparision of Settlement of Tank Foundation with and Without Granular Piles

Type of Foundation	Tank size (m)		Edge settlement (cm) (average)
	Diameter	Height	
Granular pile foundation	34.2	16.5	14.6-17.0
Pad Foundation	39.0	16.5	27.2

(b) Settlement at Ultimate Load

Hughes and Withers (1974) and Hughes et.al (1975) demonstrated through tests that the settlement of the granular pile at ultimate pile capacity is found to be equal to 58 percent of the pile diameter. This finding is based on their actual in situ tests at Convey site (UK). However, this value was found to be between 10-15 percent of the pile diameter on the basis of full scale field tests (Thorburn 1975).



Curves neglect immediate settlement and shear displacements

Piles assumed resting on firm clay sand or harder ground

FIG·2.31. SETTLEMENT DIAGRAM FOR GRANULAR PILES IN UNIFORM SOFT CLAY (AFTER GREENWOOD 1970)

(c) Settlement Based on Radial Strains

Thornburn (1975) proposed a method for settlement computation based on radial strain. The magnitude of the total settlement experienced by the structure may closely be approximated by the vertical strain at the top of the granular pile plus the compression of the soil below the layer of the reinforced ground. The vertical strain in the granular pile is equal to two times the maximum radial strain within the pile, the volume of the pile at working loads remaining constant. The radial strain can be determined from pressuremeter tests.

It is demonstrated from field tests on granular piles that the vertical displacement of the top of the pile may range between 5mm to 10mm and may increase to 30mm (Floss 1979). This value will be added to the computed settlement of the soil strata below the reinforced ground which is significantly stressed. This will give the total settlement and will also allow for vertical displacement of the layer reinforced with granular pile.

(d) Settlement Based on Material Properties

Baumann and Bauer (1974) considered stress-strain behaviour of granular pile material and the soft clay surrounding it for settlement computations

The immediate settlement (S_1) is obtained from the equation:

$$S_1 = \frac{H}{E_s} p_s \qquad \ldots 2.41$$

Where

H is the depth of granular pile compaction,

p_s is the load shared by the soil

E_s is the soil modulus of the surrounding soil.

The consolidation settlement S_2 defined as the vertical compression which will occur under a sustained load and which is associated with a decrease in voids of the soil.

The amount of settlement is given by:

$$S_2 = \sum_{0}^{H} \frac{\Delta\sigma}{E} \quad \Delta z \qquad \ldots 2.42$$

Where

E and Δ are drained modulus of deformation and increase in vertical stress at any depth Z below the foundation.

H is the thickness of compressible soil layer.

E is related to coefficient of volume compressibility (m_v) as

$$E = \frac{1}{m_v} \qquad \ldots 2.43$$

The vertical stress $\Delta\sigma$ may be estimated by using Boussinesq or Newmark (1942) chart within the compacted zone. It should be noted that at a depth $Z = 5\ r_0$ (r_0 is pile radius) 95 percent of the applied load are dissipated when the load is applied on the granular pile top only.

In most practical cases however, the footings are bigger than the diameter of the granular pile and therefore the actual stress increase with depth is higher than the theoretical increase and thus will yield larger settlement than calculated.

The method of settlement calculation proposed by Baumann and Bauer (1974) is an analytical model only. It usually under estimates the consolidation settlement mainly due to two factors such as, the distribution of the friction along the granular pile and hence the amount of load carried to the point. Besides,

overlap of stresses from adjacent granular piles is also difficult to evaluate.

(e) Settlement Based on Effective Areas of Piles and Soil

Nahrgang (1976) proposed that the settlement of improved ground reinforced with granular piles decreases in the ratio of total granular pile area to the total area provided the volume of the pile during deformation under load is constant. To minimise differential settlement the improved soil has to be loaded uniformly as far as possible. Further more approximately 0.5m surcharge layer is necessary to compensate for these settlements.

Prieba (1976) recommends that the settlement of normally consolidated soils characterised by an isotropic stiffness and without intermediate layers of sand lenses is reduced to a factor of 3 to 4 when reinforced with granular piles.

(f) Settlement Based on Compressibility of Pile Material

Initial settlement of the granular pile could be estimated on the basis of the compressibility of the crushed stones measured in the laboratory test as suggested by Marsal (1967). Datye and Nagaraju (1981) have indicated that the coefficient of volume decrease m_v ($0.0002m^2$/tonne) compared favourably with those obtained by Marsal (1967) from the laboratory compression tests. Such a comparision of compressibility is reported to be valid for the ratio of ultimate vertical pile capacity to lateral limit stress ratio $(\frac{\sigma_1}{\sigma_3})$ between 3 to 4. However, this may not be true for higher values of the ratios which actually found in practice.

Broms (1979) indicated that the settlement of ground reinforced

with granular pile reduces to 80 percent.

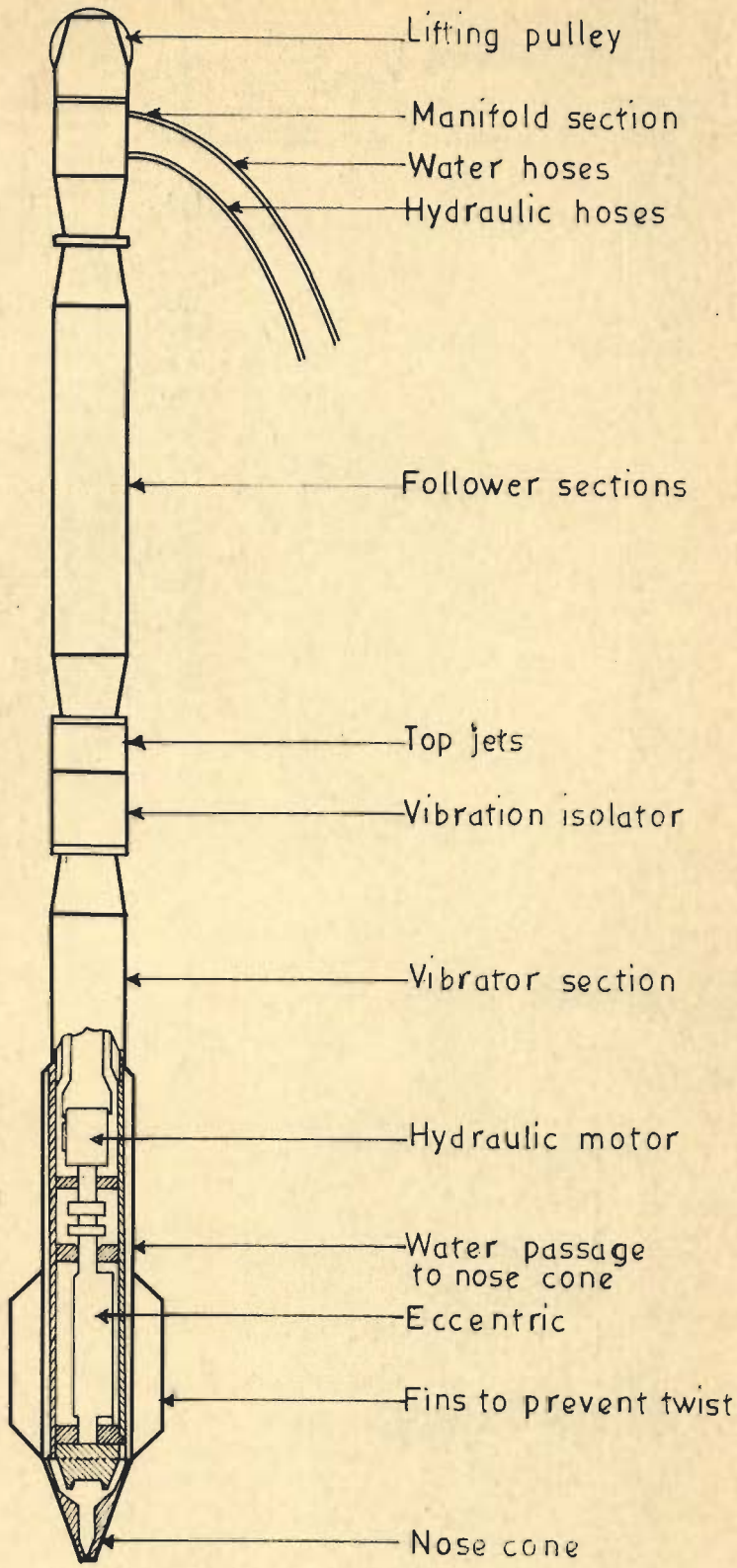
2.4 CONSTRUCTION PRACTICES

Several practices for construction of granular piles have been reported in the available literature. These have been reviewed in the following sections_ :

2.4.1 Vibro-Compaction Process

The piles can be constructed using a vibrofloat (Fig. 2.32) In the case of non cohesive sub-soil stratum the vibrofloat sinks in ghe ground under its own weight (about 3-8 tonnes depending on the total length) with the assistance of water and vibration. The length of the extension tube together with the vibrator and the lifting height of the crane is required to correspond with the total depth of penetration. After reaching the predetermined depth, the vibrator is then gradually withdrawn from the ground causing compaction. Three basic steps are involved in the construction (Fig. 2.33) (Engelhardt and Kriseh 1977). The effective-neess of the compaction is dependent on the characteristics of the vibrator in terms of energy input, amplitude, frequency and its shape. In well graded sand even the most powerful vibrators available to-date require centre spacing of 3 to 3.5m in equilateral triangular grids to produce 65 to 70 percent of relative density in the centroid between the three compaction points. But close spacings down to 1.5m can produce minimum relative density of 90 percent and more (D'Appolonia 1953). Relation between compaction spacing and relative density of sandy deposits based on direct measurement in the field is shown in Table 2.11 (D'Appolonia 1953, Basore and Boitano 1969).

During vibration the intergranular forces between the particles



FIG·2·32. ESSENTIAL FEATURES OF A VIBROFLOT

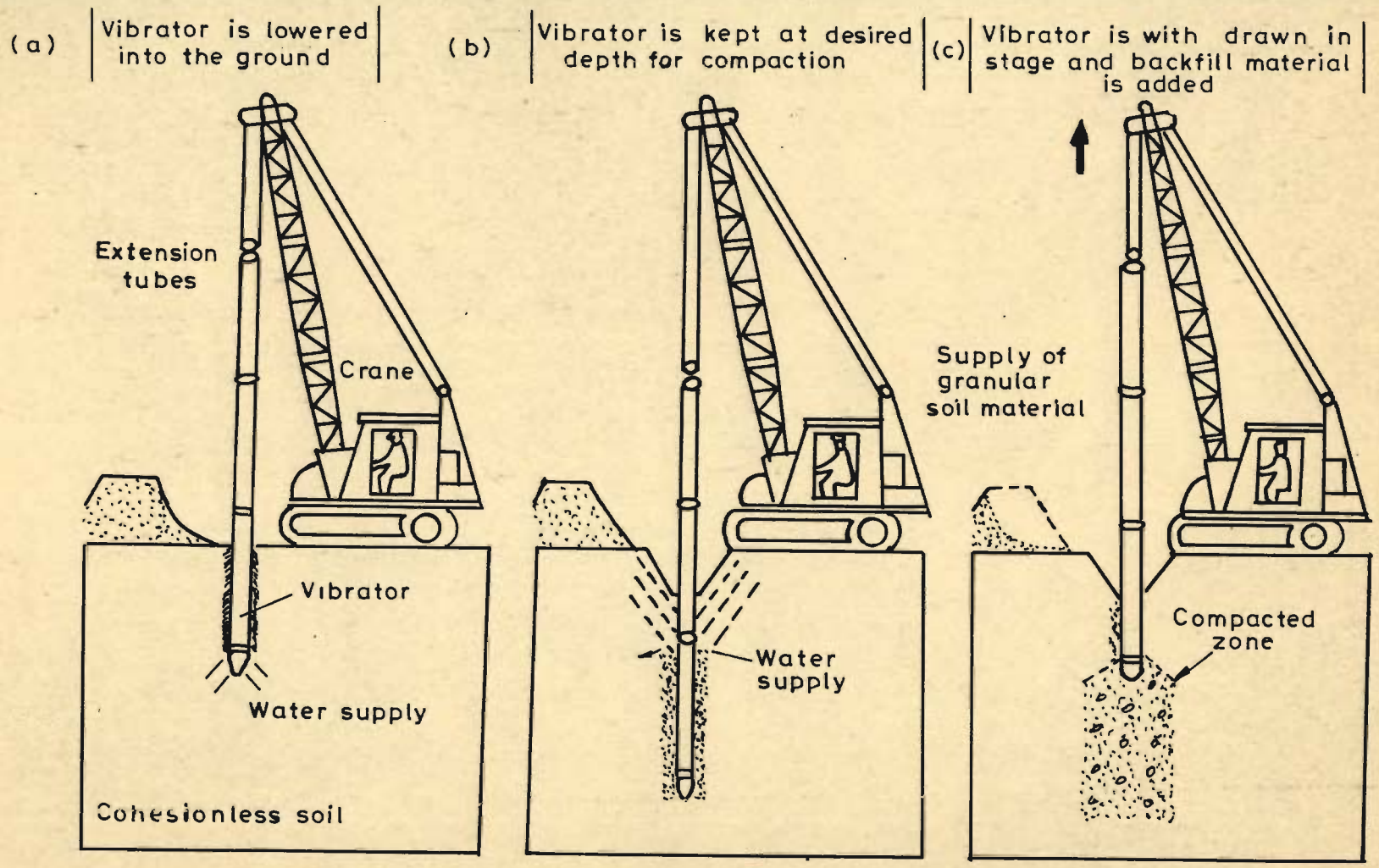


FIG.2.33. THE VIBRO COMPACTION PROCESS
(AFTER BAUMANN AND BAUER 1974)

are nullified and particles are re-arranged unconstrained, unstressed by gravitational forces to the densest possible state. The compressibility and void ratio increases significantly (Thorburn and McVicar 1968).

Table 2.11 Compaction Spacing and Relative Density Relationship

Location	Pile spacing	Relative Density (%) Untreated ground	Relative Density (%) Treated sand	Other properties
	1.8	-	85-93	Effective size 0.08mm uniformity coeff. 2 to 4 the later being more silty.
	2.4	-	75	
Dungeness Nuclear Power Station	1.8	60 to 75	91 (average)	Tests were carried out in 7m deep dewatered pit

These tests suggest that for typical unit loads 100-400 KN/m^2 compaction spacing between 1.8m to 2.5m is appropriate. All these results are applicable to American/British type of vibroflot. The system of compacting loose sand by vibroflot is already known for its application and vibro replacement method (Watt.et.al 1967).

As a standard rule sand has to be clean for successful densification by vibration (Engelhardt and Krisch 1977). More than 20 percent silt and/or clay in sand increase the binding forces between the particles to an extent where re-arrangement of individual particles to a greater stage of density is not any longer achievable by vibro-compaction. Range of soils suitable for stabilisation by vibro-compaction method is shown in Fig. 2.34.

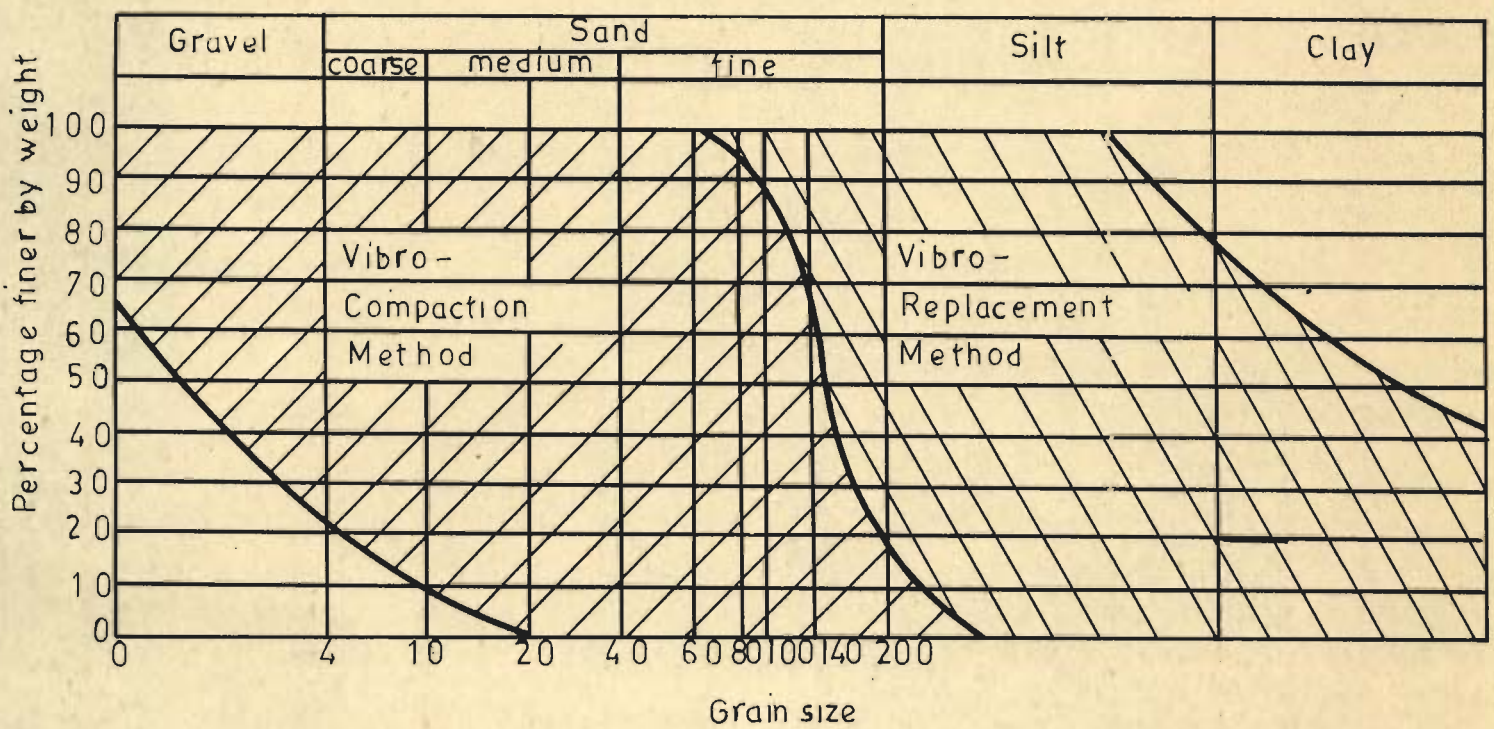
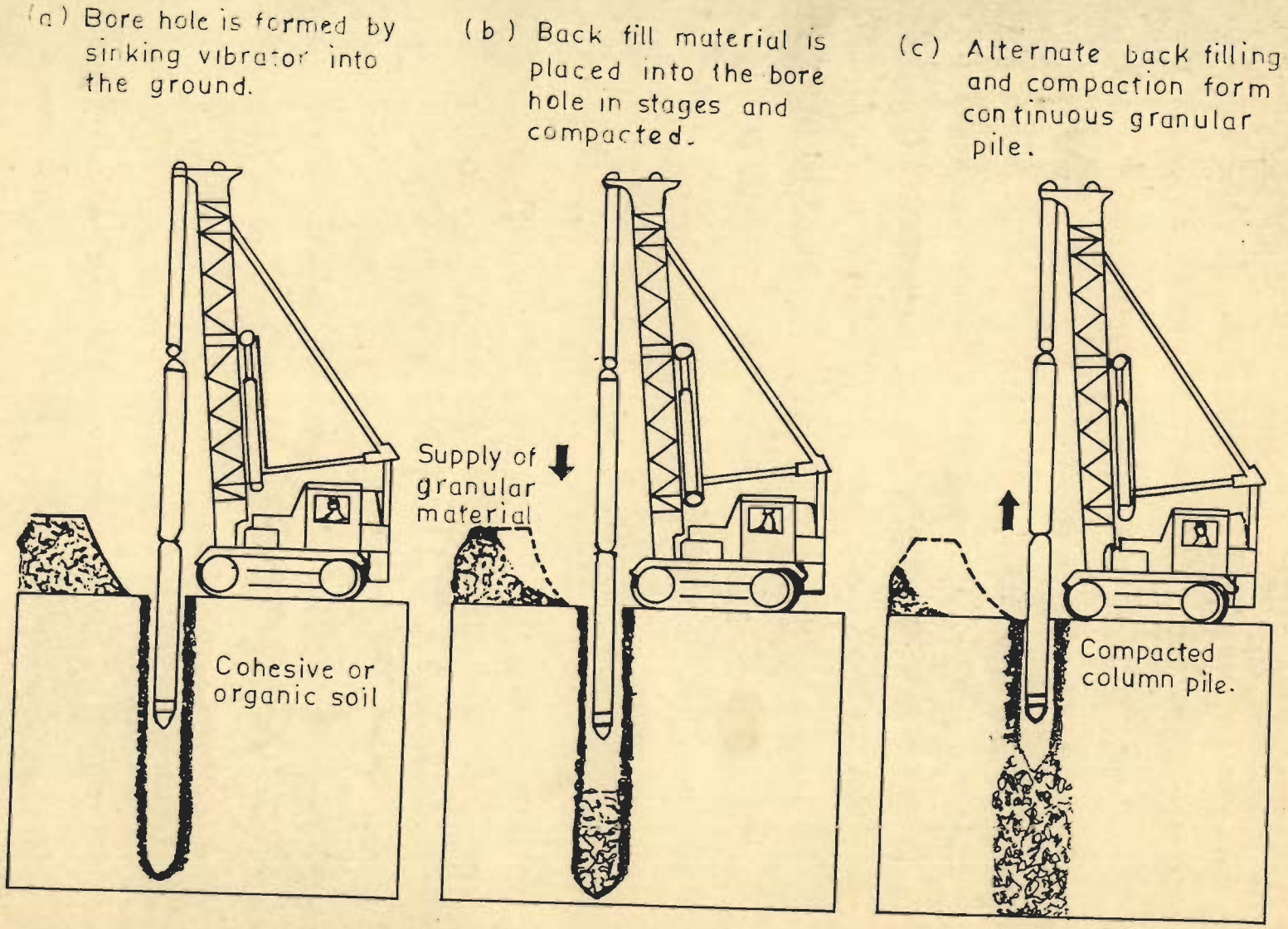


FIG. 2.34. RANGE OF SOILS SUITABLE FOR STABILIZATION.
(AFTER BAUMANN AND BAUER 1974)

2.4.2 Vibro-Replacement Process

While vibro compaction is a method to improve the density of cohesionless granular soils, vibro-replacement is used to improve cohesive soils with deep vibratory methods. The equipment used for this method is the same as that for vibro-compaction (Engelhardt and Krisch 1977, Thorburn and Mc Vicar 1968). The vibrator after being switched on Fig. 2.35 sinks rapidly under its own weight and assisted by water or air as a flushing medium into the ground untill it reaches the predetermined depth. Without the jetting fluid the soil immediately around the vibrator is disturbed or remoulded. It is, therefore, always preferable to use jetting fluid (water, or compressed air) to remove softened material (Watt et.al 1967). Generally water is used in fully saturated soil and air is used in partially saturated soil (Engelhardt and Krisch 1977). In the case of natural or artificial cohesive soils, the stratum is completely unaffected by the induced vibration because the rearrangement of the particles are prevented by cohesion between the particles (Thorburn and Mc Vicar 1968). When the vibrofloat is withdrawn it leaves a borehole of greater diameter than the vibrator. This borehole is filled partially with imported gravel usually well graded 12mm-75mm size. Also furnace slag has been used (Thorburn and McVicar 1968). The vibrator then repenetrates and displaces the back fill into the sides of the borehole in native soil and at the same time compacting underneath its tip. During the back filling the vibrator is raised and lowered by 30cm thus each batch of granular material placed in the bore hole is influenced by the weight of the machine and by a centrifugal force created by an eccentric vibration (Watt et.al 1967). Repetetion of this procedure forms a cylinderical stone column (granular pile)



FIG·2·35. THE VIBRO-REPLACEMENT PROCESS.
(AFTER BAUMANN AND BAUER 1974)

conical or straight sided dowel 0.3m-0.5m diameter with lengths 2-5m is vibrated and hole is made in soft cohesive soils (Broms 1979), remaining procedure is same as discussed above.

The diameter of the granular piles (stone column) depend on the strength of the native soil. In very soft cohesive soil the back fill material is pushed laterally and forms a granular pile and larger diameter (1.1m) where as in stiff cohesive soil slightly smaller (0.6m) pile results. Thus to a considerable extent the system is self compensating. Softer the soil larger the pile diameter (Watson and Thorburn 1966, Engelhandt and Krisch 1977, Broms 1979). Thus the improvement in the strength of the cohesive soil is dependent on the formation of very dense granular pile of coarse fill material together with the improvement in the rate of dissipation of excess pore water pressure and radial drainage conditions (Thorburn and McVicar 1968).

2.4.3 Influence Zone

The diameter of the vibrofloats are generally 30 to 50cms and its length varies between 3 to 5m. The granular piles thus constructed with either vibro-floatation process in non-cohesive soil or vibro-replacement method in clays, have diameters between 0.6m to 1.1m depending upon the compressibility of the sub-soil stratum. A large number of Dutch cone penetration tests on three sites in clayey, silty and wind blown dune sand, at various distances from the centre of the compaction points, were measured by Webb (1968)The maximum influence of radius ranged from 1.3m for clayey sand to 1.8m for dune sand. Respective cone resistances near the centre of the compaction points were of the order of 8.6MN/m^2 and 16.5MN/m^2 in comparision to 3.5MN/m^2 and 6.9MN/m^2 for untreated soil.

2.4.4 Ramming Process

Granular pile installations based on ramming of stone aggregate and sand within a cased hole has been described by Datye and Nagaraju (1975). The ramming technique was conceived to be a low cast-substitute for vibrofloatation. Datye and Nagaraju (1981) have described the method of advancing borehole placing of stone and sand and ramming techniques.

The process of ramming calls for a pile driving rig, deep hammer or disel hammer with a mandrel. The ramming is carried out in short lifts and only at limited number of stages.

2.5 Special Studies

Some important aspects which affect the stress-deformation behaviour of granular piles, such as:

(a) Granular pile diameter, spacing and depth

(b) Importance of coarse back fill

(c) Influence of granular pile installation on clay strength

(d) Sharing of the design load between pile and soil.

(e) Performance of granular piles in seismic zones have also been reported in the available literature. These aspects are discussed in the following sections.

2.5.1 Granular Pile Diameter, Spacing & Depth

Watt, Boer and Greenwood (1967) have reported that compacted granular piles are produced from 0.75m to 1.05m in diameter in cohesive soil, though these are reported to have diameters from 0.6m to 1.0m (Floss 1979).

These are used in triangular pattern under the loaded area (strip or raft) plus 1.8m to 3m wide surrounding the strip. With a view to provide overlapping zones and to cover a wide area, granular pile spacing is determined by settlement tolerances for load to be applied

and depend upon the characteristic of the soil (Watt et.al 1967 Greenwood 1970). However, granular piles of 0.6m to 1.2m diameter and 2.1m to 2.7m spacing have also been recommended by Thorburn (1966) in a triangular grid. Engelhardt and Krisch (1977) recommend diameter between 0.6m to 1.1m and spacing between 1m to 3m, depending upon degree of improvement required. In the case of triangular pattern large area is covered with minimum number of piles. The diameter and spacing of the granular piles depend upon the nature and consistancy of the soil deposit (Floss 1979). It has been recognised in practice with due regard to structural considerations that closer spacing is used under isolated footing than beneath large rafts (Greenwood 1970). The back fill material should consist of coarse gravel, crushed, stone or slag with 20mm-70mm in size (Floss 1979). Well graded coarse gravel 12mm-75mm size have been recommended by (Engelhardt & Krisch 1977).

The depth of the granular piles may extend to reach compact ground at shallow depths but is otherwise normally determined by the depth of the pressure bulb predicted by elastic theory which is usually two times the plan dimension of the footing (Greenwood 1970). However, the rafts or closely spaced footings stress deeper zones, it is found in practice that the depth of granular piles between 6m to 8m below footing level is adequate. This is found sufficient to develop arching in the treated zone to bridge loose pockets in untreated zone below.

2.5.2 Importance of Coarse Back Fill

A comparision of granular pile and sand pile behaviour is studied on a large scale field test (Greenwood 1970). The sub-soil stratum consisted of (3.3±0.3)m of peat and clay overlaid a deep deposit of fine uniform sand. Granular piles and piles 6m long were constructed

by using identical technique, using water circulation for the construction of all piles. The piles were placed in a triangular pattern having a base of 2.3m and height of 2m, Fig. 2.36. A sand blanket of 30cm thickness was placed over the highly compressible peat surface to provide working surface, Fig. 2.36b and also to ensure free draining surface. The average pile diameter as inferred from the back fill consumption was 1.2m. One group of piles sand back fill were used (0.3mm size) and remainder was formed with gravel 30-70mm. The sub-soil properties were,

$c' = 5\text{-}10\ KN/m^2$ m.c. = 400 to 440 %

$\emptyset' = 13^0\text{-}20^0$ $m_v = 0.004\ m^2/KN$

Large number of settlement plate were established. Earth fill (surcharge) was placed over it with a height of 2.5m.

After 15 months, the average settlement from measurements are shown in Fig. 2.36c. It shows that settlement in granular piles = 468±31mm and sand piles = 645±75mm whereas in untreated soil = 765±13mm. The calculated settlement for untreated soil was 665mm. This showed that with relatively wide spacing of 2.3m, the granular piles reduced the settlement to almost half whereas the reduction with sand pile was only 15 percent relative to untreated soil.

2.5.3 Effect of Granular Pile Installation on Clay Strength

Sheng Chongwen (1979) presented the observations on the influence of pile installation on the shear strength of clay due to disturbance. The measured strength by vane shear tests in-situ at two sites in China before and after the pile installation in soft clay is shown in Table 2.12.

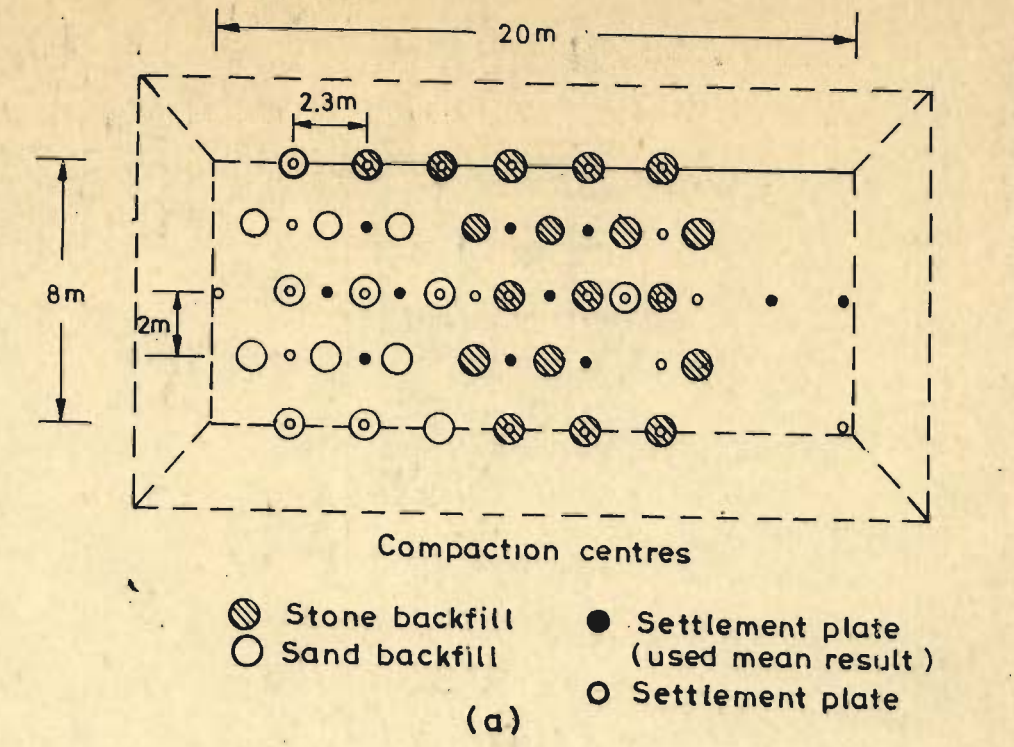


FIG.2.36. BREMERHAVEN TEST
(AFTER GREENWOOD 1970)

Table 2.12 Shear Strength Before & After Pile Installation

Projects	Soil type	In-situ shear strength t/m^2	Strength after installation of granular piles (t/m^2) after		
			15 days	21 days	52 days
Oil tank G-233 (Zhejiang)	Muddy clay	1.86	1.66	2.10	1.93
		2.55	3.21	3.18	3.30
Shipyard Workshop (Nanjing)	Muddy sand clay	2.38	2.51	-	-

The test results demonstrated that the soft cohesive soils can recover any reduction in strength in a short period of time.

2.5.4 Sharing of Applied Load Between Granular Pile & Surrounding soil

It has been indicated that the cohesive sub-soil stratum when reinforced with granular piles (Thorburn 1975), the resulting soil-pile system is transformed into a composit system consisting of granular piles and the clay in between them surrounding these piles. Thus the resulting reaction of such a reinforced ground to the applied load is quite different from that of un-reinforced cohesive soil compacted by vibro-compaction techniques. When the load of the structure is transmitted by the footing or raft foundation to the surface of the cohesive soil duly reinforced with granular piles, a large portion of the total applied load is initially resisted by the very dense granular piles which are rigid in comparision to soft cohesive soil surrounding the piles. The remaining portion of the load is carried by the clay in between the piles and in contact with the footing or raft.

The initial load on the pile top produces radial strain in the soft cohesive soil surrounding the roughly cylinderical walls of the granular

pile, thereby mobilising the passive strength of the clay deposit. The magnitude of the radial strain necessary to develop significant radial stress are small. This is due to the fact that considerable radial preloading have occurred during the construction of the granular pile. The vertical strain in the granular pile due to the application of design load causes transference of load from the yielding granular pile top to the soft clay deposit under the footing or the raft. This further causes vertical strain and so on untill the equilibrium condition is reached. The load distribution between granular pile and the unstabilised soil depends on the relative stiffness, Broms (1979).

Engelhardt and Golding (1975) have indicated that the sharing of the load between granular pile and the surrounding soil is simply a function of area and modulus of compressibilities of the respective materials at a stage when the settlements in pile and the soil are equal in the case of a rigid foundation and the equilibrium has reached. At a site consisting of esturine deposits (CL-ML) upto 10.7m depth reinforced by granular piles in a triangular grid 1.22mx1.98m approximately 83 percent of the total load was carried by the granular piles and 17 percent by the native soil.

Baumann & Bauer (1974) suggested a method of estimating the load shared between pile , q_p and the soil, q_s. If q_o is the average load transmitted to the soil-pile system by the rigid footing then

$$q_o A = q_p A_c + q_s \; As \qquad \ldots 2.44(a)$$

where

A is the area of the footing per compaction pile,

A_c & A_s are the area of the pile and soil respectively under the footing.

For a rigid footing the settlement of the granular pile S_c and that of the soil S_s in contact with the footing will be equal

$$S_c = S_s \qquad ...2.44(b)$$

The load transmitted by the footing will cause an increase in the restraining lateral pressure in the soil $\Delta\sigma_s$ given by equation

$$\Delta\sigma_s = K_s\, q_s \qquad ...2.45$$

The value of K_s coefficient of earth pressure lies between at rest pressure and passive pressure coefficient. The lateral pressure increase in the granular pile $\Delta\sigma_c$ is given by

$$\Delta\sigma_c = K_p\, q_c \qquad ...2.46$$

K_p assumes a value between K_o and K_a earth pressure coefficient. The lateral bulging of the granular pile is due to the pressure difference given by equation 2.45 and 2.46.

$$\Delta\sigma = \Delta\sigma_p - \Delta\sigma_s \qquad ...2.47$$

The radial deformation of the granular pile r can be found from equation

$$\Delta r = \frac{\Delta\sigma}{E_p}\, r_o\, L_n \left(\frac{a}{r_o}\right) \qquad ...2.48$$

where

r_o is the radius of the pile, the equivalent area $a = (A/\pi)^{0.5}$,

A is the footing area attributed to per pile point.

Hence the settlement S_p of the granular pile is given by:

$$S_p = 2\left(\frac{\Delta\sigma}{E_p}\right) H.L_n \left(\frac{a}{r_o}\right) \qquad ...2.49$$

Here H is the depth of the strata reinforced with granular pile.

The settlement of the untreated soil is:

$$S_p = \frac{H}{E_s}\, q_s \qquad ...2.50$$

Using equation 2.44, 2.48 and 2.49, the following relationship, can be established:

$$\frac{2\Delta\sigma}{E_p} H. \text{ Ln } \frac{a}{r_o} = \frac{H}{E_s} q_s \qquad ...2.51$$

and utilising equation 2.47, the above equation can be re-arranged

$$\frac{2K_p}{E_p} q_p H. \text{ Ln } \frac{a}{r_o} - \frac{2K_s q_s}{E_c} H. \text{Ln } \frac{a}{r_o} = \frac{H}{E_s} q_s \qquad ...2.52$$

Thus the ratio of q_p/q_s is given by

$$\frac{q_p}{q_s} = \frac{(1+2 \frac{E_s}{E_p} . K_s \text{ Ln } \frac{a}{r_o}}{2(2 \frac{E_s}{E_p} K_p . \text{Ln } \frac{a}{r_o})} \qquad ...2.53$$

Thus if the pile spacing and the diameter of the pile is known, the sharing of load between granular pile and the soil can be found out.

The modulii of deformation E_p and E_s can be found from field or laboratory. Various values of soil-pile properties may be taken from Table 2.13 as suggested by Baumann and Bauer. The value of q_p/q_o may be found from from eqn. 2.44(a).

Table 2.13 - Soil-pile Properties

Soil Type	E_s/E_p	Pile Radius r_o(m)	K_s	K_p
Loose sand	1/12	0.70	1.0	0.4
Medium dense sand	1/4	0.75	0.8	0.4
Dense Sand	½	0.80	1.2	0.4
Clayey Silt	1/16	0.40	1.5	0.6
Sandy Silt	1/8	0.40	1.5	0.6
Silty soft clay	1/25	0.45	1.25	0.75

2.5.5 Performance of Granular Piles in Seismic Zones

Greenwood (1970) indicated that in situations where treatment of ground is essential to prevent liquefaction or settlement of loose sand subjected to earthquakes or artificial vibrations greater depth of granular piles may be necessary. This ensures dynamic stress level induced due to vibration of the vibrofloat during treatment every where exceeds those anticipated under working load.

Besides densification of sandy soils between granular piles (vibro-replacement points), the use of gravel as back fill material results in installation of numerous vertical drains which allows pore water pressure generated by cyclic loading to dissipate as fast as they are generated (Seed and Brooker 1976). The use of 12mm - 75mm size gravel as a back fill material in sandy soils susceptible to liquefaction provide a new technique for earthquake engineering (Engelhardt and Krisch 1977). Mori (1979) demonstrated that the embankment for Tohoku express way Japan on Aneba section was founded on a 6m deep peat deposit underlain by 6m thick soft clay reinforced with granular piles 60cm dia., more than 12m length at a spacing of 1.2m in April 1977. This embankment suffered Miyagoki earthquake of magnitude 7.5 in 1978 without any damage.

Engelhardt and Golding (1975) have described the field testing details to evaluate the performance of granular piles in seismic areas. For the construction of the new 16 mgd sewage treatment plant on soft to medium stiff fine grained esturine deposits, soil improvement with granular piles was considered as one of the foundation. At the construction site, the esturine deposit extended to a depth of 10.7m below grade. At a location on the site 18 granular test piles were installed in a 1.22mx1.98m triangular grid pattern. Each granular

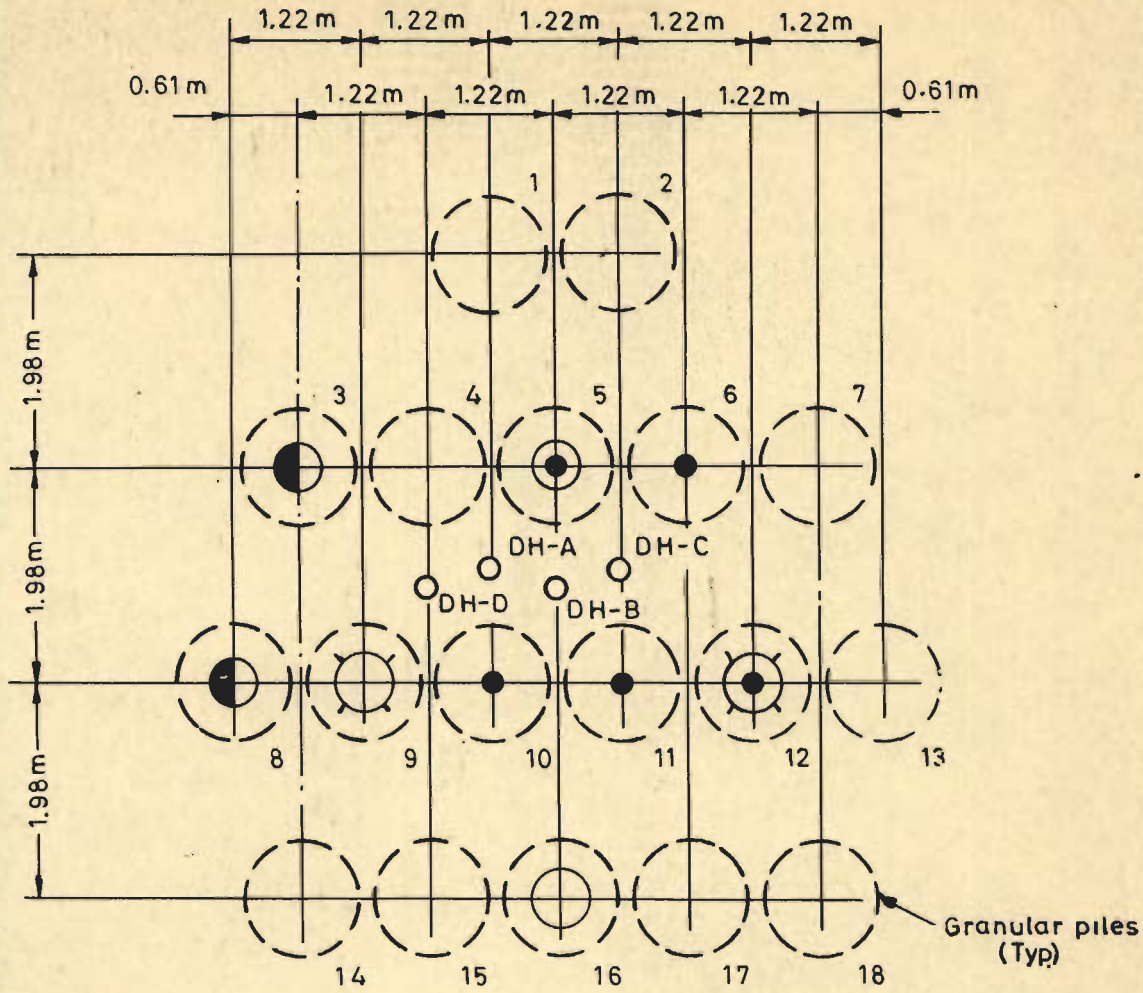
pile extended through the esturine deposit and terminated in the underlying marine sands. The granular piles were constructed by wet method with 19mm to 76mm natural broken stones. Based on the consumption, the average diameter of the granular pile was 1.1m. The test programme consisted of standard penetration tests, boring before and after the installation of granular piles, two horizontal shear tests on the granular pile alone and two similar tests on the combined system of granular pile and contributory natural surrounding soil, and two vertical load tests. The site plan and location of test points is shown in Fig. 2.37. The SPT values indicated a marked increase indicating that the specific sand lenses were densified significantly and the corresponding relative density was 92 percent which in the range where liquefaction is very unlikely.

2.6 SKIRTED GRANULAR PILES

The granular piles, unlike r.c.c. piles when installed in soft cohesive sub soil stratum and subjected to sustained vertical loading, fail in bulging (refer art. 2.2.2). To reduce the bulging effect and further to reduce the settlement, these piles have been installed where upper two to four pile diameters have been grouted (Engethardt and Krisch 1977). It has also been indicated that the granular piles may be replaced by concrete piles in the upper portion or an additional injection of cement mortar is also possible (Floss 1979) with a view to increase the load bearing capacity of granular piles and to improve their deformation behaviour. These preceding suggestions lead towards increasing the granular pile stiffness and checking the bulging of the granular pile along the critical pile length.

2.6.1 Effect of Surcharge (Field Study)

The load-deformation behaviour of granular pile could also be



KEY

- Vertical load test
- Lateral load test (1.79 m Dia.ring)
- Lateral load test (1.07 m Dia. ring)
- SPT Boring before granular piles installation (DH.5 at DH.6 at 6etc.) (DH 5 A 5 DH 6 At 6 Etc.)
- SPT Boring after granular piles installation

FIG. 2.37 .TEST FIELD PLAN (AFTER ENGETHARDT & GOLDING 1977)

achieved by placing surcharge load around the single pile or the pile group. This will result in increasing the confining pressure over the existing in situ lateral ground stress. A similar method has shown great promise in checking incipient edge failure by rotational slip under a tank perimeter (36.6m in dia. 14.6m in height) founded on 13.7m thick soft clay deposit overlaid on glacial fill, having undrained shear strength of 0.24 kg/cm^2 to 0.48 kg/cm^2. Fig. 2.38 (Liu and Dugan 1975). The efficacy in counterweighing in resisting the settlement due to lateral flow of the soft cohesive sub-soil is reflected in settlement pattern obtained during tank loading Fig. 2.39 (Penman 1977).

2.6.2 Effect of Surcharge (Model Study)

The effect of surcharge load on the soil surrounding the granular piles in model tests in the laboratory has also been studied. The surcharge load was applied through a spring of known stiffness Fig. 2.40. The surcharge loads, q were increased from no surcharge condition to 1.1 kg/cm^2. The test results indicated a linear increase in the ultimate load carrying capacity of granular piles, q_{ult} with surcharge, Fig. 2.41 shows a typical load settlement curve of a granular pile with and without surcharge. Based on test results Mokashi et.al (1976) suggested equation

$$(q_{ult}) = q_{ult} + 6.2q \qquad ...2.55$$

and for precompressed piles

$$(q_{ult})s = q_{ult} + 6.74q \qquad ...2.56$$

where

q_{ult} is the initial pile capacity.

It has also been indicated that the surcharge stress q further

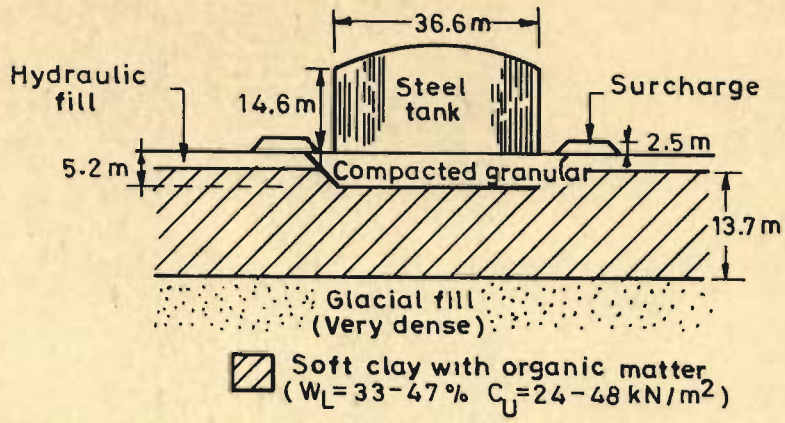


FIG.2.38. TANK FOUNDATION WITH SURCHARGE
(AFTER LIU AND DUGAN 1975)

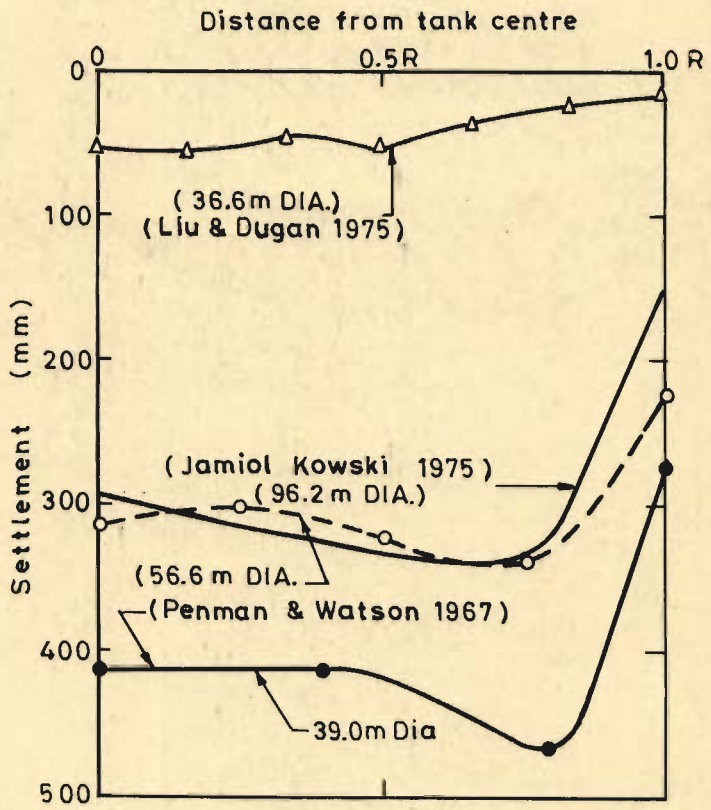
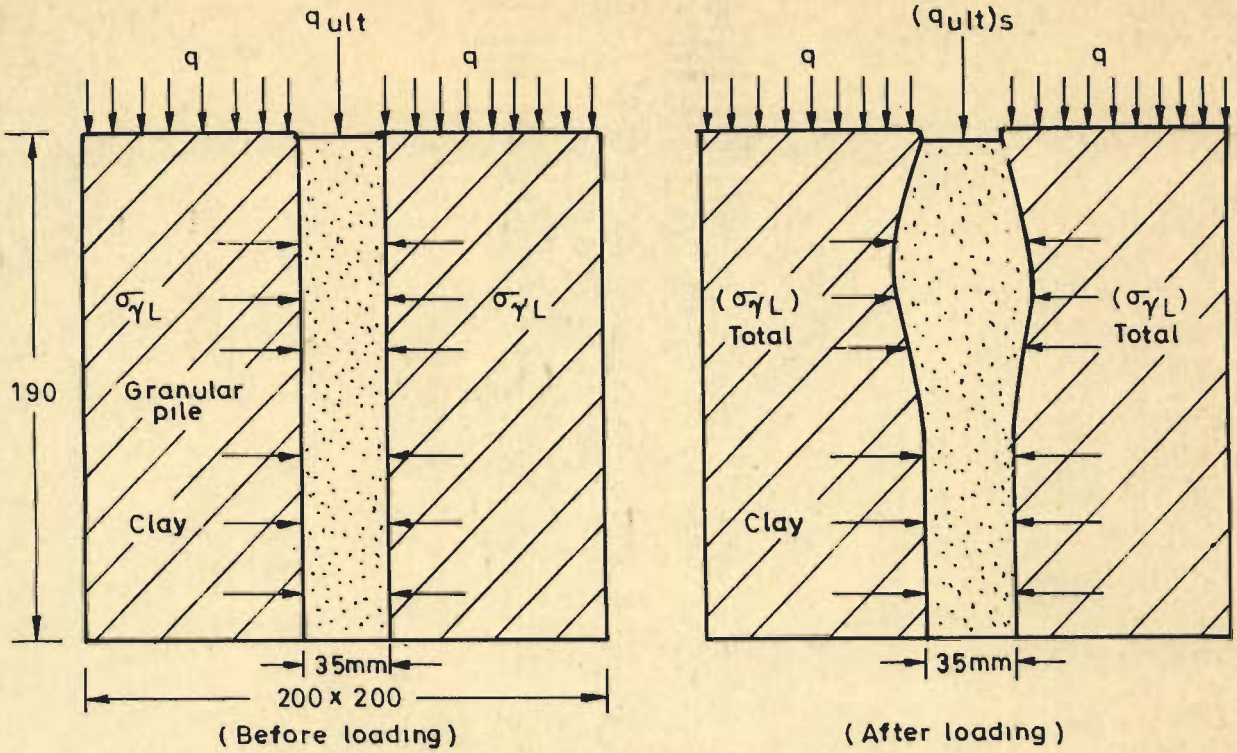


FIG.2.39. SETTLEMENT PROFILES OF FOUR LARGE TANK (PENMAN 1977)

q_{ult} = Vertical stress

$\sigma_{\gamma L}$ = Lateral resistance

q = Surcharge (0–1.1 kg/cm^2)



(ALL DIMENSIONS IN mm)

FIG.2.40. MODEL LOADING TESTS ON SINGLE GRANULAR PILE
(MOKASHI et. al. 1976)

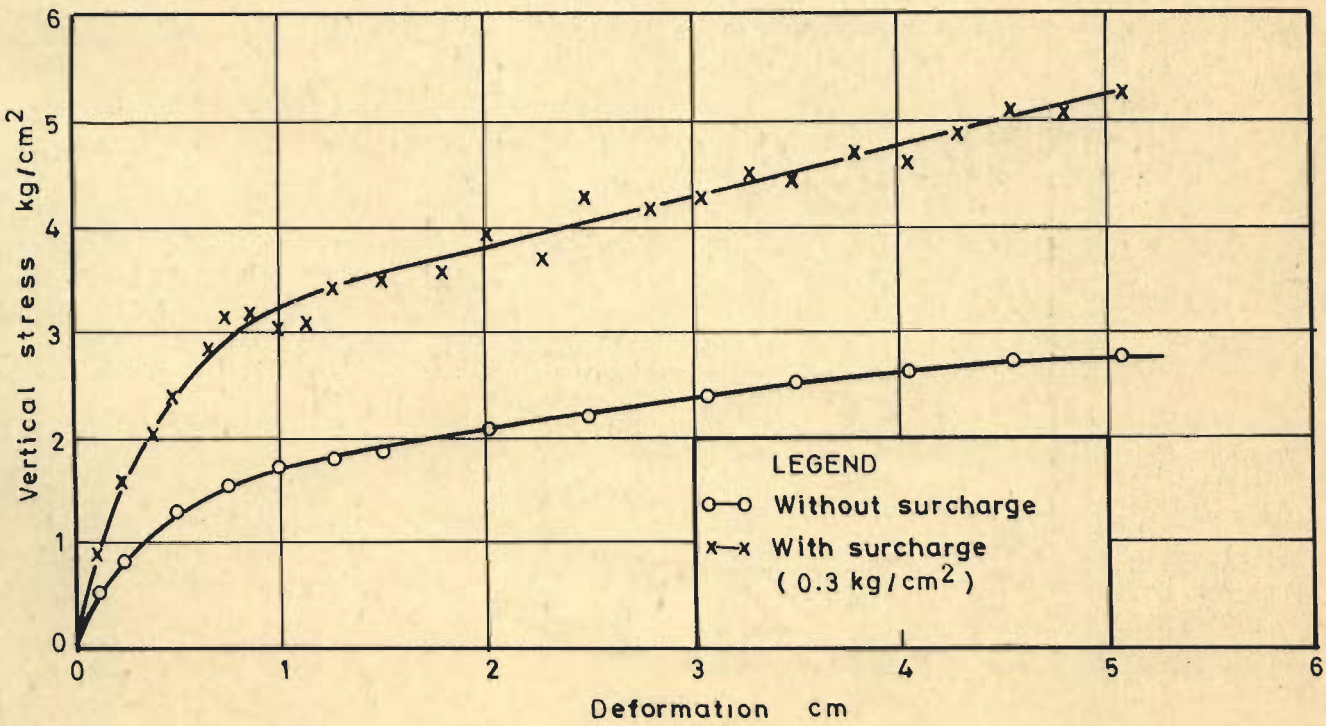


FIG.2.41. EFFECT OF SURCHARGE ON STRESS DEFORMATION RELATIONSHIP
(AFTER MOKASHI et. al 1976)

involves a lateral stress (K_1q) in the clay surrounding the granular pile where K_1 is less than or equal to unity.

Thus the radial supporting stress (limit pressure) σ_{rL} which is net lateral pressure due to bulging of the pile will also increase from (kC_u) to ($KC_u + K_2q$) where

$$\sigma_{rL} = C_u \left(1 + Ln \frac{E_s}{2C_u (1+\mu)}\right) \qquad ...2.57$$

$$= K C_u \qquad ...2.58$$

Thus with surcharge load $\sigma_{rL} = (KC_u + K_2q)$ ...2.59

and the total confining pressure

$$(\sigma_{rL}) \text{ total} = K_1q + (KC_u + K_2q) \qquad ...2.60$$

$$(\sigma_{rL}) \text{ total} = KC_u + q(K_1+K_2) \qquad ...2.61$$

Though the test results indicated the value of (K_1+K_2) as 1.3, however for the design purposes Mokashi et.al suggest its value as unity. Hence (σ_{rL}) total $= KC_u + q$ ...2.62

Therefore the ultimate bearing capacity with surcharge is given by

$$(q_{ult})s = K_p.(K.C_u+q) \qquad ...2.63$$

where $K_p = \tan^2 (45+\emptyset'/2)$ ...2.64

and $K = 1+L_n \dfrac{E_s}{2C_u(1+\mu)}$ ...2.65

Equation 2.63 suggest that the ultimate bearing capacity of a granular pile increases directly by an amount equal to coefficient of passive pressure of the granular pile (K_p) times the surcharge intensity q.

These observations suggest that the load bearing capacity of granular pile may be increased by providing a surcharge load around the pile, with due attention that the surcharge load be limited to the safe bearing capacity of the virgin soil surrounding the granular piles. However, in actual practice, placing of surcharge in any form may not always be practical as well as feasible due to many considerations like functional utility of the structure. This advantage may be successfully achieved by providing a rigid skirt in the form of mild steel pipes or r.c.c. pipes around the pile. The depth of the individual skirt is to be limited to the critical pile length

2.6.3 Concept of Skirted Granular Pile (A new approach)

In actual practice, granular piles are always installed in groups, may be small or large, depending upon the design load and size of the structure. In the case of a group of granular piles, the sustained vertical design load is shared between the granular piles, and the virgin soil in between the granular piles. This part of the design load transmitted to the surrounding soil will further increase the confining stress of an individual pile inside the group besides restraint provided by the passive pressure provided by the virgin weak soil and surrounding piles. Thus increase in confining stress is likely to help in reducing the vertical displacement of the intermediate granular piles and thus subsequently checking the failure by bulging resulting into higher pile capacity. However the peripheral granular piles in the group is not likely to have any confinement as in the case of intermediate piles. Thus these peripheral piles will experience lateral restraint available due to available passive pressure of the virgin soil. In view of these facts, and with a view to achieve the same advantage of surcharge as discussed earlier (refer art. 2.6.1 & 2.6.2), a collective rigid skirting around the pile group may be an appropriate and economical alternative to surcharge load.

2.7 Case Records

Several cases where granular piles have been used successfully in soft clay, silt and sandy clays have been described by several researchers/practicing engineers during last one and a half decades all over the world to support isolated or strip footings, rafts, embankments and heavy storage tank foundations besides high rise building foundations. Prominent among these are case histories by Grime and Kantley (1965), Watson and Thorburn (1966), Watt et al (1967), Thorburn and McVicar (1968), Greenwood (1970), Jordin (1974) Baumann and Bauer (1974), Rathegeb and Kutzner (1975), West (1975), Mc Kenna et.al (1975), Engethardt and Golding (1975), Hughes et.al (1975), Thorburn (1975) Datye and Nagaraju (1975, 1977 & 1981). Engelhandt and Krisch (1977), Vaturin (1977) Mori (1979) and Sheng Chongwen (1979).

2.7.1 Granular Piles for 20 Storeyed Building

Grime and Kantley (1975) describe a case where 13m loose sand and clay deposit is reinforced with 7.5m long granular piles to support a design load of 193 KN/m^2 (1.93 kg/cm^2) transmitted through a 13m wide raft. The raft was used to support a 20 storey building. The post treatment testing had indicated satisfactory results. These were proved during working life of the structure. The calculated settlement was 95 to 210mm against the observed value of 20-25mm.

2.7.2 Blast Furnace Slag Piles For the Warehouse Floor

Watson and Thorburn (1966) described a case where fly ash fill laid over peat and soft clay deposit was used to construct a concrete floor slab transferring a design load of 0.5 kg/cm^2. The settlement records of the existing floor indicated that out of 750mm of total

calculated settlement 350mm had already occurred in 13 months. Therefore, for a similar floor on the same site the sub-soil stratum was reinforced with blast furnace slag 100mm graded size with 2.25 to 2.7m spacing and 3m to 6.6m depth. The reinforcement process was followed by an in-situ load test on a 750cmx750cmx15cm cast in-situ concrete slab. The test results demonstrated an improvement of 62 percent in settlement over the virgin filled up soil for an intensity of design load equal to double the design load (1 kg/cm^2) and the improvement was found 90 percent against the design intensity (0.5 kg/cm^2).

2.7.3 Blast Furnace Slag Piles for the Steel Tank Foundation

A similar case is described by Watt Boer and Greenwood (1967) where blast furnace slag piles 75 to 105cm in diameters, depth 6 to 7.2m were constructed for six nos. steel tank foundation having diameters 17m to 67m at Tees port site, UK. The blast furnace slag was 25 to 50mm in size, piles were constructed with vibroflot replacement method. The sub-soil consisted of hydraulic filling with cohesive soils. At another similar site Heldon (UK), the seventh tank was constructed whose diameter was 43.89m. The height of the various tanks varied from 12.8m to 16.45m. The water test results demonstrated that after the ground is reinforced with blast furnace slag granular piles, a load exceeding the theoretical ultimate bearing capacity of the virgin soil can be applied, besides substantial decrease in the overall compressibility of the reinforced soil can be fully demonstrated.

The observed settlement at Teesport and Heldon sites were less than one third of the predicted settlement. The differential settlement along perimeter varied from 1/4 to 1/7 of the maximum settlement and the tilt was less than 1/3 of the maximum settlement measured along any diameter. The most striking feature is the granular

piles is the uniformity they impart to the treated ground.

2.7.4 Blast Furnace Slag Piles for Multi-Storeyed Buildings

Thorburn and McVicar (1968) have described the use of blast furnace slag piles under a long block of three storey building on a very sandy clay stratum varying in thickness from 1.33m to 1.82m, overlaid on a firm glacial fill. The bearing capacity of the sub-soil was 1.07 to 1.37 kg/cm^2 for the sandy clay stratum. The allowable design load for the slag piles constructed at a spacing of 0.91m to 1.22m with depths varying from 4.27m to 4.57m. The setltment records indicated that the total observed settlement did not exceed the allowable settlement of 25.4mm and the differential settlement did not exceed 9.5mm with a angular distortion of 1 in 640. The building is reported to have performed satisfactorily.

In another case where six, five storey blocks were constructed on slag piles with a spacing of 0.68m to 1.8m having varying depths 3.66 m to 4.5 m under the load bearing walls. The allowable loads on each pile was not less than 15 tonnes. The sub-soil stratum consisted of 2.7m made up ground 4.2m thick very silty clay overlaid on a 5m thick soft brown silty clay. The bed rock was encountered at a depth of 23.8m to 32.3m below existing ground level. The water table was at a depth of 1.5m to 3m. The settlement records of 5 blocks (except block No. 2) indicated a total settlement of 44.5mm and differential as 19.1mm with an angular distortion of 1 in 350. Though in block No. 2, a total maximum and differential settlement of 143mm and 28.6mm with an angular distortion of 1 in 350 was observed. The investigation revealed the presence of silty clay deposit containing organic matter. These blocks withstood the stress and strains induced by the consolidation settlement satisfactorily without any damage.

2.7.5 Filled up Soils

Greenwood (1970) has discussed the application of granular piles in filled up soils consisting of brick bats, rouble debris where clay overburden from fresh excavations were dumped to make up the ground.

2.7.6 Eight Storey Residences on Silty and Sand Silty Clays

Baumann & Bauer (1974) have described the application of 8-storey student residences in silty sand and silty clay deposits in Germany. The diameter of the granular piles were 1m and the depth was 5.5m. The soil was improved 3m beyond the edge of the building to ensure against bearing capacity failure. The design load was 1 kg/cm^2 corresponding to a load bearing capacity of 17 tonnes for each pile which was well within the ultimate load of such a pile. The computed settlements were 16mm as immediate and 5mm against the observed settlement of 3cms (total) showing inadequacy of the method of settlement computation based on Baumann and Bauer (1974) method.

2.7.7 Granular Piles for Road Embankment

Jordin (1974) has described a case in France where granular piles were used to increase the bearing capacity of a road embankment. The piles were installed in soft organic clay with 1.6m spacing. Granular piles for 14 storey office building and two liquid storage tanks. West (1975) has discussed four cases of granular pile application for 14-storey office block to permit a bearing capacity of 4.3 kg/cm^2 in sand. Besides, sewage purification plant, two liquid storage tanks, 55m diameter 18m in height. The granular piles were 7m deep cast out of vibroflot replacement method in a

1.8m x 2.1m triangular pattern for the storage tanks. The maximum differential settlement across the diameter was 1 in 800, with small differential movement around the perimeter against the specified differential settlement of 1 in 300. The tank was tested for a water load of 1.75 kg/cm^2.

2.7.8 Ineffectiveness of Granular Piles in Clay Stratum

A case where granular piles did not prove effective has been discussed by Mc Kenna (1975) The granular piles were installed in a silty clay stratum with a shear strength of 10-30kPa having 0.9m dia. and 11.3m length. The spacing of the piles were 2.4m. It was postulated that the effectiveness of the granular piles was reduced by clay slurry during the installation. It was also possible that the soil close to adjacent piles have been disturbed during installation.

2.7.9 Granular Piles for Bridge Foundation

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Engelehardt and Krisch (1977) have described a case where granular piles for foundation of three bridges resulting into technically and economically satisfactory foundation solution. The granular piles 6 to 8m long, with a spacing of 1.8 to 2.2m was found to be an economical alternative to bored piles (90-210cm dia.). The sub-soil stratum consisted of loose fine uniform gravel with lenses and layers of organic matters (N_{SPT} = 3), having modulus of compressibility equal to 20-80 kp/cm^2 (kg/cm^2). The anticipated setltement was 50-80mm. The observed settlement during construction of the bridge piers and abutments were between 10-20mm with negligible differential settlements. A total of 8000m of granular piles with 90cm diameter was required for the three bridges.

2.7.10 Granular Piles as a Replacement of Sand Drains for Steel Tank Foundation

To stabilise the weak esturine deposit in the south of Hangzhon

estury in China, for a steel tank foundation granular piles were used as an alternative to sand drain. Past experience demonstrated (Sheng Choweng 1979) that the preloading time required for a 10,000 cum. tank was between 84-166 days which proved to be a serious disadvantage with the sand drain technique. To reduce the preloading time, possibility of using granular piles were investigated by Shen Choweng and his team in 1978. For this purpose, a small tank of 700 cum. capacity (9.85m dia. x 10.67m height) was chosen. The sub-soil consisted of 4m thick esturine clay (C_u = 0.18 kg/cm^2). Overlying the clay a 1m thick hard clay crust was found. The clay layer was underlain by a 34m thick muddly sandy clay (C_u=0.255 kg/cm^2). Beneath this layer was found another muddy clay layer of mean thickness equal to 9.3m with few thin lenses of silty fine sand. The base layer was consisting of medium fine sand having a thickness of more than 5m. Using 20-40mm size backfill material, 80cm diameter granular piles, 87 in number, with a spacing of 1 and 2m was constructed using ZCQ-30 vibroflot, with a quantity of back fill as 960 kg/m length of pile. The water test was carried out with full design load of 13t/m^2 within 54.5 hours. The corresponding immediate settlement was found to be 11.7cm and 28.1cm after 28 days. After further 10 days, the settlement almost completed.

The maximum differential settlement was 10.3cm with a angular distortion of 1.05 percent. The ultimate total settlement was 31cms. However, for the sand drain the maximum estimated settlement was 46.5cm. This demonstrated that the reduction of settlement for ground reinforced with granular piles is 67 percent over the ground treated with sand drains.

2.7.11 Granular Piles for Foundations in India

Granular piles for three number, 25m dia. phosphoric acid tank

foundation was constructed by Cementation Co. (Information Sheet Ind-4) as a part of Zuari Agro Fertilizer Complex in Goa, India. The tank farm is located on reclaimed land in the Vaseo bay and the soil condition comprised of 4.6m thick layer of silty sand underlain by a 4m thick layer of marine clay. The soft marine clay in turn rested on a dense sand stratum. Very soft to soft consistency of marine clay (C_u=0.04 kg/cm^2 to 0.26 kg/cm^2) and its variable thickness posed a serious foundation problem for the large tanks from both shear and settlement considerations. For such a situation, granular piles by vibro-replacement method was considered an acceptable solution to the problem. The piles were constructed at 1.75m spacing in a triangular grid to provide an overall bearing capacity of 2 kg/cm^2. Thus 253 piles per tank making a total of 769 piles were used. The granular piles were extended 0.3m beyond the compressible layer. The depth of piles varied from 10-12m. Granular piles have also been used by Cementation Co. at Madras and Farakka.

Datye & Nagaraju (1977) have described the performance of granular piles for the foundation of structures - supporing equipment, bulk storage structures warehouse and phosphric acid storage tanks for Kandla Fertilizer project complex in India designed to tolerate anticipated settlements.

The sub soil conditions comprised of highly compressible clays upto large depth including silt of low shear strength. The firm strata supporting piles was available at 20m depth.

The bulk storage building was founded on sand drain and granular piles was preloaded for the full design load of 13.2t/m^2 while that of phosphoric acid tank foundation consisting of sand drain and granular pile were preloaded to half the design load (10t/m^2) owing to the large height of the preload and anticipated shear

failure of the virgin ground. In the case of granular piles for pipe racks and conveyer support preloading was not carried out. All the structures are reported to perform satisfactorily. The depth of the granular piles are reported to be 17m and the preloading was used in order to reduce the settlement.

2.8 Summary & Comments

2.8.1 General

Though the technique of reinforcing weak sub-soil stratum by granular piles was known to the French Engineers in 1830's however, the process drew attention of research workers only during the last decade and a half. A review of available literature reveals that the solutions for the load bearing capacity of the single granular pile contained in a soft cohesive subsoil strata have been attempted using analytical approach, empirical methods and experimental investigations. These approaches have certain inherent shortcomings.

2.8.2 Ultimate Bearing Capacity (analytical approaches)

The expression for ultimate bearing capacity has been arrived at by utilising passive pressure of the soft clay deposit surrounding the pile (Greenwood 1970, Hughes & Withers 1974). This approach leads to conservative estimates. The approach considers a two dimensional plastic failure case while the actual problem is a three-dimensional case. Energy concept is yet another approach (Datye & Nagaraju 1977) which has been advocated for the computation of the bearing capacity besides cavity expansion approach (Datye & Nagaraju 1981) which uses the limit pressure required for the expansion of a cylinderical cavity in infinite soil mass subjected to a uniform all round pressure on the wall of the cylinder.

The ultimate bearing capacity has been compared for the two extreme consistency of the clay (soft and stiff clays, for both drained and undrained conditons. The analysis considers the granular pile basically as a shear pin. However, it is observed that in both the drained and undrained cases, the analysis is based on assumptions of (a) sharing of load between pile and soil (b) depth of failure of cylinderical cavity (granular pile) taken 5.33 times the pile diameter and (c) computation of rigidity index for soft and stiff clays or loose to dense non cohesive soils. Basis of these assumptions is not given. Further, it is left to the designer to guess the actual load bearing capacity of the pile after the max. and the min. capacity for the soft to stiff and loose to dense cases have been computed.

The analysis proposed by Madhav and Vitkar (1978) is based on shear failure modes for the computation of the bearing capacity of strip footing placed on granular trench in cohesive soft soils.

The study is limited to strip footings in cohesive soils only thus its utility is limited though the benefit of using the granular trench is clearly demonstrated.

2.8.3 Allowable Pile Capacity (empirical approaches)

Computation of bearing capacity based on empirical approaches attempts to relate the pile capacity with the undrained shear strength of the clay deposit C_u (Thorburn and McVicar 1968, Thorburn, 1975) and are reported to compare well with the allowable load predicted by Hughes and Withers (1974). In developing these empirical charts it is assumed that the total design load is carried by the pile alone and based on this, the total number of pile should be calculated. This assumptions though gives adequate

factor of safety towards bearing capacity and provides adequate stiffness but no attempt is made for settlement predictions. Such an assumption might be true for strip footing resting on granular piles but may not be applicable to raft or embankments which transmit the load to a large area. This fact, though realised by Thorburn (1975) but is attributed to the non availability and complexity of the theoretical solutions. However, the modified chart proposed by Smoltczyk (1979) relating allowable load per pile, undrained shear strength and settlement is definitely a step forward in this direction and deserve attention.

2.8.4 Allowable Pile Capacity (semi-empirical approach)

Semi empirical design approach based on pressuremeter theory (Bishop et.al 1945, Gibson and Anderson, 1961) for the prediction of ultimate bearing capacity of granular piles in clay sub-soil, has been proposed by Hughes and Withers (1974), Hughes et.al (1975). The predicted pile capacity has been reported to compare well with actual load test results (Hughes et.al 1975).

It is of interest to observe that the semi empirical design methodology uses Hughes and Withers (1974) equation for the prediction of ultimate pile capacity and is given by Eqn 2.66 as already expalined.

$$q_{ult} = K_p\left[\sigma'_{ho} + C_u\left(1 + \mathrm{Ln}\frac{E_s}{2C_u(1+\mu)}\right)\right] \qquad \ldots 2.66$$

where

$$k = 1+\mathrm{Ln}\frac{E_s}{2C_u(1+\mu)} = 1 + \mathrm{Ln}(I_r) \qquad \ldots 2.67$$

and the rigidity index (Vesic 1972) I_r for cohesive soils is given by

$$I_r = \frac{E_s}{2C_u(1+\mu)} \qquad \ldots 2.68$$

I_r = 10 of soft clays and 300 for stiff clays with these values of (I_r), the value of k will have a value between 3.33 to 6.7

Therefore

$$q_{ult} = K_p [\sigma'_{ho} + KC_u] \qquad ...2.69$$

The fact which should be borne in mind is that the equation 2.69 is developed on the basis of the assumption that the design load is exclusively sustained by the pile only and is contrary to actual practice. Moreover, it is also indicated that the reliability of the above equation is inter dependent on the values of:

a) Coefficient of passive pressure, K_p

b) Total in-situ lateral initial ground stress, σ'_{ho}

c) Coefficient, k

a) Coefficient of Passive Pressure, K_p

The value of K_p is dependent on Ø' (for granular material) and varies from 3.69, 4 to 5.89 depending on Ø' values from 35°, 38° to 45° as found from actual tests on granular piles (Hughes and Withers 1974, Engelhardt and Golding 1975, Broms 1979) respectively

b) Total In-situ Lateral Initial Ground Stress (σ'_{ho})

This may be found either from undrained shear strength C_u and taken as equal to $2C_u$ or from the limit pressure strain relationship obtained from insitu pressuremeter tests (Hughes et.al 1975). Though the latter is recommended to be preferable. Besides this Mori (1979) has used average effective over burden pressure ($0.5\gamma' h$) for finding the value of σ'_{ho}. Here, h is taken as 5m below ground level and is equivalent to effective overburden pressure at a depth of 2.5m which is nearly 4 times the pile diameter.

c) The Coefficient, k

It is seen that the value of K varies from 3 to 5 Table 2.3 as found by different workers (Mokashi et.al 1976, Hughes Withers, 1974 and Broms 1979) for soft clays. However, the value of k determines the compressibility of the clay surrounding the pile and is dependent on the rigidity index I_r (Vesic 1972). The value of I_r is 10 for soft clay to 300 for stiff clays. The corresponding values of k is found between 3.33 to 6.7. In view of these observations the value of k equal to 4 as proposed by Hughes and Withers (1974) appears to be on conservative side and at the same time it may be agreed that k cannot have a value as 6.7 which is for stiff clays. Thus k equal to 5 appears to be a reasonable approximation as proposed by Broms (1979) and Mori (1979).

It is, therefore, observed that the empirical or semi empirical design methodology available for granular piles in clay needs improvement in view of the divergent opinions by different workers for the values of the various parameters used by the method besides the method does not account for non-cohesive soils and also the share of the design load by the soil surrounding the pile is ignored.

2.8.5 Experimental Studies

The experimental investigations in the laboratory have been very few e.g. Hughes and Withers (1974), Mokashi et.al (1976) and Madhav (1981). Radiographic technique has also been used (Hughes and Withers 1974). Influence of surcharge, rate of loading have been studied. Large scale field investigations have been reported by Hughes et.al (1975) Sheng Ghongwen (1979), Engelhardt and Godding (1975), Greenwood (1970), Datye and Nagaraju (1981).

It is noted that all the models or field tests are limited to investigation on single granular pile reinforced in soft clay deposit. The effect of pile groups has not been considered. Further, though the studies in model incorporates the effect of surcharge, depth of pile penetration and rate of loading in soft clay deposit however, the influence of design load shared by the surrounding clay soil on the ultimate capacity of the piles have not been studied so far.

2.8.6 Prediction of Settlement

An analytical method of predicting settlement of single granular pile in clays have been proposed using finite element and finite difference methods by Balaam et.al (1977). Hughes et.al (1975) has put forward a method based on insitu pressuremeter test, utilising total stress versus radial strain relationship.

Based on empirical approaches Hughes and Withers (1974) and Preba (1976) based on their experiences indicate that settlement of normally consolidated clays without presence of sand lenses is reduced from 1/3 to 1/4 in comparision to un-reinforced clay deposit. While Greenwood (1970) proposed a chart relating settlement ratios of reinforced and unreinforced clays with pile spacing for two types of clays having soft medium undrained shear strength. Besides these many concept have been put forward to assess the settlement of ground reinforced with granular piles such as prediction of settlement based on 'radial strains' (Thorburn 1975) using 'skin friction resistance' (Broms 1979), 'material properties' (Baumann and Baner 1974), compressibility of the pile material (Datye and Nagaraju 1981) and on effective areas covered by granular piles and the soil surrounding the pile Nahrgang (1976). However, none of these methods have so far been tried and compared with actual settlement observations.

2.8.7 Construction Methods

While 'vibro-compaction' is a process to improve the relative density of the non cohesive sub-soil deposit, the process of 'vibro replacement'has been used to construct the granular piles with deep vibrating method all over the world. (Engelhard and Krisch 1977 Baumann and Bauer 1974). The installation of granular piles by using displacement and non-displacement methods for making boreholes and usual heavy piling rigs for compaction has been used for Datye and Nagaraju (1975) the authors call the method as ramming process. Alternatively, preassembled process is also described (loc. cit 1981). However, economic considerations and availability of'vibro float' and 'piling rigs' may restrict the use. Also the preassembled process (Datye and Nagaraju 1981) may not be feasible particularly when the recommended stone aggregate sizes are 20mm to 75mm. This calls for an urgent need to develop simple method of constructing granular piles based on simple equipment fabricated indeginously and uses available labour as man power replacing costly imported machines.

2.8.8 Critical Assessment

In view of the preceeding observations, it may be noted that: Available methods of analysis and design of granular piles with respect to ultimate load carrying capacity of a single granular pile contained in only soft cohesive soil have been considered so far. The limited empirical and semi empirical approaches lead to conservative estimate of the ultimate pile capacity as all the applied load is assumed to be carried by the granular piles only. No attempt has been made to study the behaviour of pile groups.

Though the effect of applied load shared by the surrounding soft

clay sub-soil around the granular pile, on the ultimate pile capacity has been realised by few investigators however no attempt has been made to incorporate the same in the design methodology. This leads to un-economical design and need a rational basis for the consideration of this aspect.

Except for some empirical approaches, no rational method of predicting settlement of soft clay and non cohesive sub-soil reinforced with single and group of granular pile is available. This suggests a need for further research on this aspect.

As discussed earlier, when a single granular pile is subjected to sustained vertical load on pile top, fail by bulging. The length of the bulge is limited to 4-5 pile diameters. Also when these piles are used in group such as under a raft or embankment, the peripheral piles are found to have reduced load bearing capacity due to abrupt disappearance of applied loads. As a remedy to check the bulging of the piles, the following suggestions have been made.

a a) To replace the bulged portion of the pile by concrete piles or

b) Injection of cement mortar is considered as favourable or

c) The upper portion is suitably grouted.

These may help in reducing the overall settlement. Such solutions might prove uneconomical and difficult to execute. In the case of pile group under embankment, the embankment at the edges is provided with 2:1 slope and hence the question of abrupt disappearance of the applied load does not apply. However, in the case of a raft, supported on granular piles, the peripheral piles is likely to have reduced capacity as compared to intermediate piles. In such cases, to increase the passive restraint on the peripherial piles and hence the load bearing capacity, provision of surcharge load around

the raft has been suggested. The model studies on this aspect do confirm the c-ncept of increase in bearing capacity and decrease of settlement over the reinforced clay behaviour. However such solutions may affect the functional requirement of the structure besides being un-economical. These aspects need further research and demand a suitable alternative to these problems urgently for the satisfactory performance.

Besides load bearing capacity of granular piles, prediction of settlement (total and differential) under the design load is another important aspect of the problem. Though, in the literature, there is no attempt to provide a rational method for settlement computation of weak soil reinforced with granular piles except a semi-empirical approach proposed by Greenwood (1970) for cohesive soils. This clearly suggest, the urgent need of a suitable approach to predict the settlement of reinforced weak soils (non cohesive or cohesive) when subjected to design loads. The method should be based on the insitu sub-soil properties such as consistancy, compressibilities, foundation geometry etc. and its relative comparision with case records.

Lastly, a simple and economical alternative to existing construction practices of granular pile is urgently required for both cohesive and non cohesive sub-soil deposits. The methods should be able to utilise local experiences and materials besides indeginous know-how particularly suitable to developing countries like India. The construction method should be based on simple machineries, unskilled and semi-skilled available man power replacing expensive vibrofloats and piling rigs.

CHAPTER III

ANALYSIS AND PROPOSED DESIGN METHODOLOGY

3.0 ANALYSIS AND DESIGN METHODOLOGY

3.1 Introduction

In the recent past the problem of cavities in ideal soil mass has received more attention in solving foundation problems such as bearing capacity of deep foundations, interpretation of pressuremeter test data, cratering by explosives and break out resistance of anchors.

The solution to the problem of expansion of a spherical and cylinderical cavities in a frictionless media ($\emptyset$=0) proposed by Bishop, Hill and Mott (1945) for identation of metals was extended to arrive at the bearing capacity of pile in clay (Gibson 1950). Skempton, Yassin and Gibson (1953) treated the expansion of spherical cavity in a cohesionless medium (C_u=0) to obtain the bearing capacity of piles. The main feature of the approach used in the analysis is that the soil outside the plastic zone is assumed to behave as an ideal elastic and therefore, compressible solid (Vesic 1967). The expansion of a cylinderical ring constituting the radial shear zone below a pile tip was also utilised for the design of pile foundation (Vesic 1977). Interpretation of pressuremeter test data in the case of cylinderical cavity has also been considered (Gibson 1961, Kerisel 1957, Menard 1957 & Vesic 1972). Importance of volume change characteristics for the accurate determination of strength characteristic of soils with high internal friction from pressuremeter tests has also been demonstrated (Vesic 1972), besides break out resistance of anchors in non-cohesive sub-soil (Vesic 1971).

In the present analysis the expansion of cylinderical cavity in a homogeneous isotropic and infinite soil mass has been extended to predict load bearing capacity of a single granular pile in a non cohesive deposit.

3.2 Basic Assumptions

The following assumptions are made in the proposed analytical approach:

1. Granular pile is considered to be less stiff as compared to a concrete pile.

2. The granular pile is assumed to derive the load bearing capacity from the restraint provided by the surrounding soil against the lateral bulging of the granular material constituting the pile.

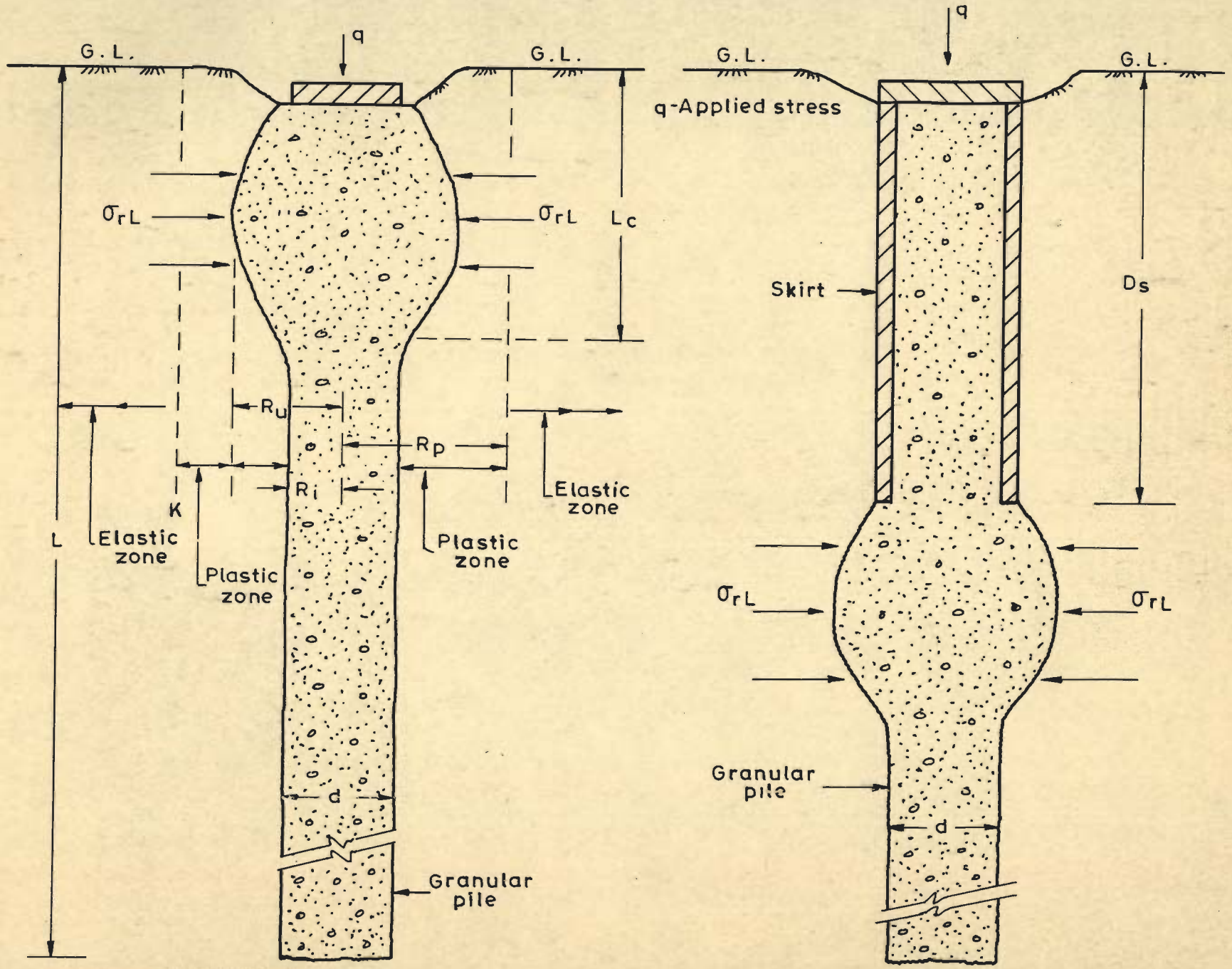
3. When a single granular pile is subjected to sustained vertical loading on the pile top only, the behaviour of such a pile is considered analogous to the case of a free standing pile, (cap not resting condition).

4. The granular pile is considered to fail in bulging at the instant when the lateral stress, σ_r induced in the pile body due to the applied vertical stress, q on the pile top attain their ultimate values σ_{rL} and q_{ult} respectively.

5. The depth of bulging or the critical pile length, L_c is assumed to be limited to 4-5 pile diameter Fig. 3.1 (a).

6. The pile is assumed to fail in bearing when the length of the pile L is less than the critical length L_c.

7. The failure modes of an individually skirted pile and collectively skirted granular pile group is assumed in accordance with Fig. 3.1 (b) and Fig. 3.2



(a) Plain granular pile

(b) Individual skirted pile

FIG.3.1. BUILDING FAILURE IN PLAIN AND INDIVIDUALLY SKIRTED GRANULAR PILE

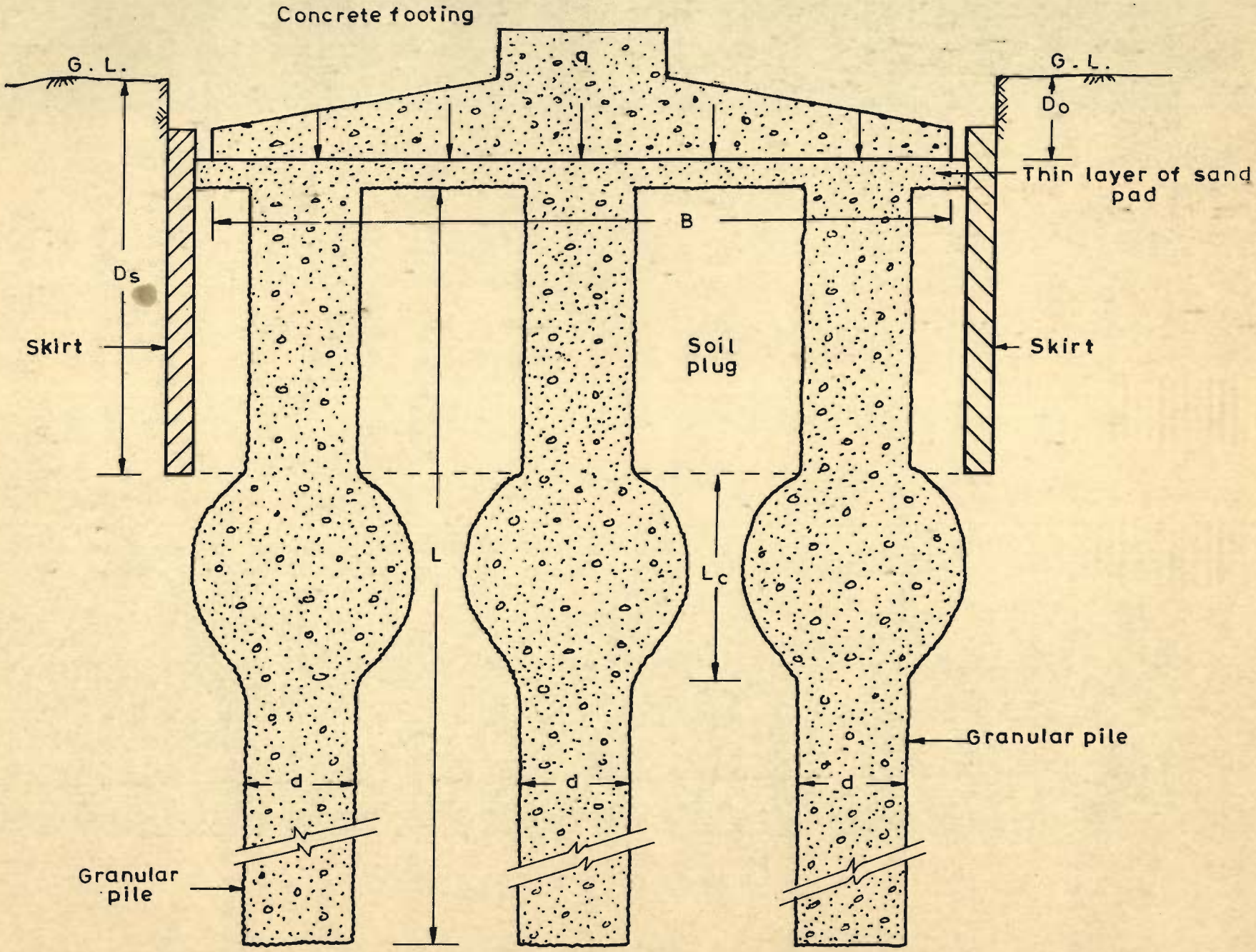


FIG. 3.2. BULGING FAILURE MODE OF COLLECTIVELY SKIRTED GRANULAR PILE GROUP

8. The bulging failure mode of a single granular pile is considered analogous to the expansion of a cylinderical cavity having the diameter, d equal to the pile diameter and depth equal to critical pile length L_c.

9. The portion of the pile equal to critical pile length is considered to bulge due to a uniform induced lateral stress, σ_r in a homogeneous, infinite isotropic non cohesive sub-soil deposit. Due to gradual increase in applied stress q, and consequently σ_r in the pile body, the cylinderical zone around the bulged pile Figs. 3.1 (a) and 3.3, will pass into the state of plastic equilibrium . Beyond the zone of plastic equilibrium the soil is considered to remain in elastic equilibrium condition.

10. The total applied load q, is assumed to be shared between the granular pile, q_p and the surrounding soil, q_s in accordance with their respective corrected elastic modulii, E'_p and E'_s

3.3 Lateral Limit Stress, σ_{rL}

Considering the problem of a spherical cavity of initial radius Ri (Fig. 3.3) expanded by a uniform pressure σ_r. If this pressure is increased spherical zone around the cavity will pass into the state of plastic equilibrium. This plastic zone will expand untill the pressure reaches an ultimate value or the limit stress σ_{rL}. At this moment the cavity will have a radius R_p, beyond that radius the rest of the mass remains in the state of elastic equilibrium. To determine limit pressure σ_{rL} and the radius R_p it is assumed that the soil in the plastic zone behaves as a

$\sigma_r = \sigma_{rL}$
$q = q_{ult}$
AT THE INSTANT OF BULGING FAILURE

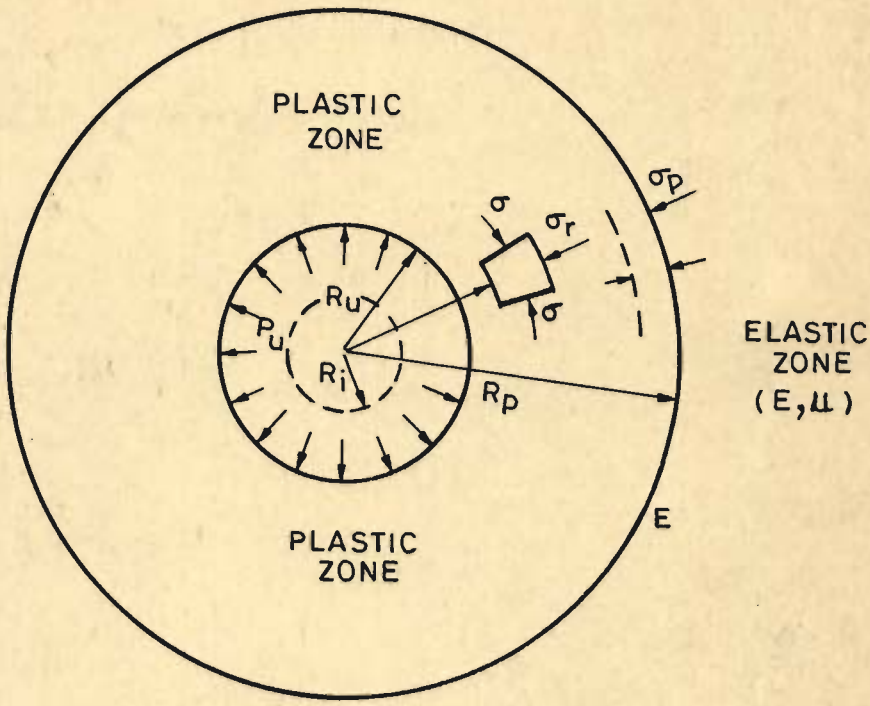


FIG. 3.3. EXPANSION OF CAVITY

compressible plastic solid defined by Coulomb-Mohr shear strength parameters c and $\emptyset$ average volumetric strain Δ which can be determined from known state of stress in the plastic zone and the volume change relationships. Beyond the plastic zone the soil is assumed to behave as a linearly deformable, istropic solid defined by a modulus of deformation, E_s and Poissions ratio, μ . It is further assumed that prior to the application of load the entire soil mass has an isotropic effective stress (σ_m) and that the body forces within the plastic zone are negligible when compared with existing and newly applied stresses (Vesic, 1972).

The problem of cylinderical cavity is completely analogous to spherical cavity with the difference that it is axially symmetrical instead of spherically symmetrical.

When shearing stresses on the element, Fig. 3.3 vanish, thus for cylinderical cavity the equations of equilibrium, for axial symmetry is given by the following equation

$$\frac{\partial \sigma_r}{\partial r} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \qquad \ldots 3.1$$

Satisfying the condition of rupture and the limit equilibrium the solution of differential equation is:

$$\sigma_r = (\sigma_{rL} + c \text{ Cot } \emptyset)\left(\frac{R_u}{R_p}\right)^{\frac{2 \sin\emptyset}{1+\sin\emptyset}} - c \text{ Cot } \emptyset \qquad \ldots 3.2$$

which for $\emptyset = 0$ reduces to

$$\sigma_r = \sigma_{rL} - 2c \text{ Ln } \left(\frac{R_p}{R_u}\right) \qquad \ldots 3.3$$

Finding the radial displacement and normal stress the value of R_p is found from the Equation 3.4

$$\frac{R_p}{R_u} = [I'_{rr} \text{ Sec } \varphi]^{0.5} \qquad \ldots 3.4$$

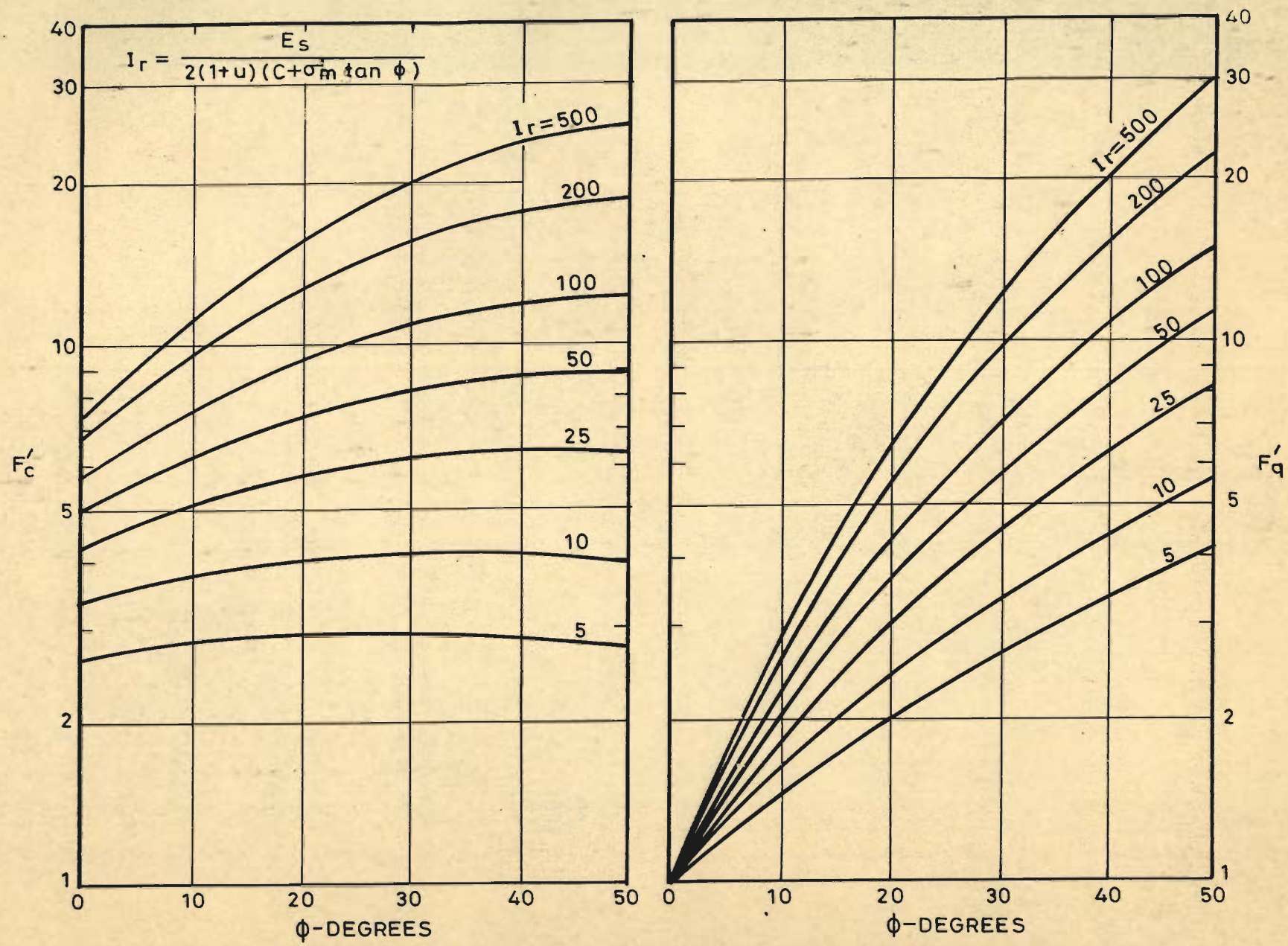


FIG. 3.4. CYLINDRICAL CAVITY EXPANSION FACTORS
(AFTER VESIC 1972)

in which I'_{rr} is the reduced rigidity index and is given by Equation 3.5

$$I'_{rr} = \frac{I_r}{1 + I_r \quad \text{Sec } \emptyset} \qquad \ldots 3.5$$

where rigidity index I_r is defined as

$$I_r = \frac{E_s}{2(1+\mu)(c+\sigma_m \tan \emptyset)} = \frac{G}{S} \qquad \ldots 3.6$$

where $_m$ = effective mean normal stress

This shows that the ratio R_p/R_u depends primarily on rigidity index of the soil. It represents the ratio of the shear modulus G of the soil to its initial shear strength(s). Typical values of rigidity index for different known media found by Vesic (1972) together with the typical resulting ratio of R_p/R_u is given in Table 3.1

Table 3.1 - Typical Values of Rigidity Index I_r

Medium	Rigidity Index I_r	Relative Radius of plastic zone R_p/R_u (If $\emptyset = 0$)	
		Cylinderical cavity	Spherical cavity
Rock	100-500	12-25	5-8
Loose-dense sand under a normal stress of $10t/m^2$	70-150	9-15	4-6
Saturated clay soft to stiff	10-300	3-17	2-7
Micaceous silt	10-30	3-6	2-3
Mild Steel	300	17	7

It may be noted that the rigidity index varies with mean normal stress σ_m(Vesic 1977). For incompressible solids $\Delta = 0$ and I_{rr} becomes equal to I_r.

Knowing the relative radius R_p/R_u the ultimate cavity pressure of ultimate limit stress σ_{rL} can be computed from Equation 3.4 and the result can be put in the following form:

$$\sigma_{rL} = cF'_c + \sigma_m F'_q \qquad ...3.7$$

in which

F'_c and F'_q are dimensionless cylinderical cavity expansion factors, defined as

$$F'_q = (1+ \sin\emptyset)\ (I'_{rr} \sec \emptyset)\ \ \sin \emptyset/1+\sin \emptyset. \qquad ...3.8$$

$$F'_c = (F'_q-1) \cot \emptyset \qquad ...3.9$$

For $\emptyset = 0$ and $\Delta = 0, \mu = 0.5$ and $K_o = 1$

$$F'_c = 1 + Ln\ (I_r) \qquad ...3.10$$

It will be noted that the equation 3.10 is identical with the value of limit pressure σ_{rL} found by Gibson and Anderson (1961) on the basis of pressuremeter theory. The numerical values of F'_c and F'_q may be found from Table 3.2 or Fig. 3.4

When cohesion C = 0, for non cohesive case

$$\sigma_{rL} = \sigma_m F'_q \qquad ...3.11$$

where σ_{rL} is the limit pressure

The limit pressure σ_{rL} is a function of angle of shearing resistance $\emptyset$

of the non cohesive sub-soil deposit, cohesion c and the rigidity index (I_r).

3.4 Correction for the Elastic Modulii

If E_s is the elastic soil modulus of the surrounding soil found from field tests, $_m$ is the effective mean normal stress of the soil at a depth z equal to critical pile length and $_1$ is the effective normal stress equal to 1 kg/cm^2 (10t/m^2) then it is assumed that the corrected soil modulus E'_s is represented by the expression 3.12.

$$E'_s = E_s \left[\frac{\sigma_m}{\sigma_1}\right]^{0.5} \qquad ...3.12$$

and similarly for the material constituting the granular pile

$$E'_p = E_p \left[\frac{\sigma_m}{\sigma_v}\right]^{0.5} \qquad ...3.13$$

where E'_p and E_p are the corrected and measured elastic soil modulus of the pile material. The multiplying factor $\left[\frac{\sigma_m}{\sigma_1}\right]^{0.5}$ make use of normal stress with the purpose of making it dimensionless.

The measured elastic modulus E_p, for the pile material and E_s for the surrounding soil is corrected by multiplying a factor equal to $\left[\frac{\sigma_m}{\sigma_1}\right]^{0.5}$ where σ_m is the effective mean normal stress at a depth equal to critical pile length L_c and σ_1 is the normal stress equal to 1 kg/cm^2 or 10t/m^2. The modulus of deformation E_p or E_s for the pile material and cohesionless soil increases with increase of the confining stress (DE BEER 1967). For the case where initial ground stress is isotropic, the modulus of deformation varies as the square root of the initial ground stress (Lambe and Whitman 1969) and art. 3.7). In view of the fact that the effect of actual state of stress on the modulus of deformation is not clearly known it is justified to assume that the soil modulus varies with the

square root of the effective mean normal stress σ_m . It is probably due to this fact the unit point resistance of piles in sand is not governed by effective normal stress σ_v but by effective mean normal stress σ_m. (Al Awakati and Vesic, 1972, Al Awakati 1975). Thus multiplying correction factor is found by making it non dimensional by dividing it by square root of σ_1 taken as $10t/m^2$.

3.5 Load Distribution Between the Granular Pile and the Soil

In actual loading conditions the applied load, q is distributed between granular pile and the surrounding soil. For this purpose the experimental structures and model tests in which only pile top is loaded and surrounding soil is maintained in vertical state of stress do not correspond with the actual conditions where surrounding soil is also loaded. In these circumstances a different state of shear between the granular pile and the surrounding insitu soil exists (Floss 1979). Though some efforts appear to have been made in this direction (Maggio 1978, Broms 1979, Baumann and Bauer 1974 and Datye and Nagaraju 1981), without any success. Broms (1979) assign an arbitrary value of 80 per cent of the design load shared by the pile. Datye and Nagaraju (1981) assumed an arbitrary value of the load shared by the granular pile and the surrounding soil without any rational basis. Though Baumann and Bauer (1974) proposed a method of distribution of the applied load between piles and soil based on analytical model. The computed settlements based on this distribution resulted in a conservative estimate of consolidation settlement (refer art. 2.5.4). However, no attempt was made by the authors to study the improvement in pile capacity due to sharing of the applied load by surrounding soil and subsequently to predict the pile load capacity.

Table 3.2 Cylindrical Cavity Expansion Factors

	Ø, in degrees									
I_{rr}	0	5	10	15	20	25	30	35	40	45
5	2.61[a]	2.71	2.79	2.86	2.90	2.92	2.93	2.92	2.89	2.84
	1.00[b]	1.24	1.49	1.77	2.05	2.36	2.69	3.04	3.42	3.84
10	3.30	3.52	3.71	3.86	3.99	4.08	4.14	4.17	4.16	4.11
	1.00	1.31	1.65	2.04	2.45	2.90	3.39	3.92	4.49	5.11
15	3.71	4.01	4.29	4.52	4.72	4.88	4.99	5.06	5.08	5.05
	1.00	1.35	1.76	2.21	2.72	3.27	3.88	4.54	5.26	6.05
20	4.00	4.37	4.72	5.03	5.29	5.50	5.67	5.77	5.82	5.82
	1.00	1.38	1.83	2.35	2.93	3.57	4.27	5.04	5.89	6.82
25	4.22	4.66	5.07	5.44	5.76	6.03	6.24	6.38	6.46	6.48
	1.00	1.41	1.89	2.46	3.10	3.81	4.60	5.47	6.42	7.48
30	4.40	4.90	5.36	5.79	6.17	6.48	6.74	6.92	7.03	7.06
	1.00	1.43	1.95	2.55	3.24	4.02	4.89	5.85	6.90	8.06
40	4.69	5.28	5.84	6.37	6.84	7.25	7.59	7.84	8.01	8.08
	1.00	1.46	2.03	2.71	3.49	4.38	5.38	6.49	7.72	9.08
50	4.91	5.58	6.23	6.84	7.40	7.90	8.31	8.63	8.85	8.96
	1.00	1.49	2.10	2.83	3.70	4.68	5.80	7.04	8.43	9.96
60	5.09	5.83	6.55	7.25	7.89	8.46	8.94	9.32	9.59	9.74
	1.00	1.51	2.16	2.94	3.87	4.94	6.16	7.53	9.05	10.74
70	5.25	6.04	6.84	7.60	8.31	8.95	9.50	9.94	10.26	10.45
	1.00	1.53	2.21	3.04	4.03	5.17	6.49	7.96	9.61	11.45
80	5.38	6.23	7.09	7.92	8.70	9.40	10.01	10.51	10.88	11.10
	1.00	1.55	2.25	3.12	4.17	5.38	6.78	8.36	10.13	12.10
90	5.50	6.40	7.31	8.20	9.05	9.81	10.48	11.03	11.45	11.71
	1.00	1.56	2.29	3.20	4.29	5.58	7.05	8.73	10.61	12.71
100	5.61	6.55	7.51	8.46	9.37	10.19	10.92	11.52	11.98	12.27
	1.00	1.57	2.33	3.27	4.41	5.75	7.30	9.07	11.05	13.27
150	6.01	7.14	8.33	9.52	10.69	11.77	12.75	13.58	14.24	14.70
	1.00	1.63	2.47	3.55	4.89	6.49	8.36	10.51	12.95	15.70
200	6.30	7.58	8.94	10.33	11.71	13.01	14.21	15.24	16.08	16.69
	1.00	1.66	2.58	3.77	5.26	7.07	9.20	11.67	14.49	17.69
300	6.70	8.21	9.84	11.55	13.28	14.96	16.51	17.90	19.05	19.92
	1.00	1.72	2.74	4.10	5.83	7.97	10.53	13.53	16.99	20.92
400	6.99	8.66	10.52	12.49	14.50	16.48	18.35	20.04	21.46	22.57
	1.00	1.76	2.85	4.35	6.28	8.69	11.59	15.03	19.01	23.57
500	7.21	9.03	11.06	13.25	15.51	17.76	19.90	21.85	23.53	24.86
	1.00	1.79	2.95	4.55	6.64	9.28	12.49	16.30	20.75	25.86

[a]Upper number F'_c for each value of I^1_{rr}

[b]Lower number F'_q for each value I^1_{rr}

In the present design methodology, the total applied load, Q is assumed to be shared by granular piles, Q_p having total cross sectional area after installation equal to A_p, and the surrounding soil, Q_s under the footing having an area equal to A_s. If A is the total area of the footing base which is defined as $(B \times L_f)$, B and L_f are width and length of the footing and α is the relative area of the pile (A_p/A), then the total applied load Q on an area A as indicated earlier (art. 3.2 - 10) may be expressed as:

$$Q = Q_p + Q_S \qquad ...3.14$$

$$\text{and } A = A_p + A_s \qquad ...3.15$$

$$\text{or } q = q_p + q_s \qquad ...3.16$$

$$\text{where } q = \frac{Q}{A}, \; q_p = \frac{Q_p}{A_p}, \; q_s = \frac{Q_s}{A_s} \text{ and } \alpha = \frac{A_p}{A}$$

As indicated earlier (art 3.2-10), the unit stress (q_p) shared by the piles and (q_s) by the surrounding soil are proportional to their respective modulii E'_p and E'_s which may be expressed as:

$$\frac{q_p}{q_s} = \frac{E'_p}{E'_s} \qquad ...3.17$$

This assumption is based on the anology of soil reinforced with granular piles with those of concrete reinforced with steel.

The equation 3.14 may be written as:

$$\frac{Q_p}{A_p} = \frac{Q_S}{A_S} \cdot \frac{E'_p}{E'_s} = \frac{Q - Q_p}{A - A_p} \; \frac{E'_p}{E'_s} \qquad ...3.18$$

Multiplying both LHS and RHS by A_p and then dividing RHS

numerator and denominator by the total area, A we have

$$Q_p = \frac{\frac{A_p}{A}(Q-Q_p)\frac{E'_p}{E'_s}}{\frac{1}{A}(A-A_p)} = \frac{(Q-Q_p)\alpha E'_p}{(1-\alpha)E'_s} \qquad ...3.19$$

where (α) is the relative pile area

Thus equation 3.17 may be expressed as

$$Q_p = \frac{Q.\alpha . E'_p}{\alpha E'_p + (1-\alpha)E'_s} \qquad ...3.20$$

In terms of unit stress (q_p) and (q) one can rewrite the above equation as

$$q_p = q\left[\frac{E'_p}{\alpha E'_p + (1-\alpha)E'_s}\right] \qquad ...3.21$$

Similarly,

$$q_s = q\left[\frac{E'_s}{\alpha E'_p + (1-\alpha)E'_s}\right] \qquad ...3.22$$

3.5.1 Equivalent Elastic Modulus (E_{eq})

In equations 3.21 and 3.22 the denominator in the RHS represents the equivalent corrected modulus (E'_{eq}) of the composite mass consisting of soil reinforced with granular piles thus:

$$E'_{eq} = \alpha E'_p + (1-\alpha)E'_s \qquad ...3.23$$

where (α) is the relative pile area and E'_p and E'_s are the respective corrected modulii of the pile material and the surrounding soil.

3.5.2 Modular Ratio, m'

If m' is the modular ratio of the pile material and the soil and

represented by equation 3.24

$$E'_p = m' E'_s \qquad ....3.24$$

Thus in terms of modular ratio equation 3.23 representing equalent modulus (E'_{eq}) may be written as

$$E'_{eq} = 1 + (m' - 1)\alpha \; E'_s \qquad ....3.25$$

and therefore, equations 3.21 and 3.22 can be expressed in terms of modular ratio and relative pile area

$$q_p = q \frac{m'}{1 + (m'-1)\alpha} \qquad ....3.26$$

$$\text{and } q_s = q \frac{1}{1 + (m' - 1)\alpha} \qquad ....3.27$$

It can be seen from equation 3.21 that the load shared by the pile may readily be found if the factor enclosed in the bracket representing ratio of the modulus of pile material (E'_p) and the equivalent modulus (E_{eq}) of the composite mass consisting of soil reinforced with granular piles are known. Similarly the load shared by the soil may be obtained from equation 3.22 if the ratio of corrected soil modulus (E'_s) and the equivalent modulus (E'_{eq}) is known. Thus the computation of the load sharing by the pile and surrounding soil involves determination of equivalent modulus (E'_{eq}) which in turn involves determination of E'_p,E'_s and relative pile area (α) consisting of total pile area A_p and the total area A under the footing. The elastic parameters E'_p and E'_s can be determined from field tests such as standard penetration tests (SPT) or static or dynamic cone penetration tests. The installed pile diameter of a single pile may however, be assumed 20-30 per cent higher over the initial pile diameter for medium dense to loose non cohesive soil deposit for preliminary design. In the final design the installed

pile diameter may however, be revised based on the average volume of the material consumption (stone aggregates and sand mixture) during pile installation for the check load testing.

The applied load shared by the surrounding soil (q_s) results in increasing the mean effective normal stress (σ_m). This effect of load sharing by the surrounding soil results in increasing the restraining pressure acting against the expansion of the pile due to bulging and thereby increasing the load bearing capacity of granular pile.

3.6 Coefficient of Earth Pressure at Rest

When a granular soil is loaded, the frictional forces at the contacts act in such a direction that horizontal stress is less than the vertical stress (Lambe and Whitman 1969). This means that the coefficient of lateral earth pressure at rest K_o is less than unity. The magnitude of K_o therefore depend on the amount of frictional resistance mobilised at contact points between the particles. Experimental values of K_o is best represented by the following equations 3.28 (Jaky 1944)

$$K_o = (1 - \sin \emptyset) \qquad ...3.28$$

However Vesic (1972, 1975) suggests an improvement over this for non-cohesive soils (C = 0)

$$K_o = 1 - \sin 1.2\ (\emptyset) \qquad ...3.29$$

3.7 Confining Stress

For a given sand, the variation of the secant angle $\emptyset'$ as a function of the porosity n and mean normal stress σ_m can be determined by means of triaxial test in the laboratory. It has been

demonstrated (DE BEER 1967) that when the porosity n is 40 per cent the secant angle Ø' is 45^0 for a mean normal stress σ_m of 0.02 kg/cm^2 and drops to 33^0 when σ_m = 50 kg/cm^2. Similarly when the confining stress is increased, the soil modulus E_s also increases. For the case where the initial ground stress σ_m is isotropic, the soil modulus is proportional to the initial ground stress σ_m and is expressed as given by equation 3.30.

$$E_s \sim \sigma_o^{\,n} \qquad \ldots 3.30$$

The exponent n varies between 0.4 to 1 and a reasonable average of 0.5 may be acceptable (Lambe and Whitman 1969). In most practical problems however, the initial ground stress is not isotropic and probably due to this fact, Al Awakati and Vesic (1972) and Al Awakati (1975) have demonstrated beyond doubt that the unit point resistance of piles in sand is not governed by the vertical stress σ_v but by the mean normal stress σ_m. The effect of actual state of stress on soil modulus is not clear but the best available rule is that the soil modulus depend on the average of principal stresses i.e. mean normal stress σ_m (Lambe and Whitman, 1969).

3.7.1 Effective Mean, Normal Stress

The mean normal stress σ_m is related to the vertical ground stress σ_v (Vesic 1977) by the equation 3.31

$$\sigma_m = 1/3 \quad 1 + 2K_o \quad \sigma_v \qquad \ldots 3.31$$

3.7.2 Elastic Modulus of Soil Deformation (E_s)

The elastic soil modulus (E_s) varies with mean normal stress σ_m as given by equation (3.32) (Lambe and Whitman, 1969)

$$E_s \quad \sigma_m^{\,0.5} \qquad \ldots 3.32$$

The equation 3.32 holds good only when coefficient of lateral earth pressure at rest (K_o) lies between 0.5 to 2, and the factor of safety against failure is 2 or more. Thus the modulus of deformation (E_s) of sand increases with mean normal stresses (σ_m) in low and elevated pressure range Vesic (1975).

3.7.3 Poissons's Ratio (μ)

The Poisson's Ratio (μ) is defined by equation 3.33 according to elastic theory

$$\mu = \frac{K_o}{1+K_o} \qquad \ldots 3.33$$

3.8 Proposed Design Methodology

The ultimate load carrying capacity of a collectively skirted pile group is considered as the sum total of the ultimate capacity of the skirted foundation above the horizontal plane YY and the ultimate capacity of the granular pile group below the plane YY, (Fig. 3.5)

3.8.1 Load Bearing Capacity of Skirted Foundation

The skirted granular pile foundation consists of the soil plug reinforced with granular piles confined with a skirt around the soil plug with the footing cast in-situ on the soil plug. The footing and the skirt has a clear space (12mm to 25mm) to enable the footing to settle independently inside the skirt. The applied load Q is transmitted through the footing to the soil plug which is resisted by

(a) The subsoil below the horizontal plane YY reinforced with granular pile.

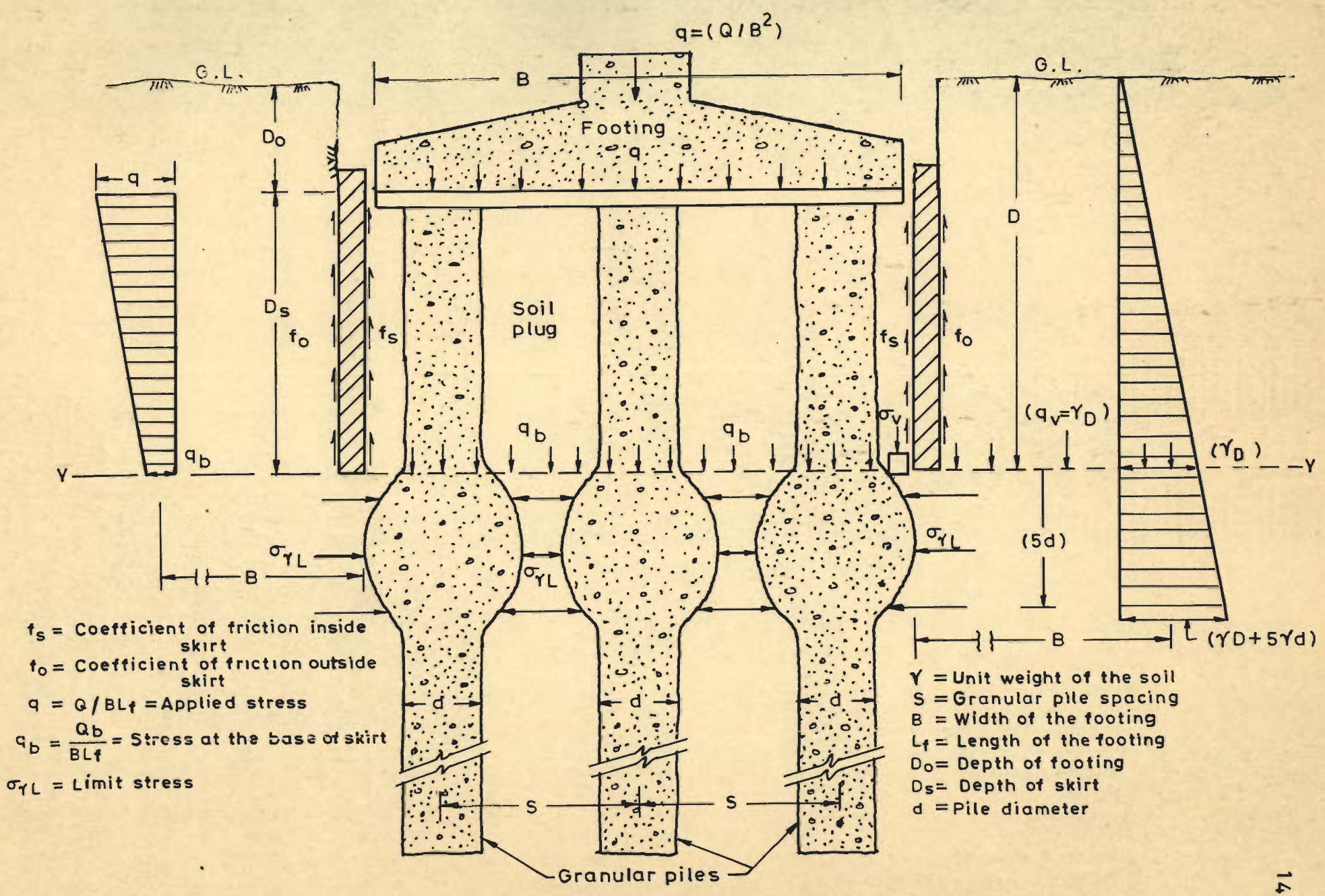


FIG. 3.5. STATE OF STRESSES AT ULTIMATE LOAD IN SKIRTED GRANULAR PILE GROUP

(b) The resistance offered due to inside friction between the soil plug and the skirt interface which is given by the equation 3.34

The resistance due to inside friction = $\frac{1}{2}\gamma D_s K_o \tan\delta \; 2(B+L_f) \; D_s$ ..3.34

The average earth pressure coefficient K'_s on the shaft of driven pile was found (Meyerhof 1951) to vary from 0.5 for loose sand ($\emptyset=30^o$) to 1 for very dense sand ($\emptyset=45^o$). Analysis of model tests (Zweck 1954) and field observations (Mansur and Kaufman 1956) of driven piles in sand shows the earth pressure coefficient K'_b on the shaft within the failure zone near the base equal to 70 per cent of the average K'_s values. Thus the earth pressure coefficient K'_b may be taken to vary from 0.4 in loose sand to 0.7 in dense sand having a variation of $\emptyset$ from 30° to 45°, (Meyerhof 1959). In view of this fact, in computation of friction between the soil plug and the wall of the skirt, use of K_o; earth pressure coefficient is justified. When the soil plug reinforced with granular piles is subjected to a uniform intensity of stress q, the footing cast on the soil plug under goes a little deformation in the initial stages of loading, while the skirt confining the soil plug does not move. However with the further increase of load the soil plug and the skirt moves down together at the same rate and same amount of deformation is observed, indicating as if soil plug, skirt and the footing is one block. In view of this fact, the effect of applied stress, q in the computation of inside friction between the soil may be ignored.

(c) The resistance due to outside friction between surrounding soil and the skirt, which is given by the equation 3.35.

Outside friction = $\frac{1}{2}\gamma \; (D+D_o)K_o \tan\delta \; [\, 2(B+L_f)D_s \,]$ ...3.35

The resistance offered by the friction on outside the skirt having small depth as compared to the width of the footing, is negligible and also due to the fact that at ultimate load the movement of the skirt is likely to be very small as observed in the field test and hence may be ignored.

(d) If the influence zone is limited to three times the footing width then

Resistance due to over burden pressure = $\frac{1}{2}\gamma\ DK_o\ \ 2B\ L_f$ ...3.36

Since the effect of overburden pressure will be considered in the analysis of the bearing capacity of granular pile (item 3.8.1 a) below the plane YY hence it is not included here. In view of the preceding discussion.

$$Q = Q_b + K_o\ \gamma\ D_s\ \tan\delta\ \ 2(B+L_f)\ \ D_s \qquad ...3.37$$

or

$$q\ (BxL_f) = q_b(BL_f) + \tfrac{1}{2}\ K_o\gamma\ D_s\ \tan\emptyset\ \ 2(B+L_f)D_s \qquad ...3.38$$

where

q is the applied unit stress,

q_b is the unit stress at the base of the skirt.

Also in view of the fact that the shearing plane due to movement of the soil plug inside the skirt is likely to be slightly away from the interface of the skirt, hence the angle of wall friction (δ) is taken equal to angle of internal friction of the soil ($\emptyset$)

The equation 3.38 is re-written as:

$$q = q_b + \frac{(\tfrac{1}{2}\ K_o\gamma\ D_s)\ \tan\emptyset\ \ 2(B+L_f)D_s}{B.L_f} \qquad ...3.39$$

In the equation 3.39, the term ($\frac{1}{2} K_o \gamma D_s$) is the average horizontal stress (σ_h) on the inside face of the skirt Fig. 3.6

Based on the actual measurements of horizontal stresses (σ_h) on the wall of a rigid cylinder filled with sand column and loaded through a loading ram, Gupta et.al (1972) demonstrated that the horizontal stress distribution in terms of applied stress expressed in percentage, varied from 38 per cent at the base of the loading ram and reducing to 32 per cent and 25 per cent at a depth equal to B/2 and B respectively (B = width of loading ram), Fig. 3.6. It was further demonstrated that almost 95 per cent of the applied load is transferred to a depth equal to B/2 of the loading ram and reduces to about 70 per cent and 20 per cent at depths equal to B and 2B. Fig. 3.6. If the loading of the sand column through the ram inside a rigid cylinder is considered analogous to the loading of the soil plug confined by the rigid skirt, then the value of average horizontal stress (σ_h) on the wall of the skirt may be taken conservatively as 40 per cent of the applied load. Therefore, the term ($\frac{1}{2} K_o \gamma D_s$) in equation 3.39 is replaced by (0.4q), which is average horizontal stress on interface of the skirt. Thus

$$q = q_b + \frac{0.4q \tan \emptyset \;\; 2(B+L_f)D_s}{B.L_f} \qquad ...3.40$$

$$\text{or } q = \frac{q_b}{1 - \dfrac{0.8\tan \emptyset \;\; (B+L_f)\, D_s}{B.L_f}} \qquad ...3.41$$

$$\text{or } q = \frac{q_b \, BL_f}{BL_f - 0.8\, D_s \tan \emptyset (B+L_f)} \qquad ...3.42$$

When the stress q_b on the horizontal plane YY, Fig. 3.5, reaches its ultimate value $(q_{ult})_1$ then the stress (q) becomes critical $(q_{ult})_2$ where $(q_{ult})_1$ is the ultimate bearing capacity of the

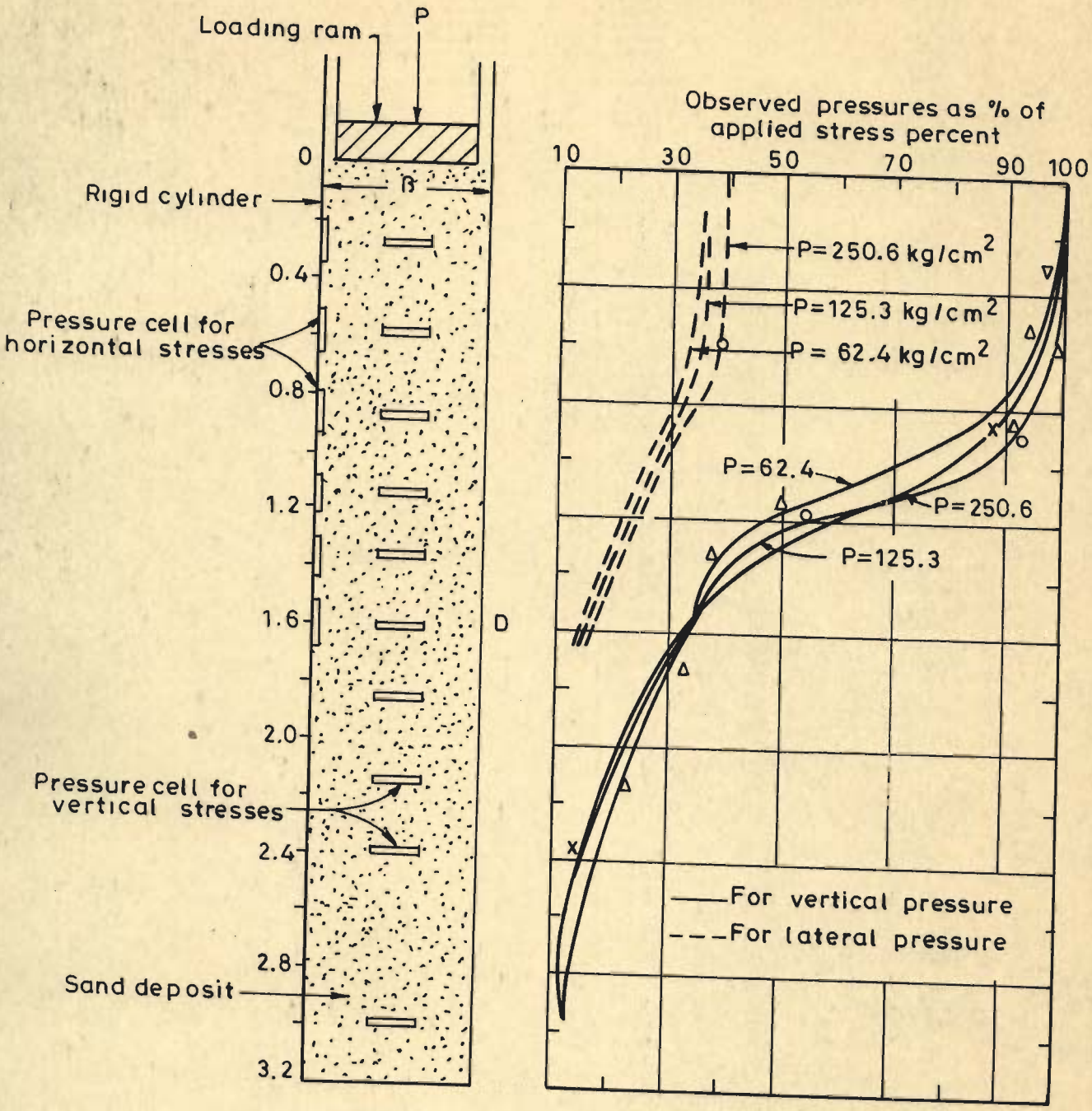


FIG.3.6. VERTICAL AND HORIZONTAL PRESSURE DISTRIBUTION INSIDE A SKIRT IN SAND

granular pile group below the plane YY and $(q_{ult})_2$ is the bearing capacity of skirted footing above the plane YY.

$$\text{Thus } (q_{ult})_2 = \frac{(q_{ult})_1 \, B.L_f}{BL_f - 0.8\, D_s \tan \emptyset \, (B+L_f)} \qquad \ldots 3.43$$

$$\text{and } q_{safe} = \frac{(q_{ult})_2}{F.S.=3} \qquad \ldots 3.44$$

For a square footing $B = L_f$ hence equation 3.42 becomes

$$(q_{ult})_2 = \frac{(q_{ult})_1}{(1 - 1.6 \frac{D_s}{B} \tan \emptyset)} \qquad \ldots 3.45$$

and for circular footing with diameter (B), the ultimate bearing capacity of skirted footing above plane YY is given by:

$$(q_{ult})_2 = \frac{(q_{ult})_1}{1-0.4q \frac{D_s}{B} \pi \tan \emptyset} \qquad \ldots 3.46$$

3.8.2 Load Bearing Capacity of Granular Pile Group

The ultimate bearing capacity $(q_{ult})_1$, of the sub-soil deposit reinforced with granular pile group below the plane YY, Fig. 3.5 is taken as equal to ultimate bearing capacity of a single granular pile expressed in unit stress multiplied by the total number of piles in the group assuming the group efficiency as unity.

The ultimate bearing capacity of a single granular pile can be estimated as detailed below:

(a) (i) Input Data

Effective angle of shearing resistance of non cohesive sub-soil ...$\emptyset$ degrees

Effective cohesion, c = $c(t/m^2)$

Unit weight of the soil = γ_{bulk} (t/m^3)

Static cone penetration resistance = q_c (t/m^2)

or standard penetration test value = N_{SPT}

Proposed granular pile diameter after installation = d(m)

(It may be taken as 1.2 to 1.30 times the diameter of the initial borehole for preliminary design)

Length of the pile = L(m)

Critical pile length (L_c) = 5d(m)

(ii) Coefficient of Lateral Earth Pressure (K_o)

$$K_o = 1 - \sin(1.2\ \emptyset) \qquad ...3.29$$

(iii) The Poisson's Ration (μ)

$$= \frac{K_o}{1+K_o} \qquad ...3.33$$

(iv) Effective Vertical Stress ($_v$)

Since the failure of the granular pile occurs at a depth equal to critical length of the pile L_c which is five times the pile diameter

$$\sigma_v = \gamma_{bulk}(5d) \quad ...t/m^2 \qquad ...3.47$$

(v) Effective Mean Normal Stress (σ_m)

The mean normal stress is found from the expression

$$\sigma_m = 1/3(1+2K_o)\sigma_v \quad t/m^2 \qquad ...3.31$$

(vi) Elastic Soil Modulus (E_s)

The elastic soil mdoulus is found out either from

$$E_s = 2\ (1+2D^2_r)\ q_c \qquad ...3.48$$

Where

D_r is the relative density of the non cohesive sub-soil deposit

or from Begemann (1974) method based on SPT values. The relative density D_r is suitably taken from static cone penetration resistance (Meyerhof 1976) or SPT (Peck Hanson and Thornburn 1974) or/and Table 3.3

Table 3.3 Relative Density and SPT Values

N_{SPT} (No.)	Compactness	Relative density D_r (per cent)	Angle of shearing resistance (Ø) degrees
Less than 8	Very loose	Less than 30	Less than 28.5
10-30	Medium	30 - 60	30 - 36
30-50	Dense	60 - 75	36 - 41
Greater than 50	Very dense	Greater than 75	Greater than 41

3.8.2(b) Corrected Soil Modulus (E'_s)

The values of E'_s is found from the measured value of E_s from the following expression:

$$E'_s = E_s \left[\frac{\sigma_m}{\sigma_1} \right]^{0.5} \quad t/m^2 \qquad ...3.49$$

where

σ_1 is taken as $10t/m^2$

3.8.2(c) Rigidity Index (I_r)

The value of rigidity index I_r is found from the equation:

$$I_r = \frac{E'_s}{2(1+\mu)(c+\sigma_m \tan\emptyset)} \qquad ...3.50$$

for c = 0 cohesionless soils

$$I_r = \frac{E'_s}{2(1+\mu)\,(\sigma_m \tan\emptyset)} = \frac{G}{S} \qquad ...3.51$$

where G is the shear modulus. I_r is the dimensionless number.

3.8.2 (d) Cavity Expansion Factors

For known values of rigidity index I_r and angle of shearing resistance $\emptyset$, cavity expansion factors F'_q and F'_c is found from Fig. 3.4 or Table 3.2. It may be seen that for non cohesive sub-soil deposit (c=0), the limit stress σ_{rL} is found from equation (3.52)

$$\sigma_{rL} = \sigma_m F'_q \qquad ...3.52$$

3.8.2 (e) Ultimate Vertical Stress (q_{ult}) of a single Granular Pile

$$(q_{ult}) = K_p \sigma_{rL} \qquad ...3.53$$

3.8.2 (f) Corrected Pile & Soil Modulii

In the above equation E'_p is found from the equation 3.54

$$E'_p = E_p \left[\frac{\sigma m}{10} \right]^{0.5} \qquad ...3.54$$

where E_p is the elastic soil modulus of the pile material and E'_p is the corrected elastic soil modulus. The value of E_p is found from field tests such as static or dynamic cone penetration tests or from the assumed value based on experience. Numerous full scale field tests allow to safely assume a value equal to 600 kp/cm^2 (600 kg/cm^2) for the elastic soil modulus for the pile material which is well compacted (Engelhardt and Krisch 1977) which was based on field tests conducted by Engelhardt and Stall (1976). This value of E_p is reported for piles constructed either by vibrofloatation technique or vibro replacement method using vibroflot in Germany. However, the experience from the present test series indicate that for piles constructed with small hammer weighing between 125 kg to 500 kg the value of

E_p equal to 230kg/cm^2 to 500 kg/cm^2 may be adopted for design purposes.

Also the corrected soil modulus E'_s may be obtained from the equation 3.55.

$$E'_s = E_s \left[\frac{\sigma'_m}{10} \right]^{0.5} \qquad ...3.55$$

Having known, values of corrected pile and soil modulii (E'_p and E'_s). The load shared by pile and soil can be determined.

(g) Contribution of Applied Load to the Pile Capacity

As discussed in art. 3.5 , the applied load is shared by the granular pile and the soil in between the piles. Based on this the load shared by the granular piles and the soil is given by equations 3.21 and 3.22, and by equations 3.26 and 3.27 in terms of modular ratio m' and relative pile area (α).

It will be checked that the load shared by the surrounding soil (q_s) should always be less than the allowable soil pressure to avoid bearing capacity or excessive settlement failure.

(h) Increase in Effective Mean Normal Stress (σ_m)

Since the mean normal stress σ_m is increased due to the applied load q of which soil has shared a value equal to q_s, thus the increased mean normal stress σ'_m is calculated from the following expression:

$$\sigma'_m = 1/3\ (1+2K_o)\ q_s \qquad ...3.56$$

$$\text{Hence soil rigidity } I_r = \frac{E'_s}{2(1+\mu)(\sigma'_m \tan\emptyset)} \qquad ...3.57$$

and thus cavity expansion factor F'_q is again found out from Fig. 3.4 or Table 3.2 for this value of I_r and $\emptyset$.

(i) Increase in Limit Lateral Stress

$$\sigma'_{rL} = \sigma'_m F'_q \qquad ...3.58$$

(j) Increase in Ultimate Stress due to Applied Load

$$(q_{ult})' = K_p \sigma'_{rL} \qquad ...3.59$$

(k) Total Ultimate Stress

Therefore the total ultimate vertical stress of the soil pile system $(q_{ult})_1$ is found by summing equations 3.53 and 3.59 and is given by equation (3.60)

$$(q_{ult})_1 = (q_{ult}) + (q_{ult})' \qquad ...3.60$$

and the ultimate load capacity of a single granular pile is given by

$$Q_{ult} = A_p (q_{ult})_1 \qquad ...3.61$$

and

$$Q_{safe} = Q_{ult}/FS=3 \qquad ...3.62$$

(l) Safe Pile Capacity

Thus the total group capacity may be found from equation 3.63.

$$(Q_{allow}) \text{ group} = n (Q_{allow}) \qquad ...3.63$$

where,

n is the number of piles in a group.

The factor of safety is usually taken between 2-3 but not less than 2.

The value of K_p in equations 3.53 and 3.59 may be assumed equal to 6.

3.9 Determination of Soil Modulus E_s

Buisman (1950) suggested the use of the following expression, based on deep cone penetration resistance q_c, for the determination of soil modulus E_s in sand. This expression is related to determination of settlement of shallow foundations in sand:

$$E_s = 1.5\ q_c \qquad ...3.64$$

Extensive field experience with this approach has demonstrated that the actual observed settlements of foundations were almost twice as large as predicted ones (DE BEER 1945, 1948, and DE BEER and Martens 1957). Also later experiences indicated that the factor 1.5 is not a unique constant but varies with soil compressibility ranging from 1.5 to 3 (Schmertmann, 1970, 1975, DE BEER 1974, Trofimenkov 1974). These suggest that in an average

$$E_s = 3\ q_c \qquad ...3.65$$

where q_c is the static cone resistance

However, Vesic (1967) suggested an improvement over the Buisman expression for potential use in practice and is expressed as:

$$E_s = 2\ (1 + D^2_r)\ q_c \qquad ...3.66$$

where D_r is the relative density of the sand.

The ratio E_s/q_c varies quite well with the field experience.

In case the static cone penetration resistance data q_c is not available, the same may be changed from standard penetration test values N_{SPT} based on Sanglerat (1972), from Table 3.4

Table 3.4 Static Cone & Standard Penetration Resistance Ratios

Soil Type	Ratio q/N_{SPT}
Soft to medium clay	2
Silt or silty sand and loose sand	3
Dense sand	4
Gravelly sand	8

The elastic soil modulus may also be directly found from the expression proposed by Begemann (1974)

$$E_s = 40 + C(N-6) \text{ kg/cm}^2 \qquad ...3.67$$

If N_{SPT} is greater than 15

$$\text{and } E_s = C\ (N+6) \text{ kg/cm}^2 \qquad ...3.68$$

When N_{SPT} is less than 15

The value of C is taken as 3 for silt and sand however, for gravelly sand C is 12. It may also be found from the laboratory triaxial test using the expression

$$E_s = \frac{\sigma_1 - \sigma_3}{\delta L/L} \quad \text{kg/cm}^2 \qquad ...3.69$$

Within the elastic range, in the expression $(\sigma_1 - \sigma_3)$ is the deviator stress and $\delta L/L$ is the corresponding strain.

3.10 Ultimate Bearing Capacity in Cohesive Soil ($\emptyset$=0)

In the case of cohesive soil ($\emptyset$=0) and Poisson's ratio μ =0.5 as shown in equation 3.10, the cavity expansion factor F'_c is equal to $[1 + \ln (I_r)]$. where the rigidity index I_r as given by

equation 3.6 is modified for cohesive soils, and is given by equation 3.70

$$I_r = \frac{E_s}{3C_u} \qquad ...3.70$$

Therefore equation 3.7 which gives limit lateral stress may be re-written as:

$$\sigma_{rL} = \sigma_m + c_u (1+Ln\ I_r) \qquad ...3.71$$

In the case of cohesive soil, taking lateral earth pressure coefficient as unity, the effective mean normal stress, $_m$ will be equal to effective normal stress , thus the limit lateral stress σ_{rL} is given by equation 3.72

$$\sigma_{rL} = \sigma_v + [1+ Ln\ (I_r)]C_u \qquad ...3.72$$

or

$$\sigma_{rL} = \gamma' (L_c) + (1+Ln\ I_r)\ C_u \qquad ...3.73$$

where

γ' is the unit submerged weight of the clay, L_c is the critical pile length equal to 5 times the installed pile diameter (d).

For soft to stiff clays the rigidity index I_r varies from 10 to 300 (Table 3.1) and the corresponding value of $(1+L_n\ I_r)$ from Fig. 3.4 varies from 3.42 to 7. Thus for soft to medium stiff clays the value of $(1+l_n\ I_r)$ equal to 5 may safely be assigned. Thus equation 3.72 may be written as

$$\sigma_{rL} = \gamma' d+5C_u \qquad ...3.74$$

It may be recalled that the term ($\gamma' d$) in the above equation is the initial lateral stress on the pile ($\sigma' h' o$) which was taken as equal to ($2C_u$) by, Hughes, Withers and Green Wood (1975).

However Mori (1979) assigned a value equal to ($0.5\gamma' H$) where H was taken as 5m.

Therefore ultimate bearing capacity (q_{ult}) is given by:

$$q_{ult} = K_p [0.5 \gamma' d + 5 C_u] \quad ...3.75$$

If q_s is the load shared by the surrounding soil which may be found from equation 3.22, the increase in pile capacity due to applied load is given by:

$$q'_{ult} = K_p [q_s + 5 Cu] \quad ...3.76$$

Hence the total pile capacity (q_{ult}), is given by equation 3.75

$$(q_{ult})_1 = q_{ult} + q'_{ult} \quad ...3.77$$

3.11 Settlement Considerations (non cohesive soils)

The settlement of a footing placed on a loose-medium dense non-cohesive sub-soil stratum reinforced with three different conditions of partially penetrating granular piles have been considered in the following sections, viz,

(a) Sub-soil stratum reinforced with partially penetrating plain granular piles.

(b) Sub-soil stratum reinforced with partially penetrating individually skirted piles.

(c) Sub-soil stratum reinforced with partially penetrating, collectively skirted granular piles.

3.11.1 Plain Granular Piles

Settlement in a sub-soil stratum reinforced with partially penetrating plain granular piles is computed on the basis of equivalent

soil modulus of deformation of the composite mass. The following steps by step-procedure is proposed:

(a) The applied stress q at the footing base is assumed to be dispersed by 2:1 method Fig. 3.7 and the values of q_1, q_2, $q_3 \ldots q_n$ are found far each layer 1,2,3....n
The settlement ΔL in the reinforced sub-soil (FGTK) of thickness (L) equal to the pile length, and the settlement ΔH in the un-reinforced subsoil having a thickness H is computed separately. The sum of these two settlement gives the total settlement.

(b) The equivalent soil modulus of deformation (E_{eq}) of the composit mass is found from equation 3.78

$$E_{eq} = \alpha E_p + (1-\alpha) E_s \qquad ..3.78$$

(c) The coefficient of volume compressibilities (m_v, m_{v2}, m_{v3}.. m_{vn}) for different layers having thicknesses $h_1, h_2, h_3 \ldots h_n$ is found out from the values of E_{eq1}, $E_{eq2} \ldots E_{eqn}$.

(d) Thus the settlement of the reinforced stratum is given by Equation 3.79

$$\Delta L = q_1, m_v, h_1 + q_2\, m_{v2}\, h_2 + q_3\, mv_3\, h_3 + \ldots q_n\, m_{vn} \qquad ...3.79$$

(e) Similarly the settlement (ΔH) of the layers for the un-reinforced depth H KROZ is found out from the equation 3.80

$$\Delta H = q_1\, m_{v1}\, H_1 + q_2\, m_{v2}\, H_2 + q_3\, m_{v3}\, H_3 \ldots q_n\, m_{vn}\, H_n \qquad ...3.80$$

where

$q_1\, q_2 \ldots q_n$ and m_{v1}, $m_{v2} \ldots m_{vn}$ and $H_1\, H_2 \ldots H_n$ are the stresses

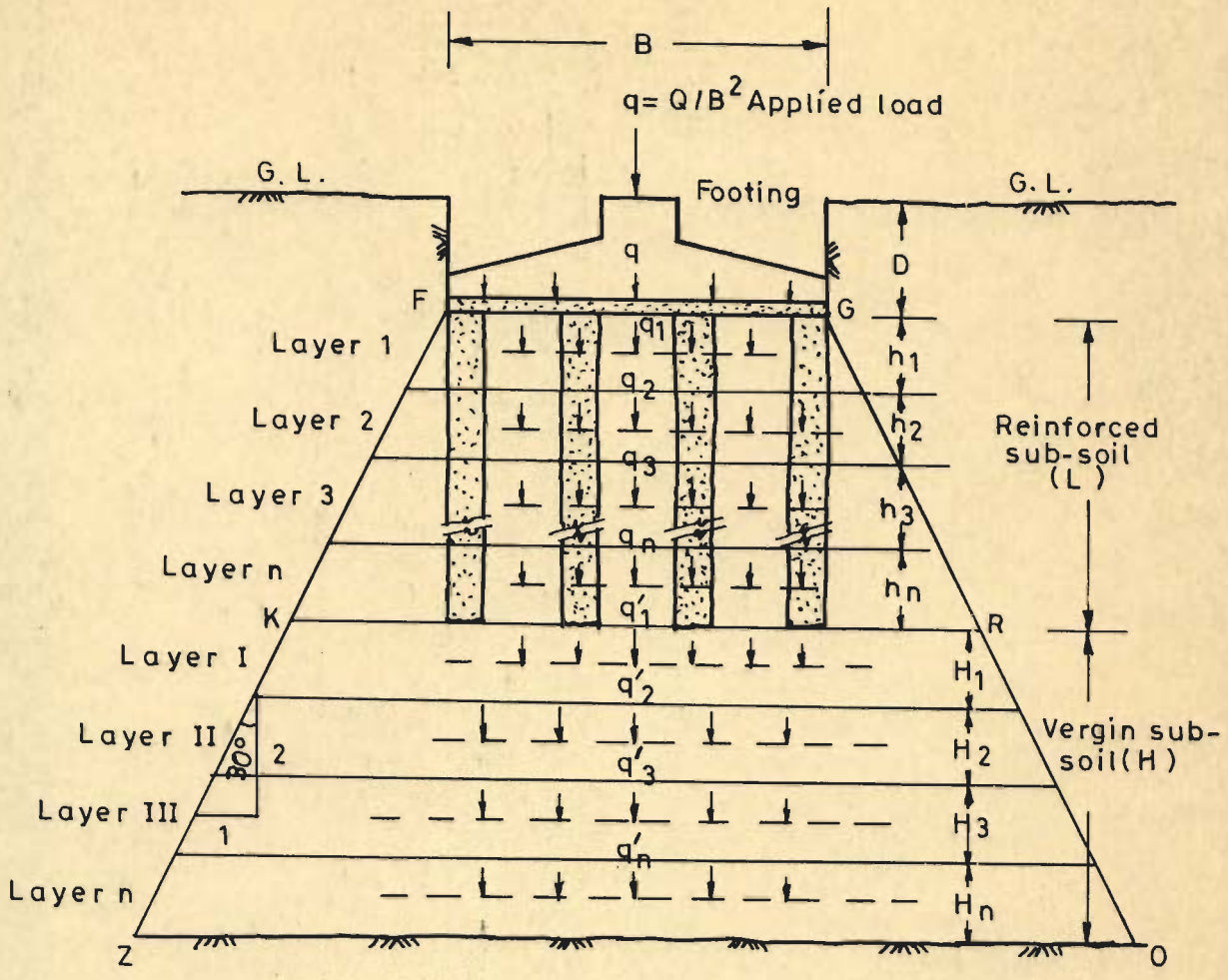


FIG.3.7. SETTLEMENT OF A SUB-SOIL STRATUM REINFORCED WITH PARTIALLY PENETRATION PLAIN GRANULAR PILES

at mid layers , coefficient of volume compressibilities for the layers I II III.....n having thicknesses as $H_1, H_2\ H_3 \ldots\ldots H_n$

(f) Thus the total settlement(s) under the footing is found from equation 3.81

$$S = \Delta L + \Delta H \qquad \ldots 3.81$$

If the granular piles are resting on a hard stratum, the value of ΔH is taken as zero.

3.11.2. Individually Skirted Granular Piles

Settlement of sub-soil stratum reinforced with partially penetrating, individually skirted granular piles is considered in the following manner:

(a) The applied stress q at the footing base F G is assumed to act on a equivalent footing placed on the plane F'' G'' having a depth equal to two third of the depth of the individual skirt (D_s), Fig. 3.8

(b) From the level F'' G'' at a depth equal to 2/3 D_s, the applied load q is assumed to disperse by 2:1 method (at an angle of 30° from the vertical)

(c) It is further assumed that the settlement in layer (F'' G'' G_1F_1) is very small and negligible.

(d) The stresses at mid layers $q_1, q_2, q_3 \ldots q_n$ of layers 1,2,3... n in the reinforced depth F_1G_1 T'K' and unreinforced layers I, II, III.... , q_1, q_2, $q_3 \ldots q_n$ are computed.

(e) Subsequently the coefficient of volume compressibilities m_{v1}, $m_{v2} \ldots m_{vn}$ and m_{v1} m_{v2} $m_{v3} \ldots m_{vn}$ are computed from the corresponding values of equivalent soil modulus of deformations (E_{eq})

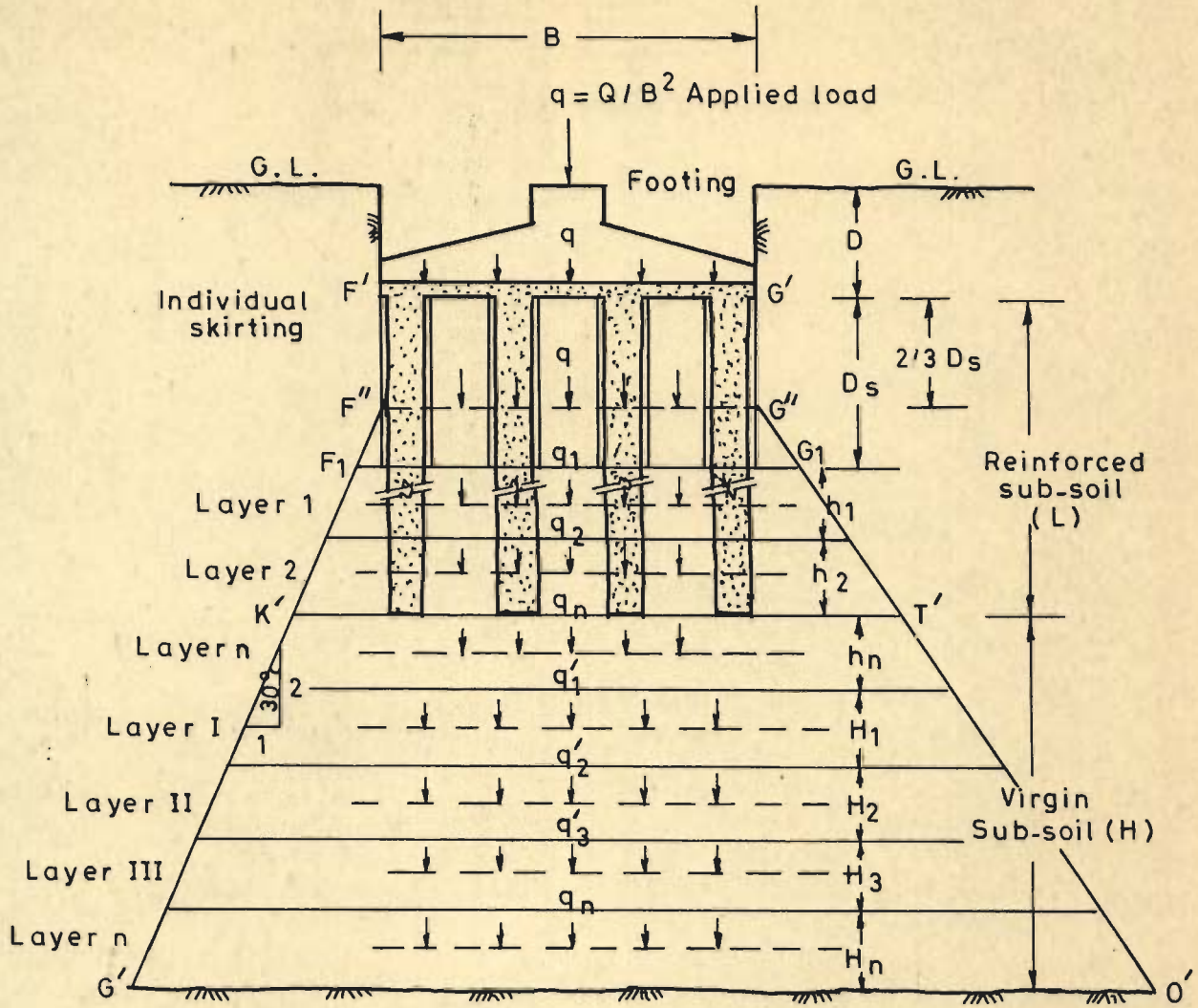


FIG.3.8. SETTLEMENT OF A SUB-SOIL STRATUM RE'NFORCED WITH PARTIALLY PENETRATING INDIVIDUALLY SKIRTED GRANULAR PILES

and E_s of the various layers of reinforced and un-reinforced stratum.

(f) Thus the total settlement of the footing is given by equation 3.82

$$S = \Delta L + \Delta H.. \qquad ...3.82$$

If the granular piles are resting on hard soil stratum, the value of ΔH is negligible.

3.11.3 Collectively Skirted Granular Piles

The settlement of a footing placed on a weak sub soil reinforced with collectively skirted granular piles is considered as sum total of the following settlements as indicated in Fig. 3.9

(a) Settlement (Δs) in skirted reinforced soil plug (MNGF) having thickness equal to D_s.

(b) Settlement (ΔL) in Reinforced soil of the thickness (L)

(c) Settlement (ΔH) in un-reinforced virgin soil of thickness (H)

3.11.3(a) Settlement in Skirted Reinforced Soil Plug (Δs)

The confinement provided by the rigid skirt, provided around the reinforced soil plug (MNGF) Fig. 3.9 will result in preventing the lateral displacement of reinforced soil plug under the applied load, q . Thus the vertical displacement will be reduced significantly. From the theory of elasticity and equations of displacements in three principal planes may be written as:

$$\epsilon_1 = \frac{\sigma_1}{E_{eq}} - \mu \left(\frac{\sigma_2 + \sigma_3}{E_{eq}}\right) \qquad ...3.83.$$

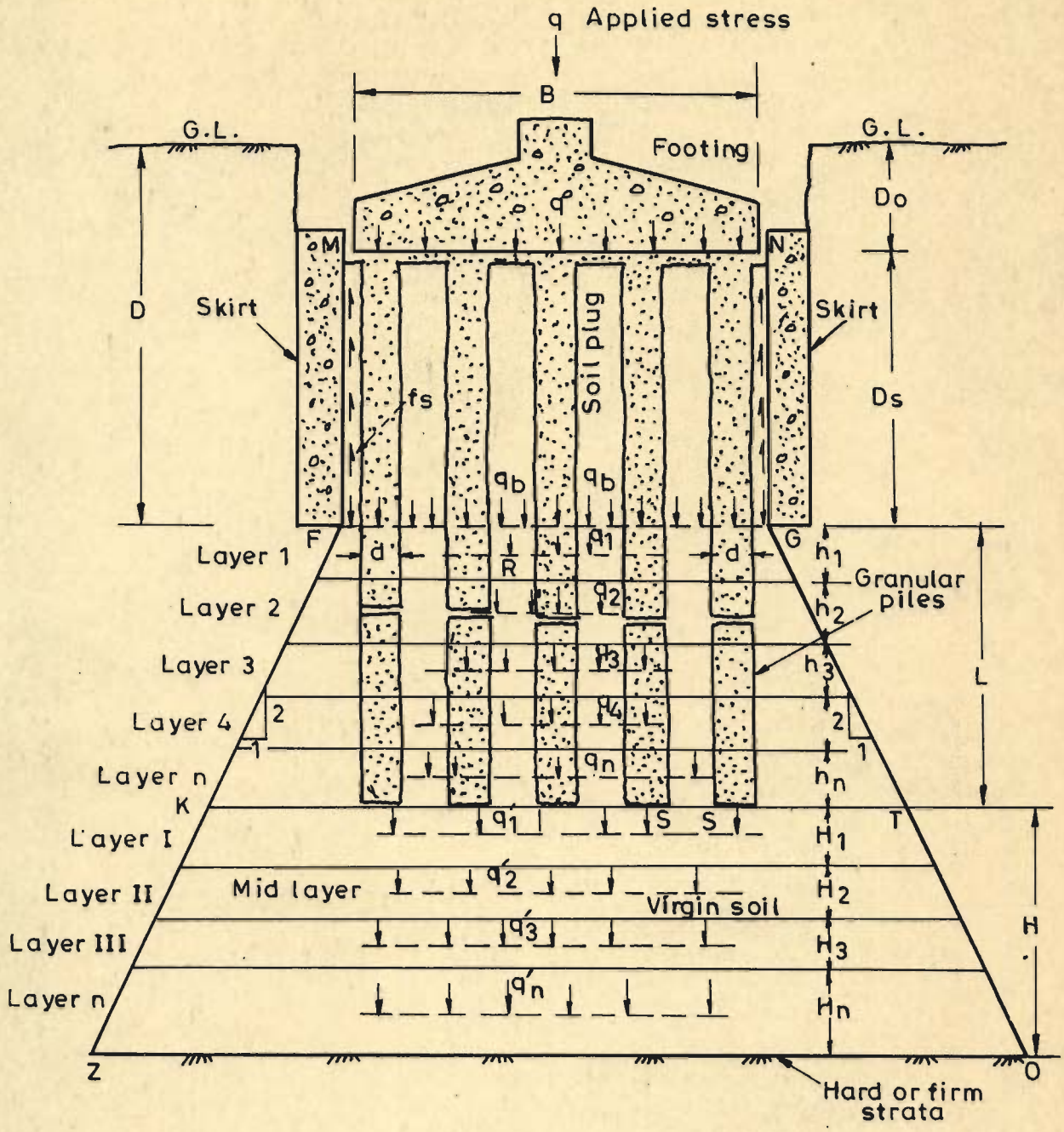


FIG.3.9. SETTLEMENT COMPUTATION OF A SKIRTED GRANULAR PILES FOUNDATION

$$\epsilon_2 = \frac{\sigma_2}{E_{eq}} - \mu \left(\frac{\sigma_3 + \sigma_1}{E_{eq}} \right. \qquad \ldots 3.84$$

$$\epsilon_3 = \frac{\sigma_3}{E_{eq}} - \mu \left(\frac{\sigma_3 + \sigma_1}{E_{eq}} \right) \qquad \ldots 3.85$$

$\epsilon_1, \epsilon_2, \epsilon_3$ are the strains and σ_1, σ_2, σ_3 are the stresses in three directions of the principal planes, μ is the Poissons ratio and E_{eq} is the equivalent modulus of deformation of the reinforced soil plug. Since the lateral displacement is completely checked due to the confinement provided by the skirt thus the displacements

$$\epsilon_2 = \epsilon_3 = 0 \qquad \ldots 3.86$$

Therefore equation 3.85 can be written as

$$\sigma_3 = \left(\frac{\mu}{1-\mu} \right) \sigma_1 = K_o \left(\sigma_1 \right) \qquad \ldots 3.87$$

Substituting for σ_3 in equation 3.83 from equation 3.87 and putting $\sigma_1 = q$ which is the applied load, the confined compression ϵ is given by

$$\epsilon = \frac{q}{E_{eq}} \left[1 - 2\mu K_o \right] \qquad \ldots 3.88$$

where

ϵ is the confined compression and $\left(\frac{q}{E_{eq}} \right)$ is the unconfined compression representing settlement in skirted reinforced soil plug and a plain footing respectively. Thus the settlement in the skirted reinforced soil plug MNGF will be reduced by a factor $(1 - 2\mu k_o)$,

It can be seen that in the case of loose sand ($\emptyset = 30^o$)

$K_o = 1 - \sin(1.2\,\emptyset) = 0.4122$

and $\mu = 0.2963$.

The factor $(1-2\mu\ k_o) = 0.7559$ and for dense sand

$(\emptyset = 36^o)$

$K_o = 0.3154, \mu = 0.2398$

The factor $(1-2\mu\ k_o) = 0.8437$.

This demonstrates that in loose to medium dense non cohesive soil reinforced with collectively skirted granular piles, the reduction in settlement due to presence of a rigid skirt is likely to vary between 75 to 84 per cent. Thus the corresponding reduction factors (RF) for loose to medium dense sand vary from 0.25 to 0.16.

The settlement (Δs) in the soil plug MNGF reinforced with collectively skirted granular piles is found from equation 3.89

$$\Delta s = (RF) \quad \Delta s' \qquad ...3.89$$

where,

RF is the reduction factor due to confinement provided by the skirt. The value of reduction factor varies between 0.16 to 0.25 as stated earlier. The term (Δs) is the settlement of the reinforced soil plug MNGF without any skirting. For computing the value of (Δs), the applied load q is assumed to disperse at an angle of 30° from the vertical at the base of the footing MN, Fig. 3.9, and the stratum up to a depth equal to the depth of the skirt (D_s) is computed by dividing is in different layers. Thus the settlement (Δs) is the sum total of the settlements in different layers upto the base of the skirt FG and is expressed by equation 3.90

$$(\Delta s) = q_{s1}\ h_{s1}\ m_{vsi} + q_{s2}\ h_{s2}\ m_{vs2} + q_{s3}\ h_{s3}\ m_{vs3} + \ldots q_{sn}\ h_{sn}\ m_{vsn} \qquad .. 3.90$$

where,

$q_{s1}, q_{s2}, q_{s3} \ldots q_{sn}$ are the stresses in the middle of the individual

layers, h_{s1}, h_{s2} h_{s3}...h_{sn} are the corresponding thicknesses of these layer and m_{vs1}, m_{vs2}...m_{vsn} are their respective coefficient of volume decrease for the composit mass of the individual layers which are obtained from equation 3.78. Adopting the value of reduction factor as 0.25 conservatively, the equation 3.89 is written as

$$\Delta s = 1/4\ (\Delta s') \qquad ...3.91$$

where,

(Δs) is found from equation 3.91

3.11.3(b) Settlement in the Reinforced Soil (ΔL)

The computation of settlement in reinforced soil below base FG, the soil plug having a thickness equal to L, Fig. 3.9 is made in the following steps.

(a) The applied stress q at the footing base shall be reduced to a lower stress q_b acting at the base of the soil plug FG. The stress q_b shall be found from equation 3.37.

(b) The stress (q_b) shall be distributed by 2:1 method from below the skirt and thus stresses q_1, q_2, q_3... and q_n acting at the middle of layers, 1,2,3...n having thicknesses h_1,h_2,h_3...h_n shall be calculated.

(c) The equivalent modulus of deformation (E_{eq}) for each layer shall be calculated from equation 3.70 and hence the corresponding coefficient of volume compressibilities (m_{v1}), (m_{v2}).....$(m_v)_n$ shalll be found out.

Thus the settlement (ΔL) of the reinforced layer is sum of settlement of different layers and is expressed by equation 3.92

$$\Delta L = q_1 h_1 (m_v)_1 + q_2 h_2 (m_v)_2 + q_3 h_3 (m_v)_3 \ldots + q_n h_n (m_v)_n \ldots 3.92$$

3.11.3(c) Settlement in unreinforced Soil (ΔH)

The stresses q'_1, q'_2, $q'_3 \ldots q'_n$ at the middle of the layers I,II, III....n having thicknesses H_1 $H_2 \ldots H_n$ shall be found as earlier. The corresponding values of m_{v1}, m_{v2}, $m_{v3} \ldots$ of different layers shall be found either by E_s values or from the soil samples in the laboratory or from any field penetration tests. Thus the settlement of the strata having thickness H, is the sum of the settlements in each layers and is expressed by equation 3.93

$$\Delta H = q'_1, H_1 m_{v1} + q'_2 H_2 m_{v2} + \ldots q'_n H'_n m_{vn} \ldots 3.93$$

Therefore the total settlement of a skirted granular pile foundation in weak sub soil stratum is expressed by adding equations 3.91 to 3.93

$$S = \Delta S + \Delta L + \Delta H \ldots \qquad \ldots 3.94$$

It may be pointed out that the significant reduction in settlement due to granular pile reinforcement occurs only if the piles are closely spaced (spacing kept as less than or equal to 5 times the installed pile diameter) and only if the piles are installed for the full depth of the consolidating layer i.e. upto depth =(OZ Fig. 3.7.

3.12 Ultimate Uplift Capacity of Skirted Granular Piles With a Central Unit Pile

The ultimate uplift capacity of skirted granular pile with a central unit pile (Load Bearing Element) Fig. 4.30 is considered as the sum of ultimate uplift resistance offered individually by (a) Group of granular piles (b) the friction between the reinforced

soil-plug and the concrete skirt interface and (c) the friction between the unit pile /LBE and the surrounding soil. Analytical solutions for these are proposed in the following pages.

3.12.1 Uplift Resistance of a Single/Group of Granular Piles

3.12.1 (a) Assumptions

The following assumptions are made in the proposed analysis:

1. The granular pile is considered to be less stiff as compared to a concrete pile. The pile fails to transfer the applied stresses from the toe of the pile to the pile top.

2. The granular pile is assumed to derive the resistance against uplift force from the restraint provided by the surrounding soil around the critical height, H_c, Fig. 3.10 .

3. The granular pile is considered to fail in bulging at the instant when the lateral stress σ_r induced in the pile body due to applied stress q at the pile toe attains its limit value σ_{rL} and q_{ult} respectively.

4. The height of bulging or critical pile height 'H_c" is limited to 5 pile diameter, Fig. 3.10.

5. The bulging failure mode of a single granular pile is considered analogous to the expansion of a cylinderical cavity having the diameter (d) and the height equal to H_c Fig. 3.10.

6. The bottom portion of the pile equal to critical pile height is considered to bulge due to a uniform induced lateral stress σ_r in a homogeneous, infinite, isotropic non-cohesive subsoil deposit due to the gradual increase in the applied uplift stress q, and consequently the σ_r in

the pile body. The cylinderical zone around the bulged pile having a radius, R_u, Fig. 3.10, and 3.3, will pass into a state of plastic equilibrium. Beyond this zone of plastic equilibrium of radius R_p, the soil is considered to remain in elactic equilibrium condition.

7. The ultimate uplift load Q_u applied at the pile toe is considered to be resisted by the weight of the pile W_p and the load required to provide restraint against bulging of the pile.

8. The unit friction along the granular pile shaft is not considered as there is no relative movement between the granular pile and the surrounding soil.

3.12.1 (b) Ultimate uplift Load (Q_o)

The ratio of the radii of the plastic zone and the cylinderical cavity (R_p/R_u), the reduced rigidity index I_{rr}, the rigidity index I_r and the lateral limit stress σ_r for the uplift capacity of the granular pile is defined in a similar manner as equations 3.1 through 3.8 for cohesive and non-cohesive soils. Thus the ultimate uplift stress q_{ult} is defined as:

$$q_{ult} = K_p \ \sigma_{rL} = K_p \left[C_u F'_c + \sigma_m F'_q \right] \qquad \ldots 3.95$$

where,

K_p is the coefficient which is assigned a value equal to 6 based on equation 2.30, c_u is the undrained shear strength, F'_c and F'_q are the cavity expansion factors dependent on (Ø) and the rigidity index I_r defined by equations 3.5 and 3.6. These may be obtained from Table 3.2 or Fig. 3.4.

The elastic modulus of the soil E_s is corrected in accordance with art. 3.4 as:

$$E'_s = E_s\left[\frac{\sigma_m}{10}\right]^{0.5} \qquad \ldots 3.96$$

and the rigidity index is obtained from equation 3.97

$$I_r = \frac{E'_s}{2(1+\mu)(C_u+\sigma_m \tan \emptyset)} \qquad \ldots 3.97$$

Thus the ultimate uplift stress q_{ult} for cohesive soil is obtained from the following equation:

$$q_{ult} = K_p[\sigma_v + C_u F'_c] \qquad \ldots 3.98$$

or

$$\sigma_{rL} = K_p\left[\sigma_v + C_u (1+L_n I_r)\right]$$

and the uplift load Q_o is given by equation 3.100

$$Q_o = K_p A_p [\sigma_v + C_u (1+L_n I_r)] \qquad \ldots 3.100$$

where,

σ_v is the overburden pressure at a plane xx Fig. 3.10 at a depth equal to effective depth (H_e), q is the applied stress on the pile and A_p is the cross section area of the pile.

In the case of non cohesive soil the uplift load Q_o is given by the equation, 3.101

$$Q_o = K_p(\sigma_m F'_q) A_p \qquad \ldots 3.101$$

where,

$K_p = \tan^2 (45+\emptyset'/2)$, $\emptyset'$ is the angle of shearing resistance of the material constituting the pile.

If W_p is the weight of a single granular pile then the ultimate uplift load Q_u is obtained from equation 3.102

$$Q_u = W_p + K_p\ (\sigma_m F'_q) A_p \qquad \ldots 3.102$$

Assuming the group efficiency of unity the ultimate uplift load of a group of pile (Q_g) is given by:

$$Q_g = nW_p + nK_p\ (\sigma_m F'_q)\ A_p \qquad \ldots 3.103$$

where,

n is the number of piles in a group.

3.12.2 Resistance Due to Friction Between Reinforced Soil Plug and the Skirt Interface.

The total resistance offered due to friction between reinforced soil plug and the concrete skirt interface (Q_s) is given by the following equation

$$Q_s = f_s \cdot A_{sk} \qquad \ldots 3.104$$

where, f_s is the unit skin friction and A_{sk} is the surface area of the interface of the skirt.

$$f_s = 0.5 \gamma D_s K_u \tan\delta \qquad \ldots 3.105$$

where

γ is the bulk unit weight of the soil plug and δ is the angle of friction between the reinforced soil plug and the concrete skirt interface, K_u is uplift coefficient, $\tan\delta$ is coefficient of friction between skirt and the soil.

Meyerhof (1973) recommended the value of uplift coefficient K_u depending upon the angle of shearing resistance ϕ Fig. 3.11 The various forces acting on skirt and the unit pile are shown in Fig. 3.12.

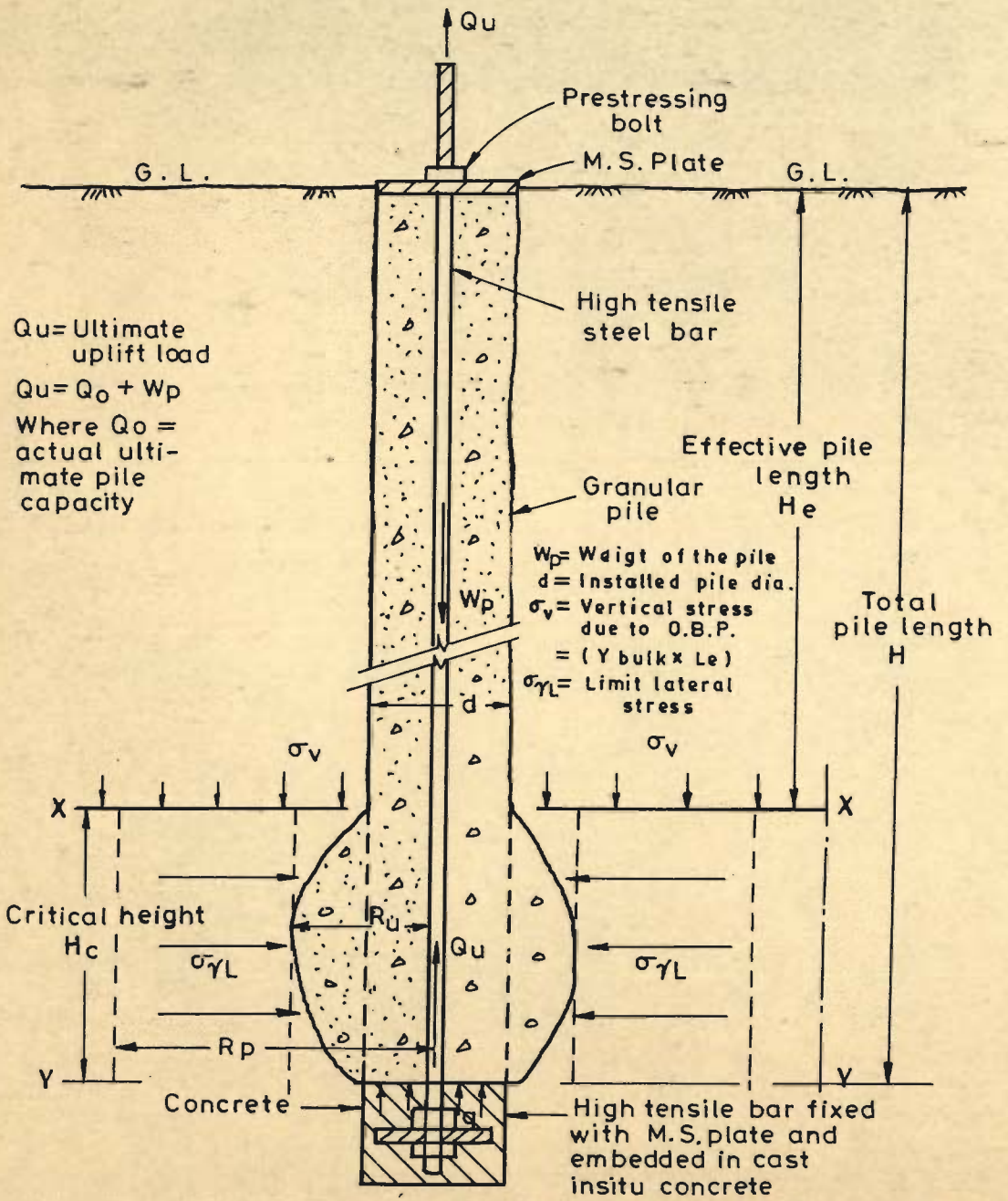


FIG. 3.10- BULGING FAILURE MODE OF A SINGLE GRANULAR PILE SHOWING VARIOUS STRESSES AT ULTIMATE UPLIFT LOAD

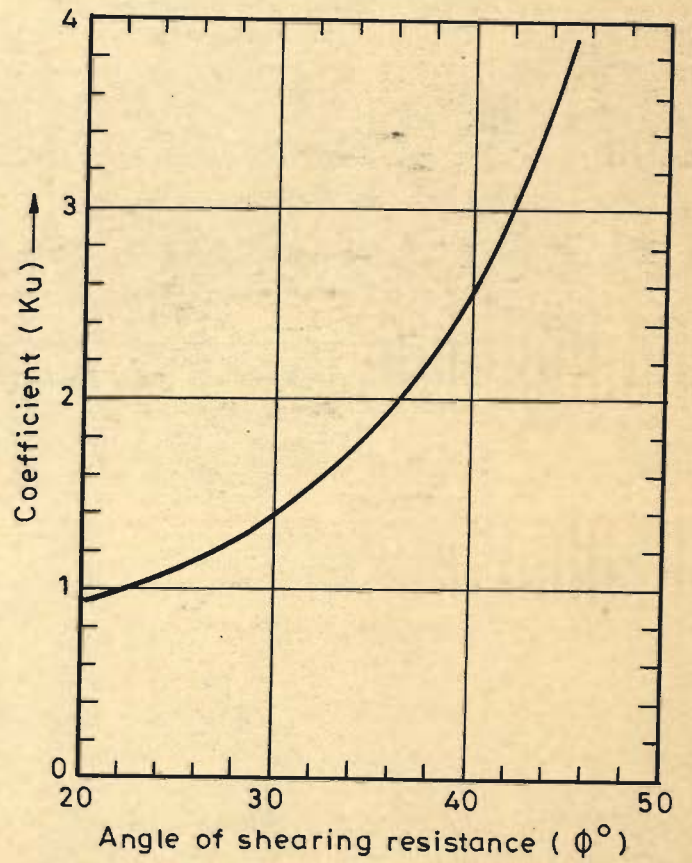
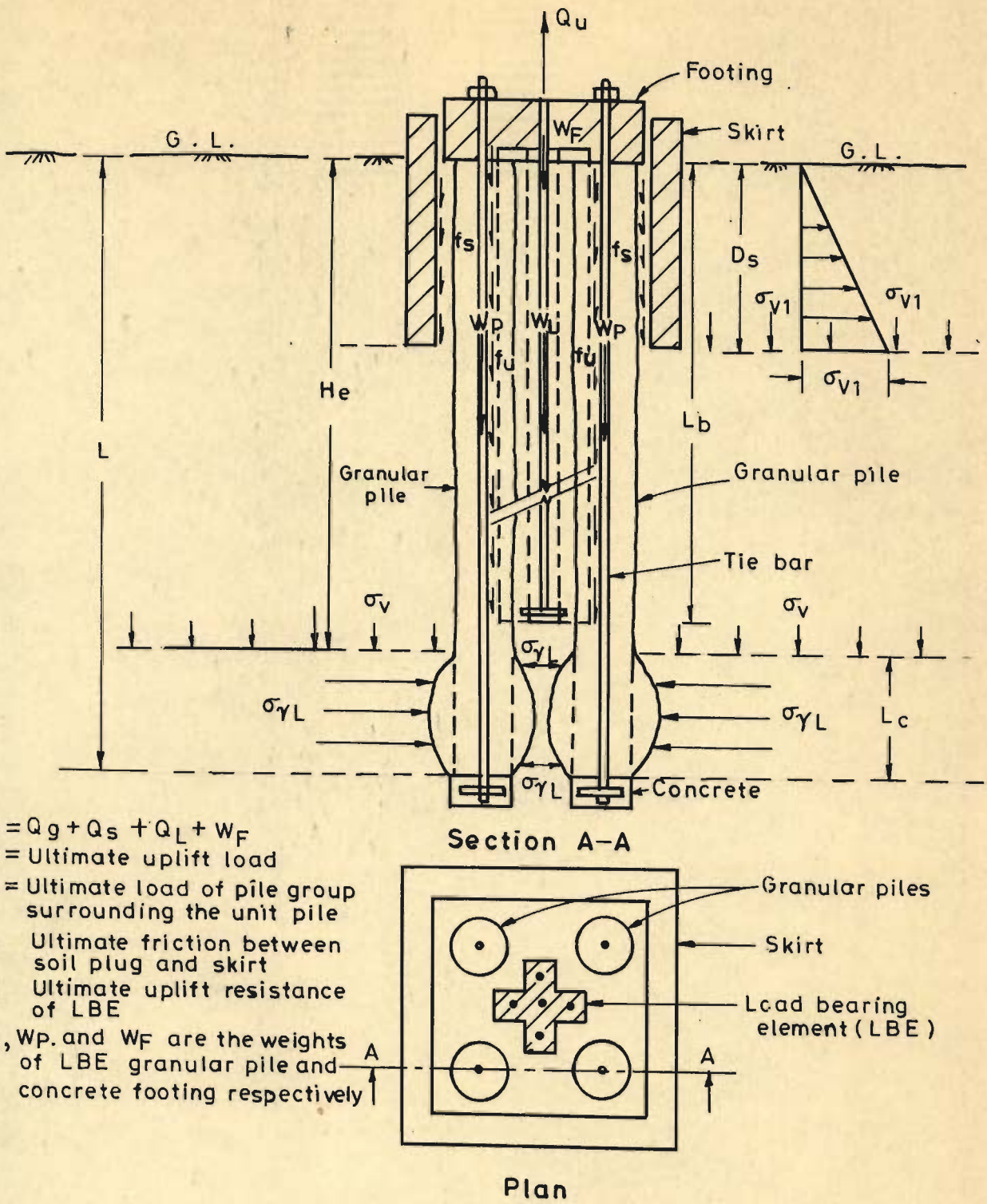


FIG.3.11-RELATION BETWEEN (ϕ) & (Ku)
(AFTER MEYERHOF 1973)



$Q_u = Q_g + Q_s + Q_L + W_F$

Q_u = Ultimate uplift load

Q_g = Ultimate load of pile group surrounding the unit pile

Q_s Ultimate friction between soil plug and skirt

Q_L Ultimate uplift resistance of LBE

W_L, W_p and W_F are the weights of LBE granular pile and concrete footing respectively

FIG.3.12. BULGING FAILURE MODE OF A SKIRTED GRANULAR PILE FOUNDATION SHOWING DIFFERENT STRESSES AT ULTIMATE UPLIFT LOAD

In the case of soil plug reinforced by granular piles, the initial density increases and influences the angle of shearing resistance $\emptyset$. Thus in such a case δ equal to (1.2 $\emptyset$) may be assumed.

Therefore, the unit skin friction, f_s is given by the following equation:

$$f_s = 0.5\gamma\ D_s\ K_u \tan(1.2\emptyset) \qquad \ldots 3.106$$

or

$$Q_s = 0.5\gamma\ D_s\ K_u \tan(1.2\emptyset).\ A_{sk} \qquad \ldots 3.107$$

3.12.3 Ultimate Uplift Capacity of Unit Pile (Q_L)

In the case of unit pile cast in non cohesive sub soil deposit, the applied pull load is resisted by the weight of the unit pile W_L, Fig. 3.13 and the frictional force between the unit pile shaft of area A_{sb} and the surrounding reinforced sub-soil around the pile. The ultimate uplift load Q_L is thus given by the following equation:

$$Q_L = 0.5\gamma\ L_b\ K_u \tan\delta.A_{sb} + W_L \qquad \ldots 3.108$$

where,

L_b is the effective length, and A_{sb} is the shaft area of the unit pile, δ is the angle of friction taken as (1.2 $\emptyset$).

Thus,

$$Q_L = 0.5\gamma L_b\ K_u \tan(1.2\emptyset)\ A_{sb} + W_L \qquad \ldots 3.109$$

Thus the total ultimate uplift capacity of the skirted granular pile foundation Q_u is given by adding equations.

$$Q_u = Q_g + Q_s + Q_L + W_F \qquad \ldots 3.110$$

where,

W_F is the weight of the footing Fig. 3.11.

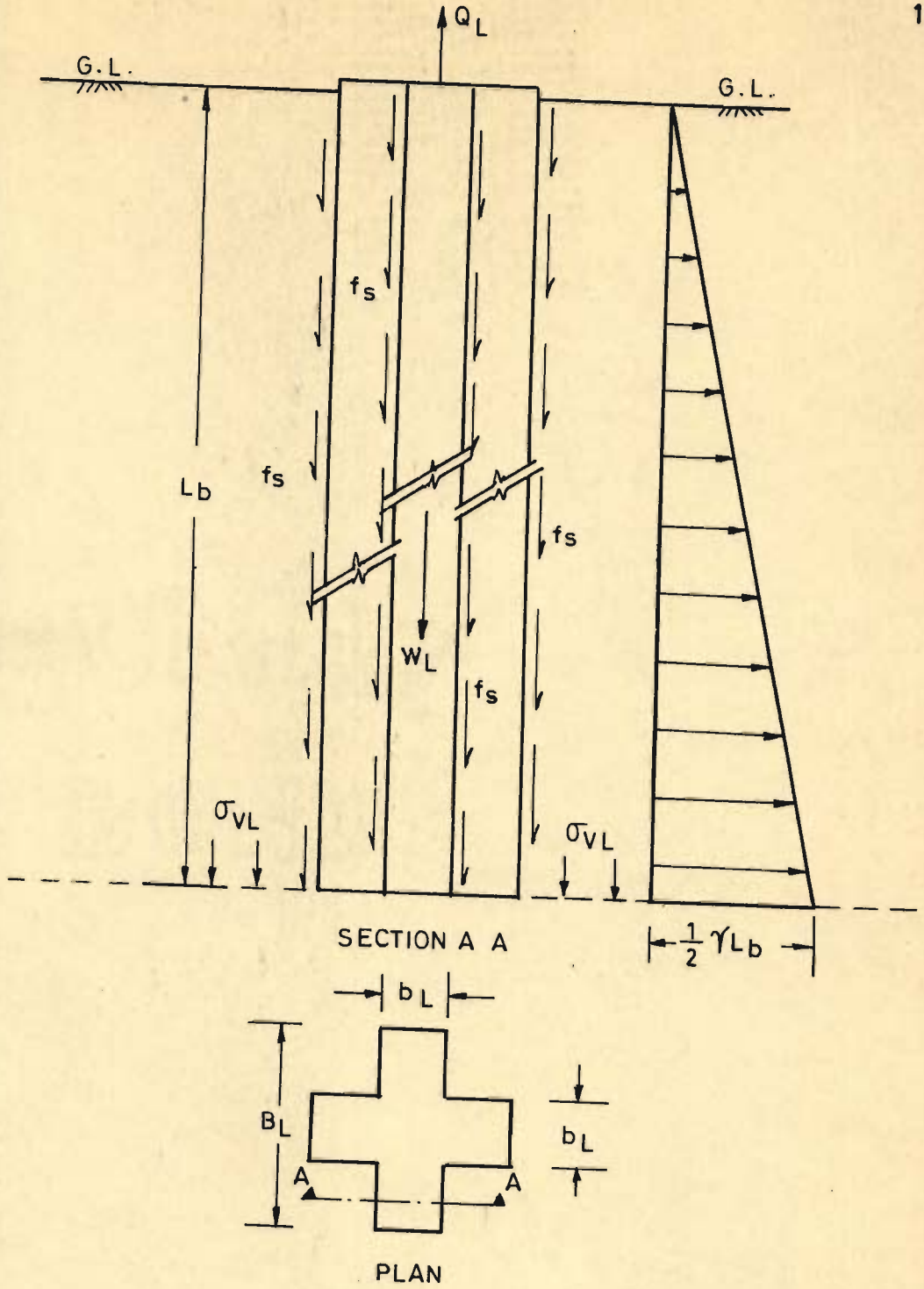


FIG.3.13. UPLIFT RESISTANCE OF A UNIT PILE (LBE)

or

$$Q_u = nK_p\sigma_m F'_q A_p + 0.5\gamma D_s K_u \tan(1.2\,\phi) A_{sk}$$

$$+ 0.5\ L_b K_u \tan(1.2\,\phi) A_{sb} + nW_p + W_L + W_F \quad \ldots 3.111$$

or

$$Q_u = n K_p\sigma_m F'_q A_p + 0.5\gamma K_u \tan(1.2\,\phi)[D_s A_{sk} + L_b A_{sb}]$$

$$+ nW_p + W_L + W_F \quad \ldots 3.112$$

CHAPTER IV

EXPERIMENTATION

4.1 Introduction

The behaviour of skirted granular piles under load is influenced by a several factors. Some of these factors being diameter and length of pile, the material used for construction, method of construction and placement of piles i.e. individual/grounp action. Besides these factors, the use of skirt around piles individually or around group collectively is also likely to influence the overall behaviour of pile group. Skirting may have a large number of variables e.g. material, shape, depth, rigidity etc. These parameters are large and are beyond the scope of a single investigation. In view of this only a few important parameters have been chosen for study in the present investigation.

4.2 Justification for Field Study

Experiments may be carried out either in the laboratory on small scale model piles in artifically prepared homogeneous soil deposits or in the field where prototype granular piles could be constructed and load tested.

The laboratory testing has the advantages of better control over various parameters which may be studied. However, it has a serious limitation of only qualitative results only which are obtained and also scale effects which are difficult to estimate precisely.

Though field tests are more time consuming and uneconomical, however, in view of realistic assessment of various parameters and the overall behaviour of granular piles field studies are better. Keeping this in view all the tests were carried out at different sites in

both, cohesive and non-cohesive deposits in the field only at five different sites.

4.3 Parameters Studied

The behaviour of granular piles is influenced by a large number of parameters. Only the following parameters have been chosen for study in the present investigation.

4.3.1 Soil Stratum

The field testing of granular piles were limited to non cohesive sub-soil stratum (c=0) and tests were conducted at three different sites, besides two more sites consisting of clayey silt and soft clay deposits.

4.3.2 Granular Pile Diameter and Length

For the purpose of in-situ load testing 20cm and 30cm diameter granular piles having 3.5m and 8m lengths were used. The diameter of granular piles were found to be 25 cm and 35 cm after installation. However, on other two sites having cohesive sub soil stratum diameter of piles were 60 and 55cm and length 9 and 11m only.

4.3.3 Pile Material

Stone aggregates 20-30mm sizes were used to construct the piles. Each layer of the stone aggregate was mixed with 20-25 per cent of locally available medium sand. Thus the granular pile material was stone-aggregate sand mixture well compacted with heavy hammers.

4.3.4 Pile Cap

In all the cases whether the granular piles were single or in groups, reinforced cement concrete footing cast on the granular pile/ pile group was used. The footing was 25cm thick and the plan dimensions were kept in such a way that it projected 10cm from the edge of the piles.

4.3.5 Skirt

Tests on piles with and without skirt were carried out using different skirting materials. The skirt was made of reinforced cement concrete, m.s. pipes, timber piles, brick panels. Depending on the shape of the installed granular piles, the skirt in collective skirting was square, circular or rectangular shapes. The depth of the skirt was in no case less than half the width of the footing/pile cap.

4.3.6 Material used for Skirt

Reinforced cement concrete skirts cast in situ in a prepared trench was generally used for testing. The cement concrete skirt was either reinforced with mild steel bars or 1.5mm thick mild steel sheets. Besides these skirts were also constructed by sand witching prefabricated brick panels of adequate sizes in green cement concrete. Another type of skirting made out by driving 10cm by 10cm timber piles in a row as near as possible around the footing/pile cap was also used as an alternative to cast insitu reinforced cement concrete skirting. The timber pile skirting was made rigid by helding all the pile top by suitable clamping or cast in situ r.c.c. beam at the foundation level. Prefabricated GI pipe 10cm in dia., two 5cm dia. pipe units welded together with vertical slits were also constructed and used to provide skirt in the test.

4.4 Sub-Soil Investigations

Detailed sub-soil investigations were carried out on all the five sites. The field tests, besides boring and sampling included standard penetration tests (SPT) dynamic cone penetration on test (DCPT) and static cone penetration tests and plate load tests (PLT).

4.4.1 Boring Sampling and Standard Penetration Tests

A 15cm diameter borehole was advanced in accordance with IS:1892-1979 (first revision) using shell and auger boring method. Standard penetration tests were carried out at every 1.5m interval as per IS: 2131-1963. Undisturbed samples were collected using thin wall samplers (IS:2132-1972) at alternate 1.5m depths. The sampling and SPT were carried out alternatively at every 1.5m intervals. The depth of the boreholes varied from 7 to 30m at different sites

4.4.2 Dynamic Cone Penetration Tests

A mild steel cone having 62.5mm dia. and 60° apex angle was penetrated by impact of a 65kg hammer falling through a height of 75cm in accordance with IS:6968 (Pt.II)-1977 (first revision). The cone was having a push fit arrangement with the standard 'A' drill rod. After the end of each test, at a location, the mild steel cone was left into the ground and the drill rod was withdrawn. The number of blows of every 30cm of penetration was counted and is taken as N_c value. Dynamic cone penetration tests were limited to 12-15m depth.

4.4.3 Static Cone Penetration Tests

A 35.7mm diameter (10 sq.cm area) cone was pushed at the rate of 1cm per sec. upto a length of 7cm. The static cone resistance was measured through hydraulic pressure gauges mounted with the sounding machine. The static cone resistance was expressed in kg/cm^2 and is measured at every 50cm intervals IS: 4968 (Part III)-1977 (first revision)

4.4.4 Plate Load Test

Plate loading tests were also carried out at 0.5m depth, below

average ground level using 60cm by 60cm plate with a criss cross surface in accordance with IS:1888 (second revision) - 1980.

4.4.5 Laboratory Tests

Soil classification and grain size analysis were carried out in accordance with IS: 1498-1979. The insitu density, natural moisture content liquid limit and plasticity index were also determined in the laboratory in accordance with Indian standard procedures.

Isotropically consolidated drained triaxial (CID) tests on soil samples having sand as main constituents (SC,SM and SP) were carried out. Mohrs strength envelopes were drawn with the test results of minimum of three samples in a test. Thus the value of cohesion (c') and angle of shearing resistance (Ø') were obtained.

4.5 Method of Construction

4.5.1 Construction of Granular Piles for Compression Tests

The construction of granular piles call for making uniform and straight borehole by resorting to displacement or no displacement methods. When the water table is sufficiently low, auger boring is generally used especially when the depth of the pile is small. In the present case for 20cm diameter, 3.5m deep piles (25cm after installation), a 20cm diameter spiral auger was used to make the borehole utilising manual labour. The borehole was extended upto 3.5m by adding extension rods to the auger. After reaching the desired depth the borehole was thoroughly cleaned manually by using specially made tools having a thin plate welded to a m.s. rod. The plate was bent at 90 degrees in the middle to enable collection of loose soil from the borehole. In the case of

30cm dia. (35cm after installation) and 8m deep piles, the borehole was made by using 30cm diameter a spiral auger upto the water table. Below the water table and upto 8m, non displacement method of boring, using a 30cm dia. mild steel casing and 15cm diameter shell with a suitable cutting shoe attached at the bottom was used. The borehole was filled with 5 per cent Na-bentonite slurry after withdrawing the casing fully.

Granular piles were cast using 20-30mm stone aggregates and 20-25 per cent of locally available Solani river sand having a uniformity coefficient of 2. The stone aggregate was placed in the borehole in layers of 30cm followed by 5 to 6cm of sand layer. A cast iron hammer having 125 kg weight and diameter less than the diameter of the bore hole was used to compact the stone aggregate and sand mixture. The hammer was operated through a tripod made out of GI pipe (10cm dia.) and a power winch Figs 4.1 and 4.2. The close up view of the hammer while lowering into the borehole is shown in Fig. 4.3. Thus the hammer was operated inside the borehole with a fall of 75cm, compacting the sand stone aggregate mixture. During the construction of the pile, two skilled labour, one on the power winch and the other on the bore hole to guide the hammer were able to do the job. In addition 4-6 labourer were required for collecting measuring and charging the stone-sand mixture into the borehole. Thus the filling and compaction of the layers was continued till the full length of the pile cast.

During the course of the impact by the hammer the sand fills into the voids of the stone-aggregates followed by lateral and downward displacement of the charged material till the full compaction is achieved. Thus the lateral displacement of the material helps in compaction of the surrounding soil resulting into higher shear

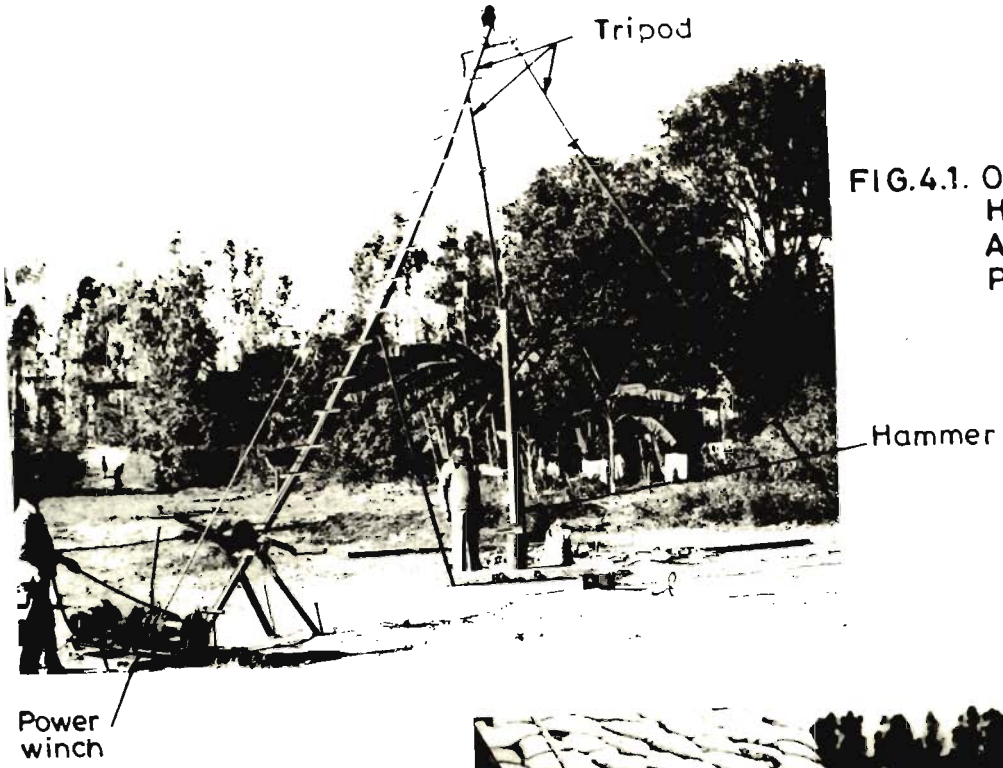


FIG.4.1. OPERATION OF HAMMER THROUGH A TRIPOD AND POWER WINCH

FIG.4.2. DETAILS OF THE POWER WINCH

FIG.4.3. CLOSE-UP VIEW OF THE HAMMER WITH A CENTRAL HOLE

strength. Also the installed granular pile has a larger diameter which was generally 20 to 25 per cent more than the initial diameter of the borehole. Various stages of installation procedure of granular piles are shown in Fig. 4.4 To have a uniform compaction all through the length of the pile, check tests by set measurements were carried out atleast at 4 different stages. In all the piles a 25mm set for 10 number of blows was decided. This was adhered to. The fall of the hammer was kept as 75cm. Few piles were exposed immediately after construction (Fig. 4.5 and 4.6) which confirmed the uniformity in construction and increase in granular pile diameter after installation. Similar procedure was adopted for construction of granular piles in groups of 2,3 and 4 piles. The piles in group were constructed at a spacing of three pile diameters. Fig. 4.7 shows a two pile group individually skirted with mild steel pipes.

4.5.2 Construction of Granular Piles for Tensile Loads

For granular piles under tensile load (pull out load test), 3.5m deep borehole was made with a 20cm diameter auger using manual labour. Sinking of the borehole and the cleaning was done as in the case of compression piles. Before casting of the granular piles, a 15cm thick 1:2:4 cement concrete mix was placed in the borehole followed by lowering of the tie rod 16mm dia. In the lower end of the mild steel rod, a mild steel plate (18cm dia., 2.5cm thick) which was held rigidily by providing nuts at the top and the bottom of the plate. To make it more strong, the top and bottom nuts were welded with the rod and the plate. Subsequently an other layer of 15cm thick cement concrete was placed and compaction was done by rodding, Fig. 4.8(a). The tie rod with mild steel plate was left in the borehole for 24 hours for curing. This was followed by placing of stone aggregates and sand mixtures

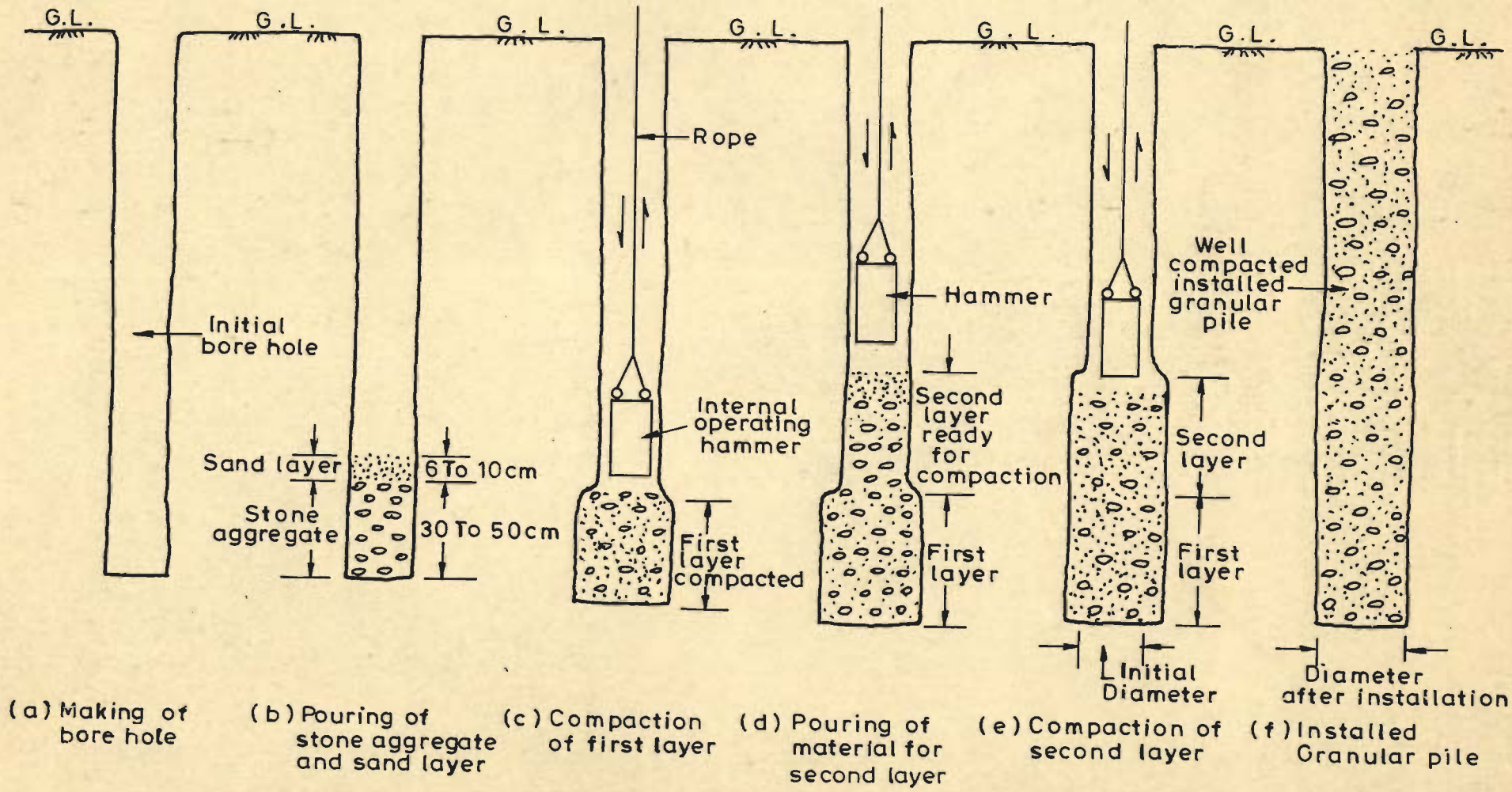


FIG.4.4. GRANULAR PILE INSTALLATION METHOD USING INDEGINOUS KNOW-HOW

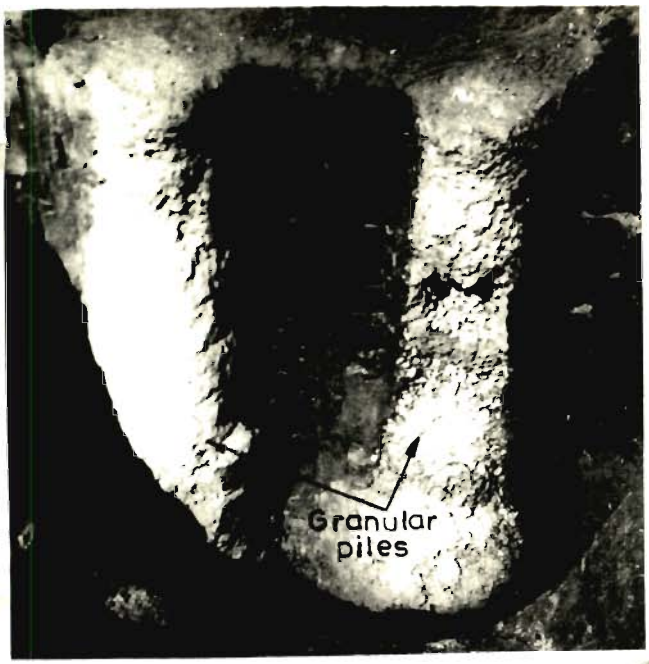
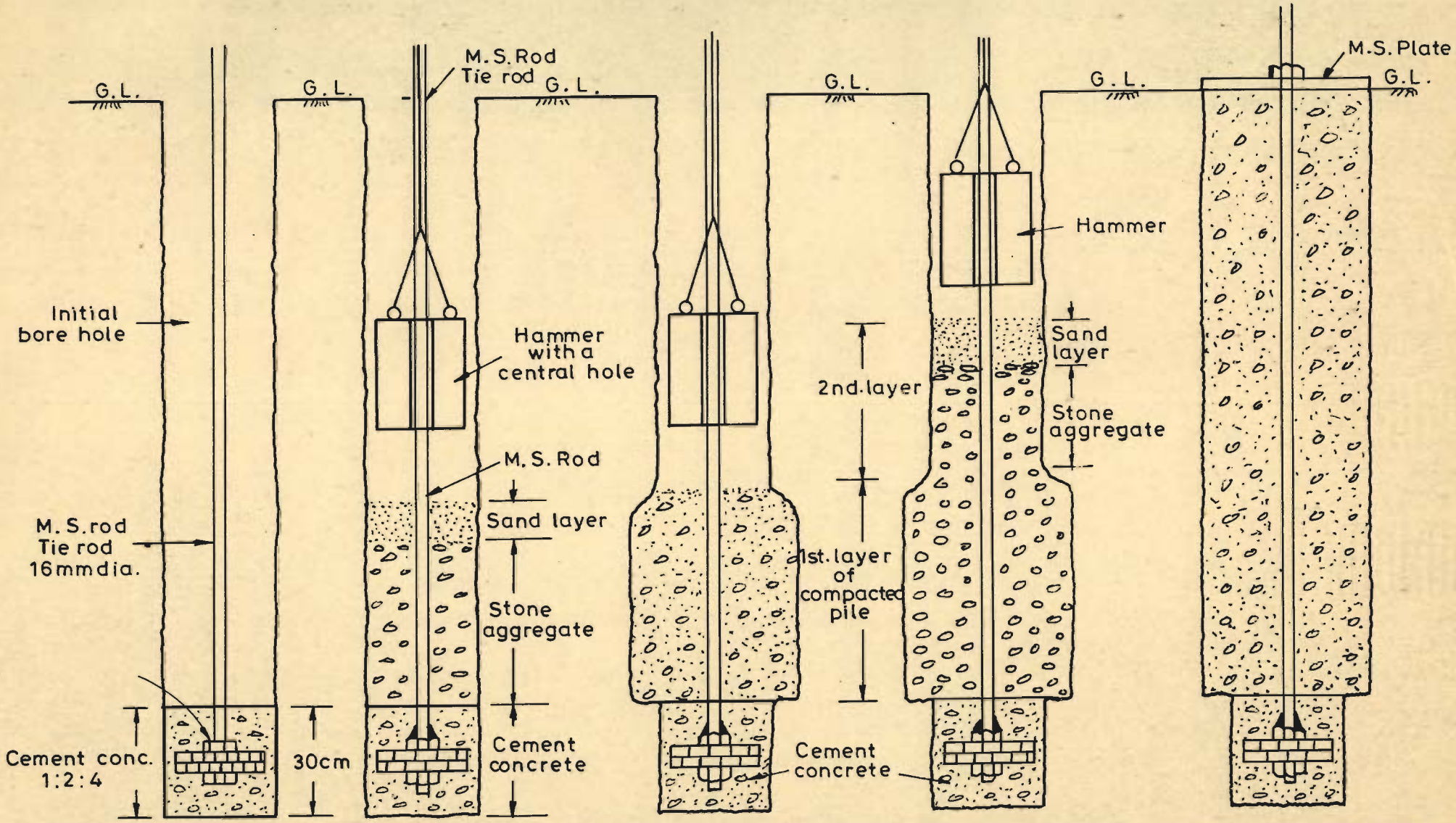


FIG. 4.5. GROUP OF TWO, PLAIN GRANULAR PILES EXHUMED AFTER LOAD TESTING

FIG. 4.6. GROUP OF TWO, PLAIN GRANULAR PILES EXHUMED BEFORE LOAD TESTING

FIG. 4.7. GROUP OF TWO, SKIRTED GRANULAR PILES EXHUMED AFTER LOAD TESTING



(a) Boring & installing the tie rod fixing in cement concrete

(b) Pouring of the 1st. layer of stone aggregate sand layer

(c) Compaction of the 1st. layer of sand stone aggregate mixture

(d) Compaction of 2nd. layer of the granular material

(e) Installed granular pile

FIG.4.8.INSTALLATION PROCEDURE OF GRANULAR PILE FOR TENSILE LOAD

in layer and compacted by internal operating hammer as in the case of compression piles. The hammer used for compaction had a hole in the case to allow the passage for the tie rod. A schematic diagram of different stages of construction is shown in Figs 4.8b through 4.8e. The procedure was followed to cast single and groups of 2,3,4 plain granular piles. When the full depth of the pile installed, a mild steel plate with a central hole (diameter larger than the tie rod) was placed on the top surface of the granular pile and a nut was secrewed tightly with a view to prestress the granular pile (Fig 4.8e). Groups of 4 plain granular piles is shown in Figs 4.9 and 4.10 before and after installation. Few piles were exhumed after pull out tests to check the uniformity in construction. Fig. 4.11 shows one such pile which confirmed the uniform compaction and good quality of construction procedure.

4.5.3 Construction of Reinforced Cement Concrete Skirt (reinforcing with mild steel rods)

In accordance with the size of the pile groups, the lay-out for the skirt trench was made on the levelled ground surface. The inner side of the trench was kept uniformly 10cm away from the edge of the granular pile. The width of the trench was kept as 20cm. The depth was kept as half of the width of the group plus 20cm. The trench was made with a specially fabricated cutting tool welded at the end of a 40mm dia. mild steel rod Fig. 4.12. The loose material (earth) from the trench was carefully removed with the help of another tool shown in Fig. 4.13. The procedure of making the trench is shown in Fig. 4.14. The skirts having larger widths and depth more than 1.5m and upto 6m can be made by using grabs as in the case of diaphragm walls.

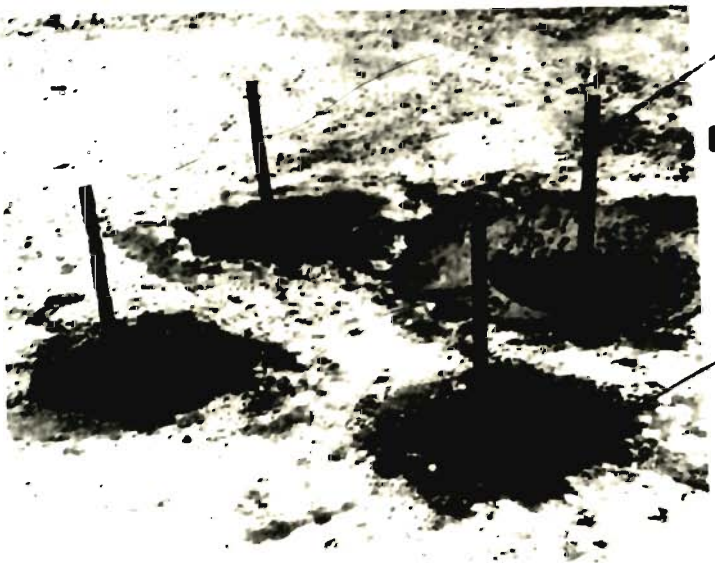


FIG.4.9. GROUP OF 4 GRANULAR PILES BEFORE INSTALLATION

FIG.4.11. SINGLE GRANULAR PILE AFTER INSTALLATION

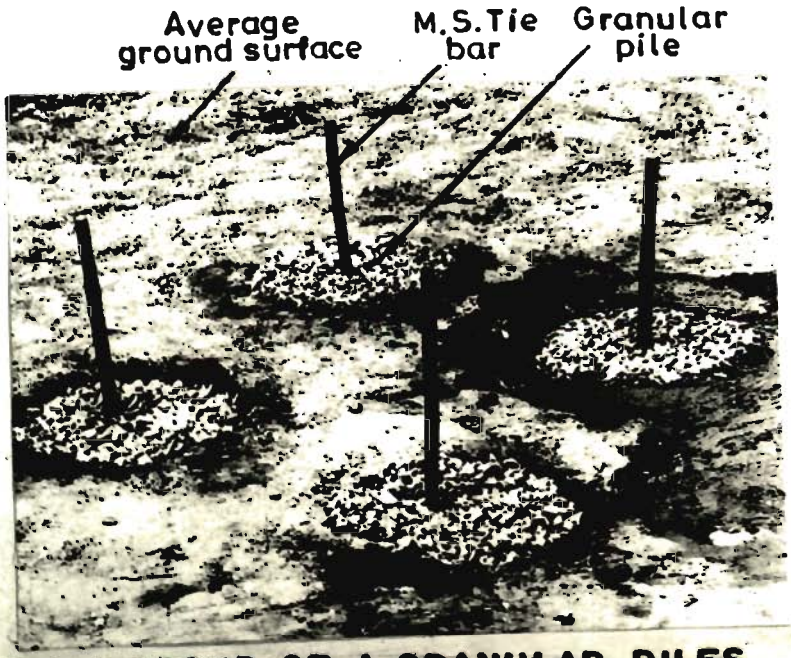


FIG.4.10. GROUP OF 4 GRANULAR PILES AFTER INSTALLATION

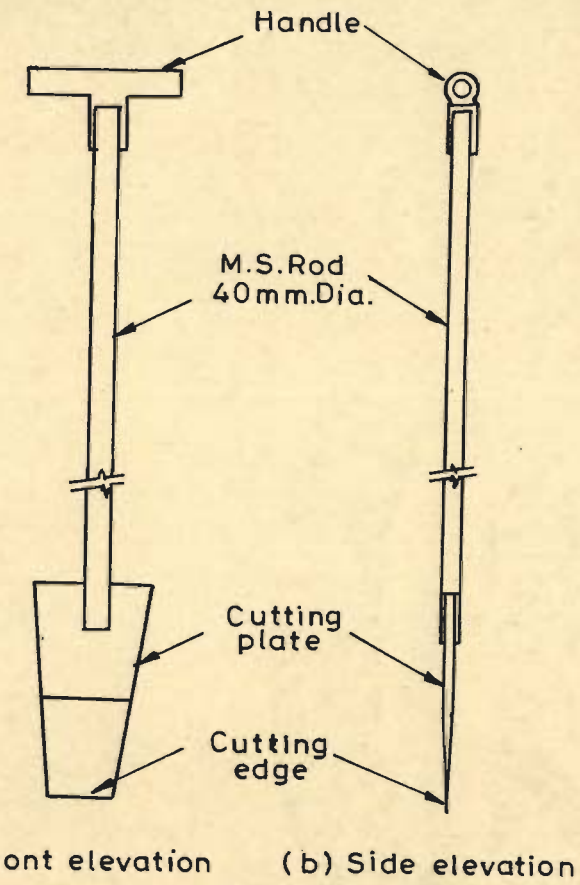


FIG.4.12. DETAILS OF THE CUTTING TOOL

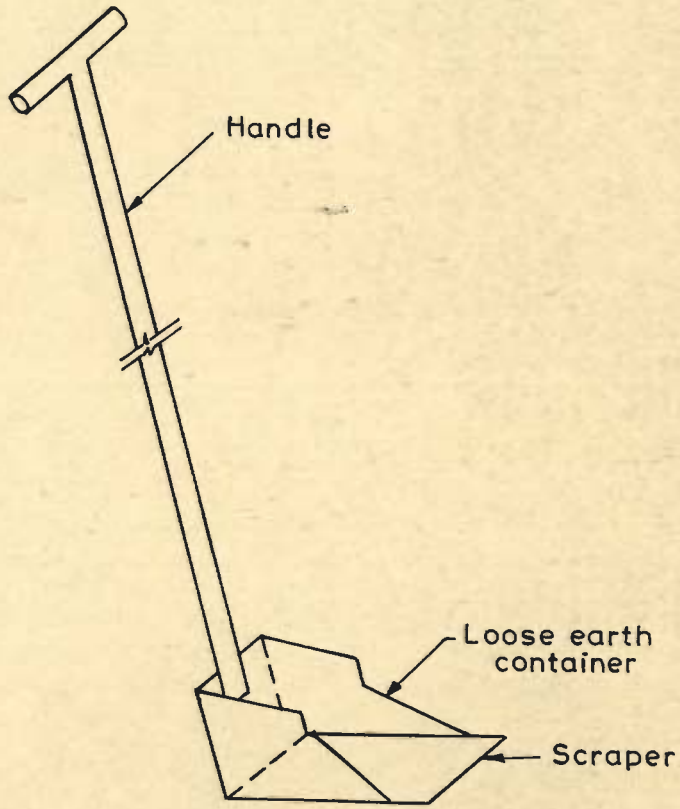


FIG.4.13. TOOL FOR REMOVING LOOSE EARTH FROM THE TRENCH

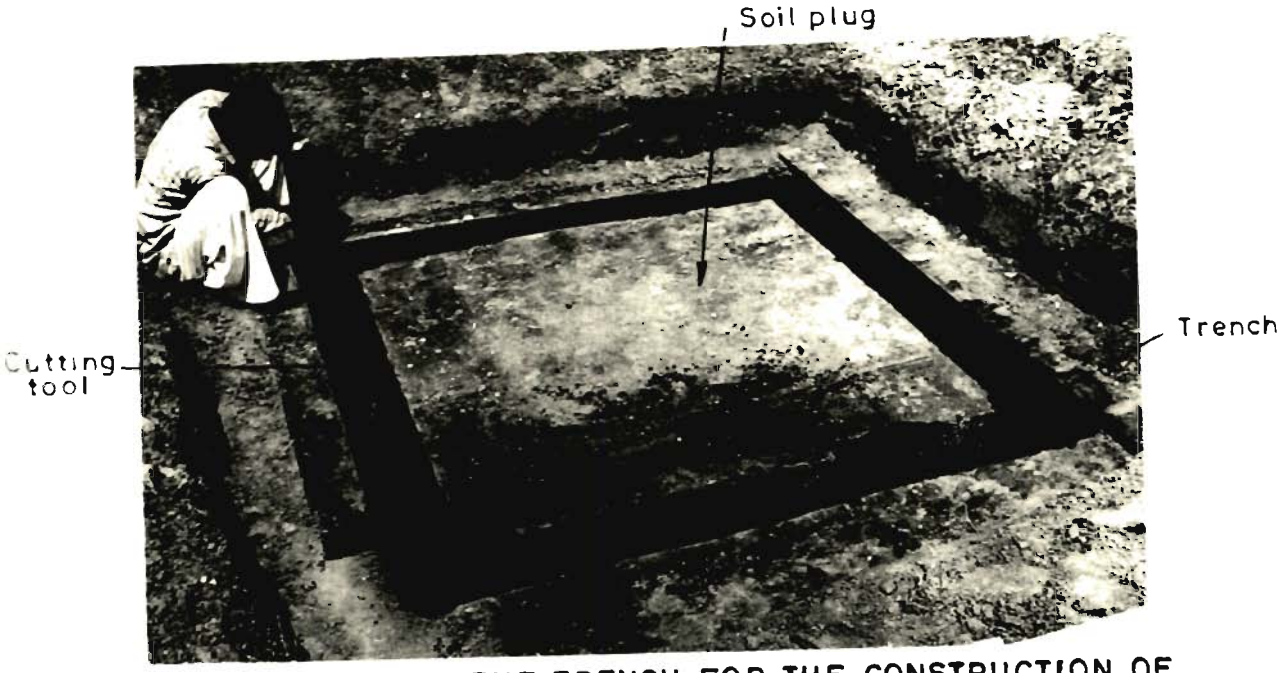


FIG.4.14. EXCAVATING THE TRENCH FOR THE CONSTRUCTION OF THE SKIRT

FIG.4.15. CONCRETING THE TRENCH AFTER LOWERING THE REINFORCEMENT CAGE FOR THE R.C.C. SKIRT

After the skirt trench is ready, the mild steel reinforcement cage was lowered and concreted. The process of placing the cement concrete after lowering of the reinforced cage and the completed skirt is shown in Figs. 4.15 and 4.16.

4.5.4 Reinforced Cement Concrete Skirt (reinforcing with 1.5mm thick m.s. sheet)

In this case the m.s. rod reinforcement was replaced by 1.5mm thick m.s. sheet with the depth equal to depth of the skirt. This was done with a view to study the feasibility of replacing m.s. reinforcement. The mild steel sheet was bent to the required shape square or circular and was revetted at the end, after providing a overlap of 20cm. The m.s. sheet was then lowered in the skirt and was placed centrally in the trench. This was followed by concreting and compaction described earlier.

4.5.5 Prefabricated Brick-pannel Skirting

In this case prefabricated brick pannels - 1m long and 50cm deep developed for roofing units Fig. 4.17 (Verma 1975) was as an economical alternative to reinforced cement concrete skirt, and the skirting trench was prepared.

Four prefabricated brick pannels 1m long, 5m wide and 7cm thick were lowered in the skirting trench. The horizontal reinforcements protuding from the joints of the brick in the panel (Fig. 4.18) was connected by placing a vertical rod at the corners and tied with binding wire. In addition to this a horizontal m.s. rod was made to pass through the hooks of all the four brick pannels at the foundation level, connecting all the pannels together. The in and outside space between the brick pannel and the vertical wall of the skirt trench and also the corners were filled with

FIG.4.16.R.C.C.SKIRT BEFORE THE CONSTRUCTION OF THE FOOTING ON SOIL PLUG

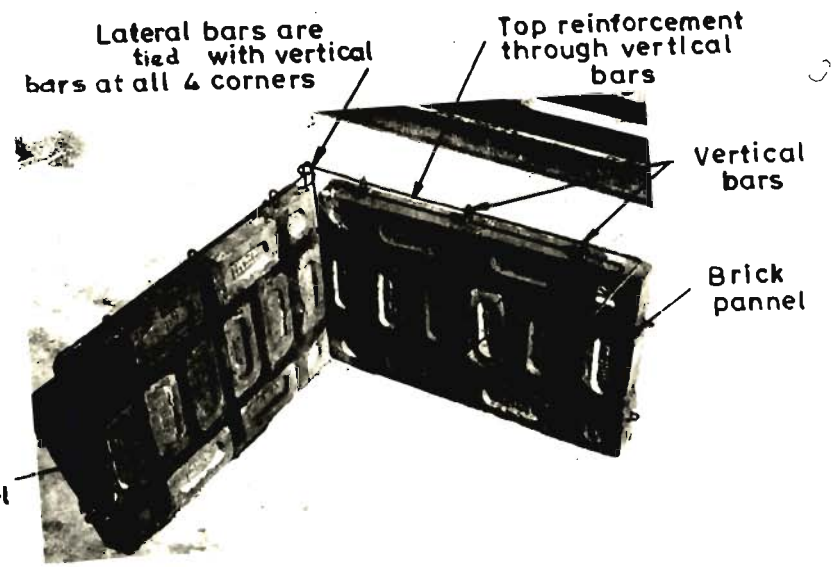


FIG.4.17. A VIEW OF THE PRE-FABRICATED BRICK PANNELS DEVELOPED FOR ROOFING UNITS ASSEMBLED ON THE GROUND

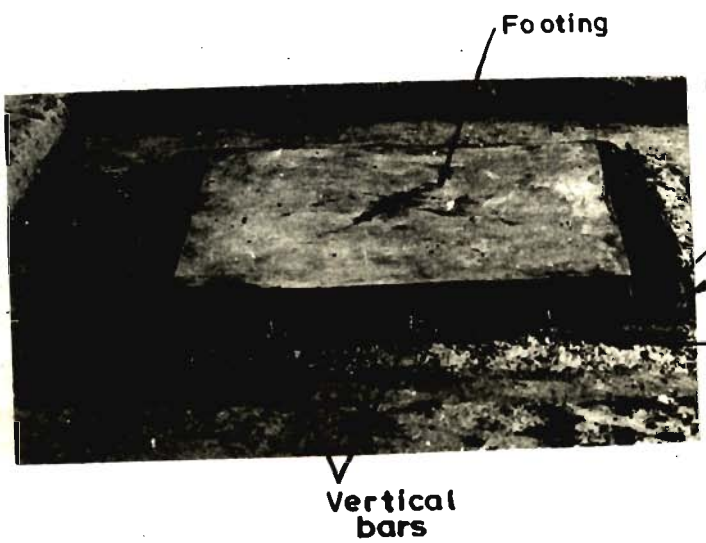


FIG.4.18.BRICK PANNEL SKIRTING SAND WICHED BETWEEN CONCRETE

1:2:4 (M-150) cement concrete mix and compacted, Fig. 4.18, shows the concreted brick pannel in the skirting trench. This was followed by finishing the top of the skirt by cement concrete and casting of the group of granular piles, Fig. 4.19. On the top of the granular pile group, the concrete footing is cast, Fig. 4.20.

4.5.6 Prefabricated Pipe Unit Skirting (Retrievable type of skirting)

For temporary structures such as oil drilling rig foundations, retrievable type of skirting is more economical as it can be re-used at other sites. In this type of skirting, several pipe units were prefabricated. One unit consisted of one 10 or 15cm dia. G.I. or m.s. pipes of 1m length and two pipes having 5 to 8cm dia. and 0.8m length. The two smaller dia. pipes were having a vertical slit 3mm wide, Fig. 4.21. The two pipes were welded with the larger dia. pipe in a straight line. The smaller pipes for the corner units are welded at right angles. This type of skirting avoids the construction trench. The pipe unit skirting is checked on ground surface before driving Fig. 4.22. The pipe units are then driven into the ground by using SPT hammer and the tripod winch (Fig. 4.23). After the first unit is driven, the second unit is placed in such a way that the slit of this unit slides in the smaller pipe of the previously driven unit to enable making a continuous wall, Fig. 4.24. A partially driven pipe units are shown in Fig. 4.25 and a complete view of the pipe unit skirting with a concrete footing cast on the soil plug is shown in Fig. 4.26. These units are retrievable one by one with the help of a bolt passing through the larger pipe in the unit and pulling it up with the help of a power winch.

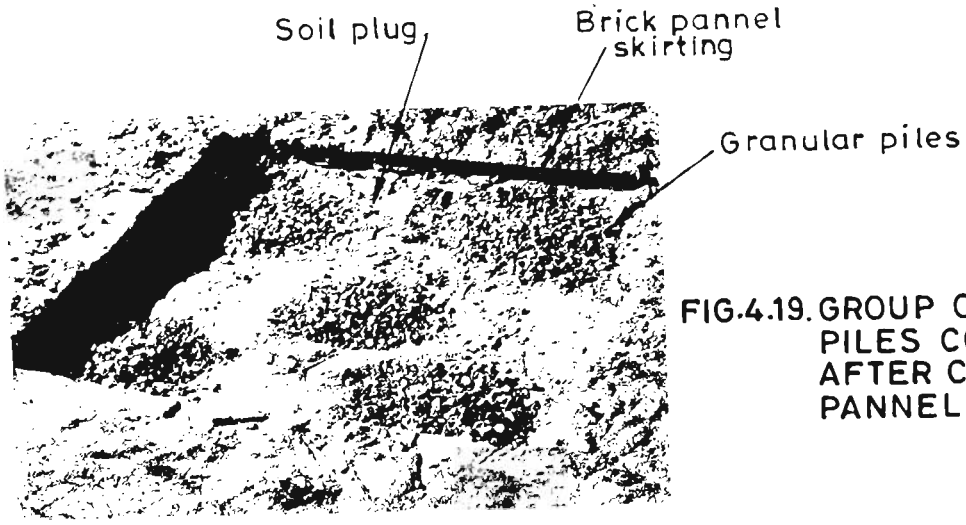


FIG.4.19. GROUP OF 5-GRANULAR PILES CONSTRUCTED AFTER CASTING THE BRICK PANNEL SKIRTING

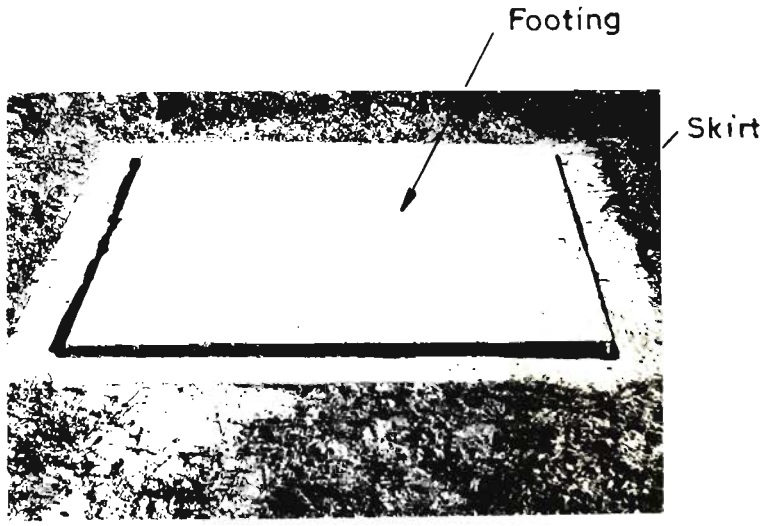


FIG.4.20. CONCRETE FOOTING IS CAST AFTER INSTALLATION OF GRANULAR PILES IN THE SOIL PLUG CONFINED BY THE SKIRT

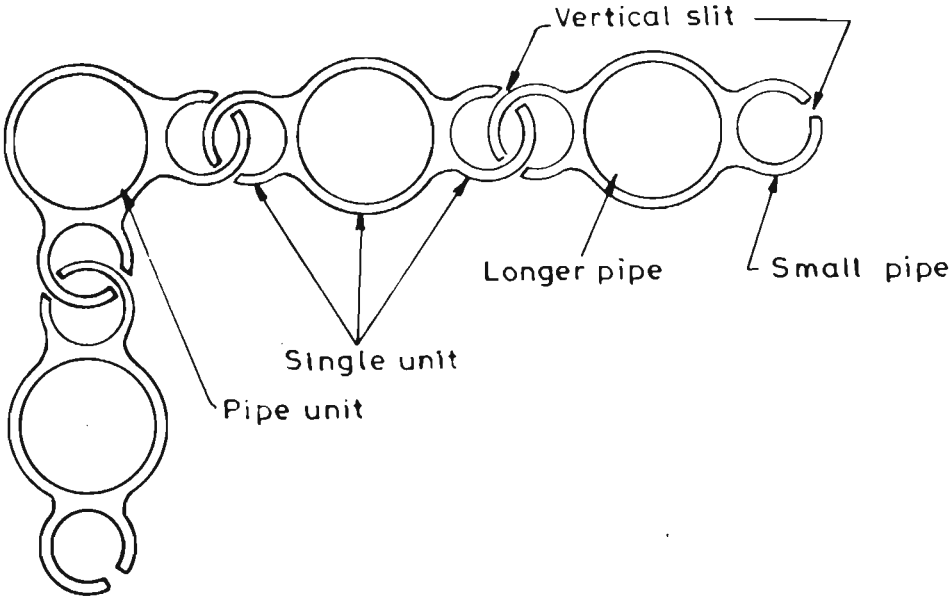


FIG.4.21. A SCHEMATIC DIAGRAM OF PRE FABRICATED PIPE UNIT SKIRTING

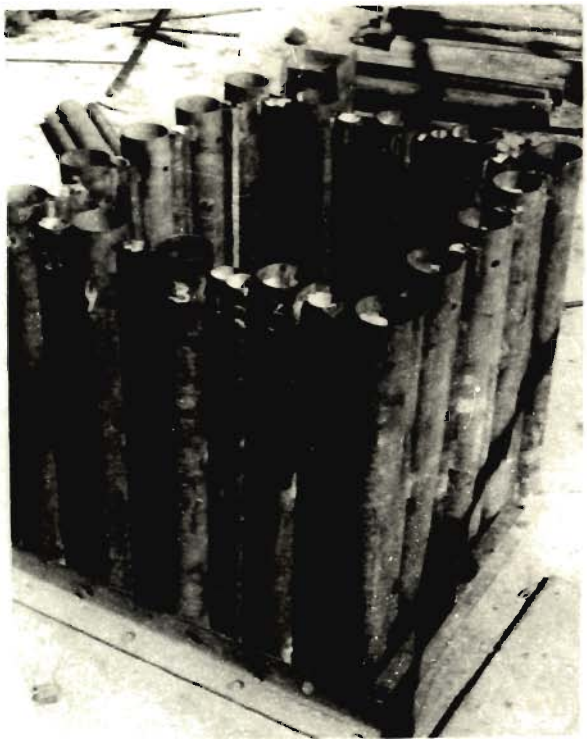


FIG.4.22. CHECKING OF THE PREFABRICATED PIPE UNIT SKIRTING BEFORE DRIVING

FIG.4.23. DRIVING OF INDIVIDUAL PIPE UNIT SKIRTING BY THE POWER WINCH

FIG. 4.24. CONNECTING ONE UNIT WITH ANOTHER UNIT

4.5.7 Precast Cement Concrete Piles, Pipe Piles, and Timber Pile Skirting.

Tests have been carried out using precast cement concrete piles, pipe piles and timber piles to form the skirt, Fig. 4.27. Unlike reinforced cement concrete skirts, these are comparatively more flexible and fail to provide sufficient confinement to the soil plug inside the skirt. Thus the main objective of providing a skirt to obtain increase in load carrying capacity and significant reduction in settlement in comparison to plain footing without skirt is defeated. Thus to make these skirts perfectly rigid, these piles (whether pipes, or timber/concrete precast piles) may be held together by casting a reinforced cement concrete beam connecting all these piles at the foundation level. The beam may be designed for lateral pressure such that it does not deflect laterally. This will make the skirt sufficiently rigid to provide required confinement to the soil plug. However, in the present test programme these skirts were made rigid by providing angle iron or steel channel clamps on all the four sides from both inner and outer faces of the pile skirting. A timber pile skirting with m.s. channel clamps is shown in Fig. 4.28.

4.5.8 Construction of Load Bearing Element

With a view to obtain additional advantage of skirted granular pile foundation to support lateral and uplift loads, the idea of providing a load bearing element in the centre was conceived.

The load bearing element was constructed by using 15cm dia. auger. A central hole Fig. 4.29 in soil plug was first made having a depth of 3m. The other four boreholes were then made at right angles in such a way that all of them merge with the central hole and

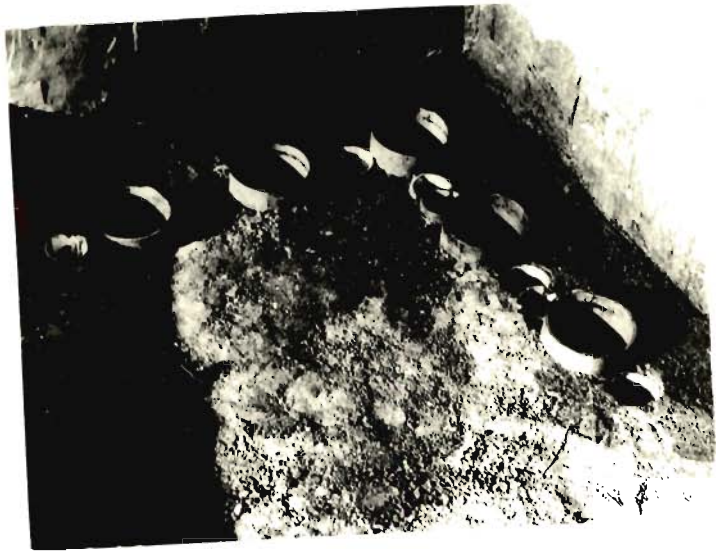


FIG.4.25. A VIEW OF PARTIALLY DRIVEN PIPE UNIT REFRIEVABLE TYPE OF SKIRTING

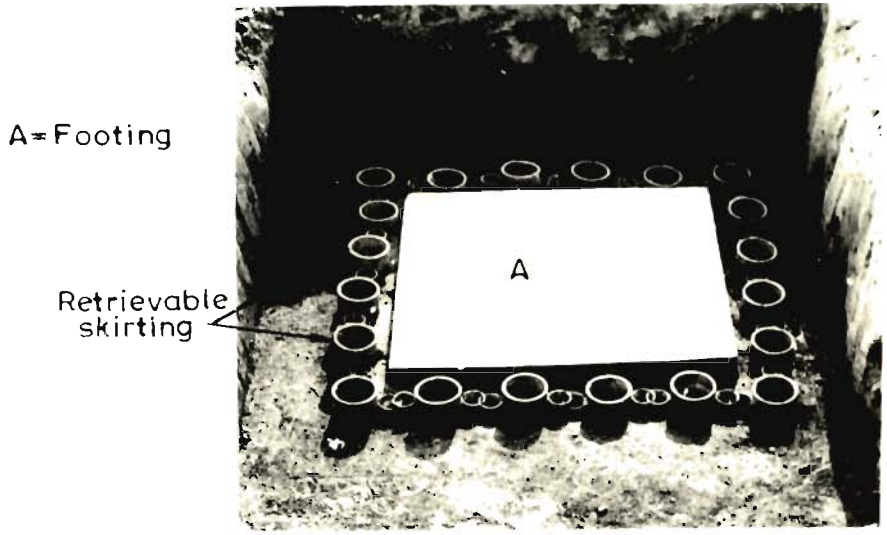


FIG.4.26. A RETRIEVABLE TYPE OF PIPE UNIT SKIRTING AROUND A CONCRETE FOOTING

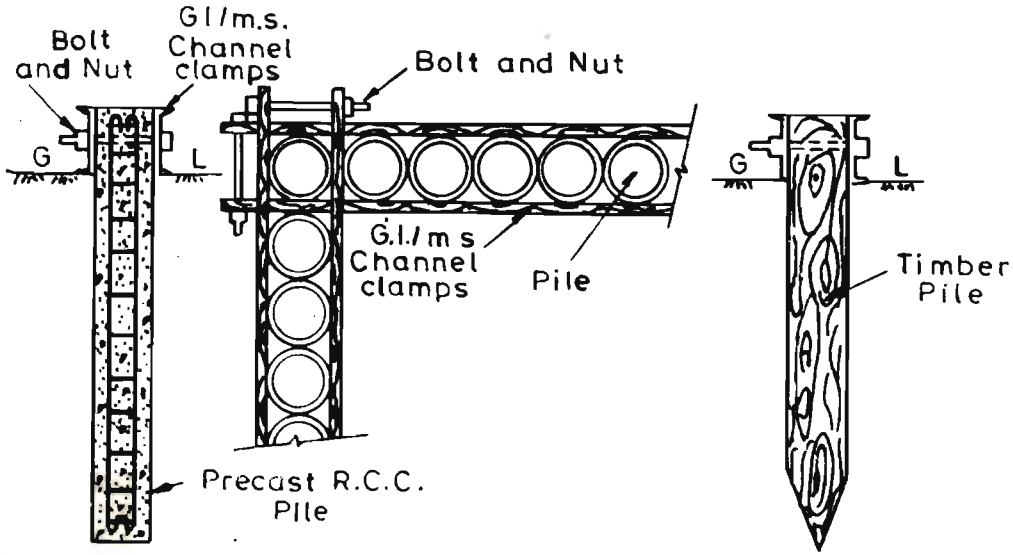


FIG.4.27. G.I./ m.s. PIPE/TIMBER/PRECAST RCC PILE SKIRTING

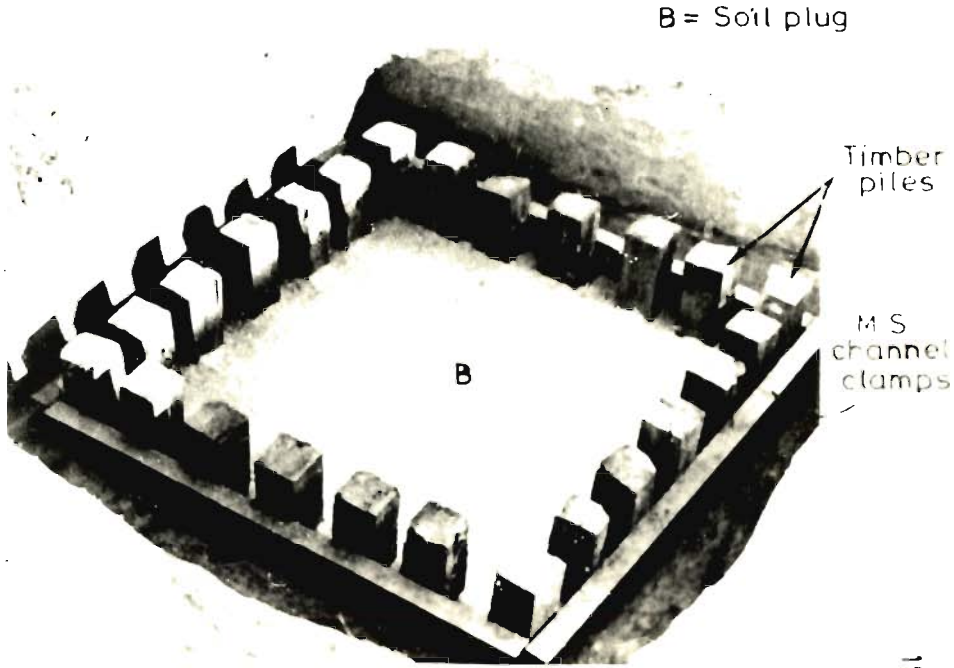


FIG.4.28. A TIMBER PILE SKIRTING

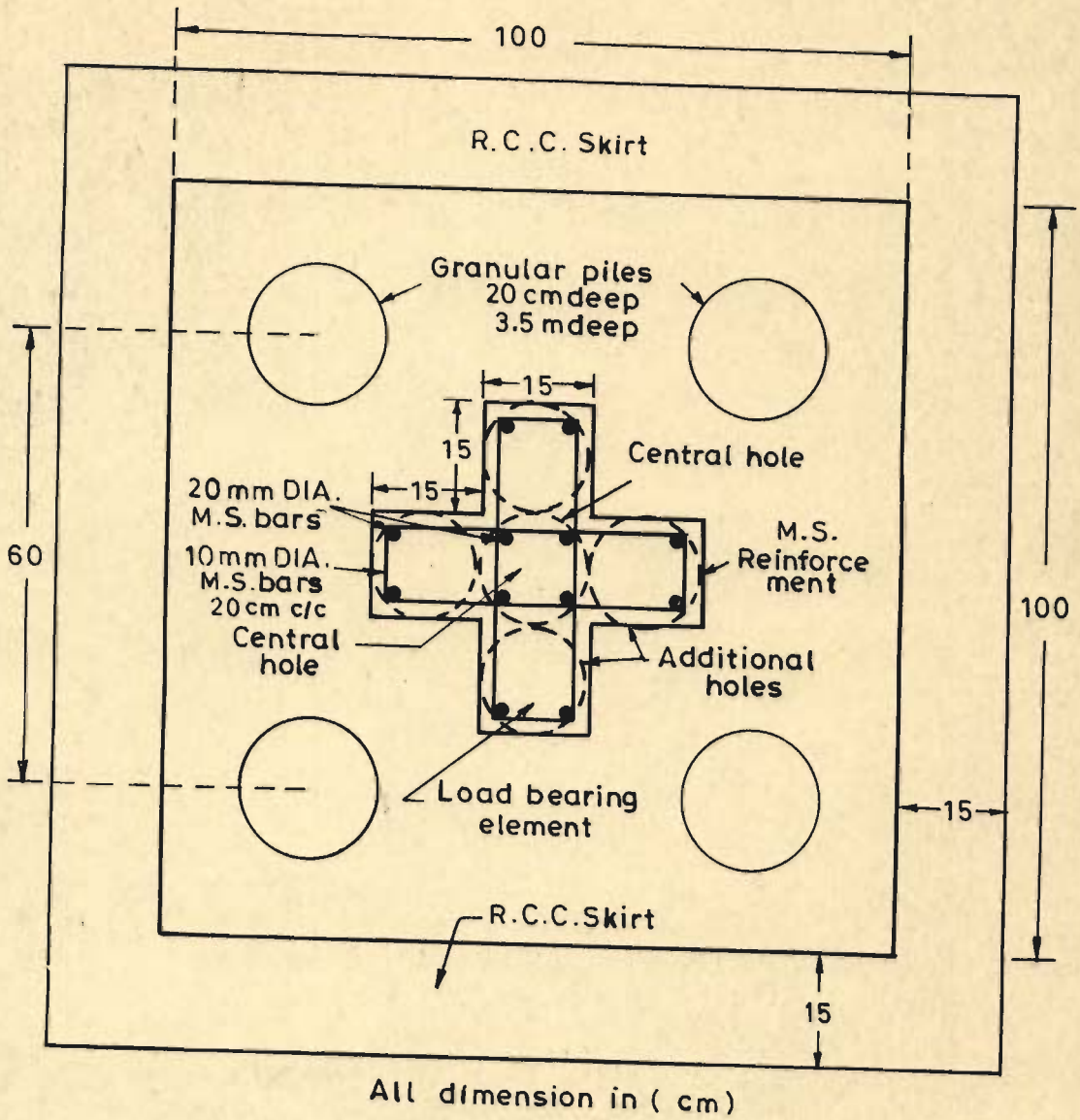


FIG.4.29. A FOUR PILE GROUP WITH A LOAD BEARING ELEMENT SCHEMATIC DIAGRAM

cruciform trench is formed. The corners and edges are then straightened. After placing the reinforcement cage, as detailed in Fig. 4.29, the element was concreted with 1:2:4 cement concrete Fig. 4.30. The vertical reinforcement bars projected outside were bent and connected with the reinforcement of the footing which was cast on the soil plug after the construction of the four granular piles, Fig. 4.31.

4.5.9 Skirted Footings and Skirted Granular Pile Foundation

Following the construction procedures described earlier, 35cm dia., 8m deep granular piles plain and collectively skirted were constructed.Single piles and in groups of two, three and four were used Fig. 4.32 &4.33. The sectional view of plain footing, skirted footing and collectively skirted granular piles without and with load bearing element are shown in Figs. 4.34 and 4.37.

4.6 Details of Test Performed

In-situ load tests were carried out at five different sites consisting of predominantly sandy soils deposit (SM,SP,SM-ML, SM-SP etc) on three sites. The fourth site was having ML & fifth as CH Group of soil deposit. Tests were conducted on single & groups of 2,3,4 and 5 piles having plain granular piles, individually and collectively skirted granular piles having diameters from 25 to 35cm after installation. Different types of skirts were also used as an alternative to permanent r.c.c. skirting. The feasibility of construction and behaviour under load of the retrievable type of skirting with a view to utilise locally available material was also checked. Besides plain reinforced concrete footings were also load tested by casting on virgin soil for comparision with those of skirted footings and skirted granular pile foundations.

FIG.4.30. A-4 PILE GROUP SKIRTED GRANULAR PILE FOUNDATION WITH LOAD BEARING ELEMENT

FIG.4.32. A-3 PILE GROUP SKIRTED FOUNDATION WITH CIRCULAR R.C.C. SKIRTING

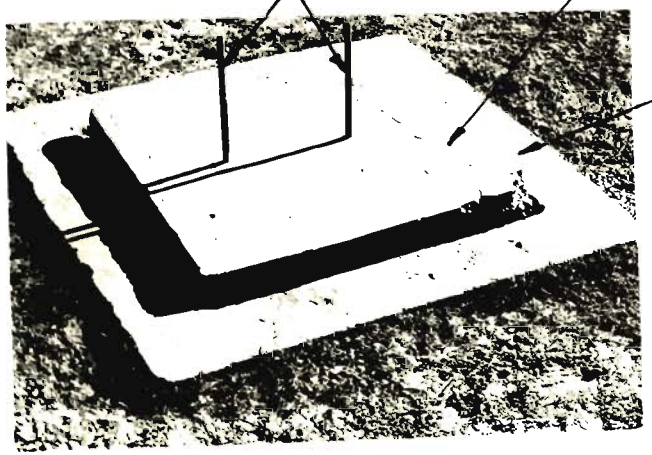


FIG.4.31. A-4 PILE SKIRTED GRANULAR PILE FOUNDATION WITH LOAD BEARING ELEMENT CONNECTED WITH CONC. FOOTING

FIG.4.33. A-4 PILE GROUP SKIRTED GRANULAR PILE FOUNDATION (PILE DIA.35cm AFTER INSTALLATION)

Legend

C - Concrete footing
S - Skirt
G - Granular pile
SP - Soil plug
LBE - Load bearing element

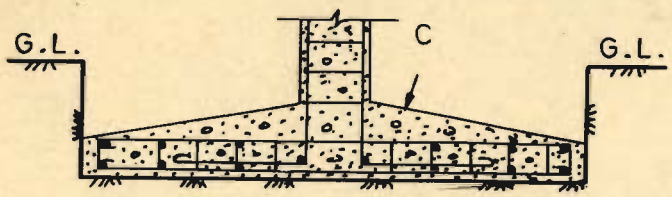


FIG.4.34.PLAIN FOOTING FOUNDATION

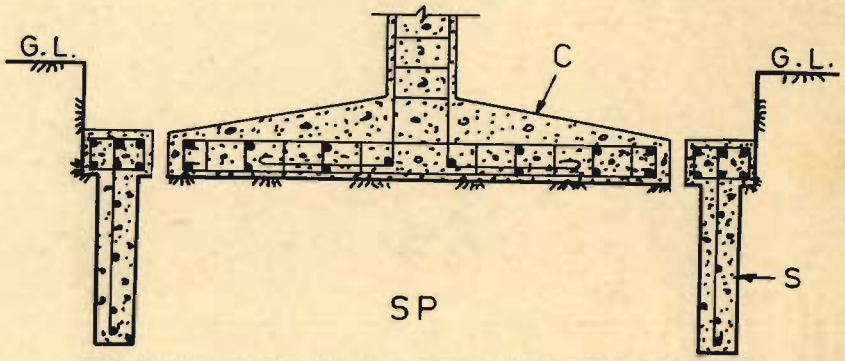


FIG.4.35.SKIRTED FOOTING FOUNDATION

(Not to scale)

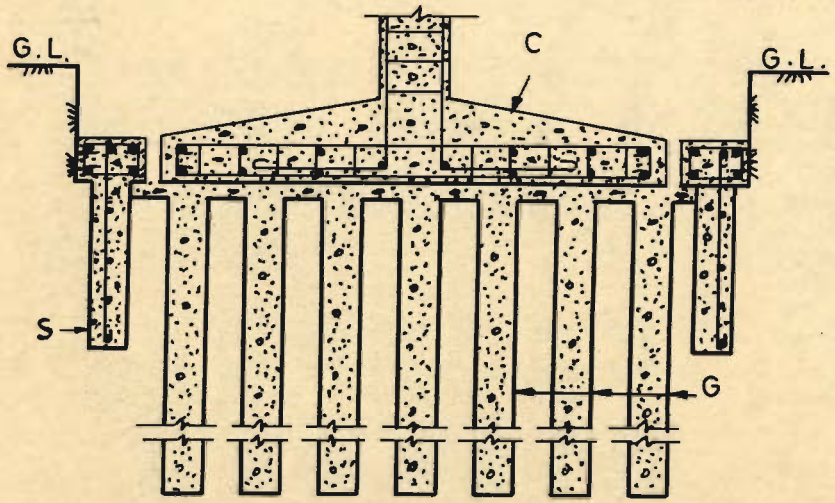


FIG.4.36.SKIRTED GRANULAR PILE FOUNDATION

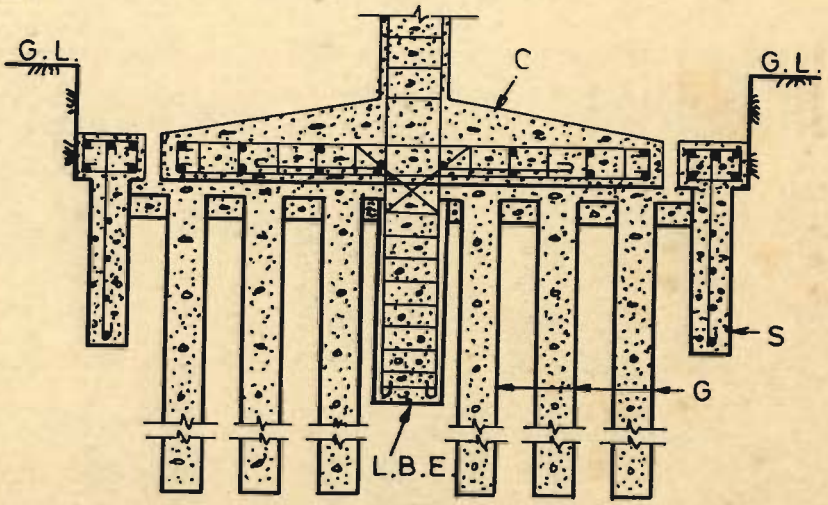


FIG.4.37.SKIRTED GRANULAR PILE FOUNDATION WITH L.B.E.

Some of the skirted granular pile foundation groups with unit piles (cross load bearing elements) were subjected to lateral and uplift loads also. Two plain pile groups, (60cm dia., 9m deep and 55cm dia., 11m deep) were also load tested on fourth and fifth site. The details of various tests performed are given in Table 4.1

4.7 In-situ Testing Procedure and Typical Test Results

4.7.1 Plain Footings on Virgin Soil

Plain Granular piles single and in groups, individually or collectively skirted were subjected to sustained vertical compressive loads upto their ultimate load capacity by insitu load testing.

Load testing on plain footings cast in-situ was carried out using 100 tonnes remote control hydraulic jack. On the surface of the footing, 30mm thick steel plate was placed having dimension slightly lesser than that of the footing over this smaller plates of the same thickness were placed centrally. The plates were properly levelled. The hydraulic jack was then centrally placed on the plate butting against the central point already marked on reaction beam. The reaction beam consisted of the 15cmx30cm RS joists rigidily clamped together at four locations. On the ram of the jack a 100 tonne proving ring with a dial gauge was placed centrally. Now a m.s. plate 25mm thick and 30cm x 30cm in size was placed and the jack was operated to enable the plate to butt against the reaction beam. The reaction beam was supported on two steel columns prefabricated out of m.s. channels welded between two thick 1.5m x 1.5m mild steel plates. The beam was rigidily anchored to the ground by hold fast and spike arrangements. The number of hold fasts used in the test depended upon the amount of reaction required in a particular test. The loads were applied in equal increments. The number of increments and its amount was adopted as one sixth of the probable ultimate load of footing or pile group.

Table 4.1 Details of the In-situ Tests Performed at Various Sites

Test No.	Single/ Group of Granular Piles	Installed pile dia. (cm)	Pile depth (m)	Whether plain (P). Individually skirted (IS) collectively skirted (CS)	Depth of skirt (cm)	Size of the pile cap/Isolated plain footing (cm)	Depth of pile cap below GL(cm)	Depth of the isolated plain footing below GL (cm)	Type of loading compression (C) Tension (T) Lateral (L)	Remarks
1	2	3	4	5	6	7	8	9	10	11
1.	CBRI Officer's Hostel									
1a	Single pile	25	3.5	P	-	25 (dia.)	10	-	C	M.S. Plate as pile cap loading the pile top only
1b	Single pile	25	3.5		-	40 x 40	10	-	C	M.S. Plate footing as pile cap
2	Group of 2	25	3.5	P	-	40 x 120	10	-	C	-do-
3	Group of 3	25	3.5	P	-	120 (Dia)	10	-	C	R.C.C. footing as pile cap.
4.	Group of 4	25	3.5	P	-	120 x 120	10	-	C	-do-
5	Single pile	25	3.5	IS	100	40 x 40	10	-	C	Individual skirting using mild steel pipes 30cm dia.

contd....

Table 4.1 (Contd.)

1	2	3	4	5	6	7	8	9	10	11
6	Group of 2	25	3.5	IS	100	40 x 120	10	-	C	Individual skirting using mild steel pipes 30cm dia.
7	Group of 3	25	3.5	IS	100	120 (Dia)	10	-	C	-do-
8	Group of 4	25	3.5	IS	100	120 x 120	10	-	C	-do-
9	Isolated Plain footing					40 x 40		20	C	M.S. Plate
10.	Footing on virgin ground					120 (dia)		20	C	R.C.C. footing
11	-do-					120 (Dia)		20	C	-do-
12	-do-					90 x 90		20	C	-do-
13	-do-					120 x 120		20	C	-do-
14	Single pile	35	8.0	P	-	70 x 70	10	-	C	M.S. plate as pile cap
15	Group of 2	35	8.0	P	-	85 x 140	10	-	C	-do-
16	Single pile	35	8.0	CS	40	70 x 70	10	-	C	Concrete skirting reinforced with 1.6mm thick M.S. sheet & M.S. Plate as pile cap.
17	Group of 2	35	8.0	CS	45	85 x 140	10	-	C	-do-
18	Group of 3	35	8.0	CS	80	160 (Dia)	10	-	C	-do- and R.C.C. footing as pile cap.

Contd....

Table 4.1 (contd.)

1	2	3	4	5	6	7	8	9	10	11
19	Group of 4	35	8.0	CS	80	150 x 150	10	-	C	-do-
20	Group of 6	25	3.5	CS	50	100 x 100	8	-	C	Brick pannel (pre-fabricated) skirting
21	Isolated plain footing on virgin ground					70 x 70		15	C	M.S. Plate
22	-do-					85 x 140		15	C	-do-
23	-do-					160 (Dia)		15	C	R.C.C. footing
24	-do-					150 x 150		15	C	-do-
25	-do-					100 x 100		10	C	-do-
26	Single R.C.C. Unit pile or load bearing element	15cm thick 45cm long two units at right angles	3.0 (LBE)						T	LBE was cast insitu R.C.C. member without skirting for uplift test
27	Single unit pile or load bearing element	15cm thick 45cm long two units at right angles							T	LBE was cast insitu R.C.C. member without skirting for uplift test lateral load test lateral load was applied at GL
28	Single unit pile or load bearing element with 4-pile group & RCC skirt	LBE 15cm thick 45cm long two units at rt angles granular piles 25cm dia.	3.0 (LBE) 3.5 (Pile)	CS	50	100 x 100	10	-	T	LBE was cast insitu RCC member with RCC skirting reinforced with M.S. bars Granular piles without tie bars & uplift load was applied on LBE only

Table 4.1 (Contd)

1	2	3	4	5	6	7	8	9	10	11
29	Single RCC unit pile or a load bearing element with 4-pile group & a RCC skirt	LBE 15cm thick 45cm long two units at rt angles granular piles 25cm dia.	3.0 (LBE) 3.5 (granular)	CS	50	100x100	10	-	T	LBE was cast insitu RCC member with RCC skirting using M.S. bar reinforcement Granular piles with 16mm dia. tie bars. Uplift load was applied on LBE and Granular pile group together.
30	Single RCC unit pile or load bearing element with 4 pile group & a RCC skirt	-do-	3.0 (LBE) 3.5 (granular pile)	CS	50	100x100	10	-	C	LBE was cast insitu RCC member with RCC cast insitu skirt using M.S. Bar reinforcement. 4 granular pile group cast without the bars. Compression load applied through a 100x100 conc. footing cast on pile top & LBE as pile cap within the skirt.
31	Plain RCC footing	-	Virgin ground	-		100x100		20	C	RCC footing
32	Skirted footigg without granular piles	-	Without piles	-	100	100x100	20	-	C	(10cmx10cm) size timber piles used as skirting while rigidily held at foundation level by M.S. Chennel clamps RCC footing as pile cap.
33	Group of 4-piles collectively skirted	25	3.5	CS	100	100x100	15	-	C	-do-
34	Plain RCC footing	-	Virgin ground	-		90 x 90		40	C	R.C.C. footing

Contd....

Table 4.1 (Contd.)

1	2	3	4	5	6	7	8	9	10	11
35	Skirted footing without granular piles	-	Without pile	-	100	90 x 90	40	-	C	10cm dia. one, 5cm dia. two nos pipe welded together with vertical slit in smaller pipes were used as skirting. The pile cap was of cast insitu slab.
36	Skirted footing with 4-pile group	25	3.5	CS	100	90 x 90	30	-	C	-do-
37	CBRI Campus site single pile	25	3.5	P	-	-	-	-	T	All these granular piles were constructed after a 16mm dia. tie rod with a m.s. plate embedded in 30cm thick concrete at the bottom of the bore hole.
38	Group of 2	25	3.5	P	-	-	-	-	T	
39	Group of 3	25	3.5	P	-	-	-	-	T	
40	Group of 4	25	3.5	P	-	-	-	-	T	
41	Molasses tank site Baudan Group of 2	35	3.5			94 x 125	15		C	Pile cap was RCC footing
42	Group of 4	35	3.5	CS	1.5	180x200	15		C	RCC skirt was reinforced with m.s. bars and pile cap was RCC slab.
43	Plain footing	-				60 x 60		150	C	RCC footing was replaced by M.S. plate

Contd.....

Table 4.1 (contd.)

1	2	3	4	5	6	7	8	9	10	11
44	Granular pile group of 230 piles	35	3.5	CS	120cm below GL, 60cm above GL	2400 cm. dia.	Flexible pad at GL	-	C	Actual application of the skirted granular pile foundation on a live structure 24m dia. 6.75m high molasses tank. 3000 cu. m. capacity. The concrete skirt was reinforced with m.s. bars and sand-witched between brick work.
45	Drilling Rig site Silchar (Assam									
	Single pile	60	9.0	P	-	75x70	15	-	C	The pile cap was r.c.c. footing
	Group of 2	60	9.0	CS	4m deep	175x100	15	-	C	Collectively skirted, using 20cm dia. 4m deep timber piles driven closely and jointed together at the foundation level by r.c.c. cast insitu beam. The pile spacing was kept as 1.35m
46	Furnace Oil Tank site Haldia (WB)									
	Group of 2	55	9.5	P	-	200x100	20	-	C	Pile spacing was kept as 1.41m
	Group of 2	55	14.0	P	-	200x100	20	-	C	This pile group was tested at mandatory crude oil tank site. The spacing was same as in furnace oil tank site.

Note: In all the cases of group testing of piles the pile spacing was maintained as 3 (pile dia. after installation) however, in the case of molasses tank site at Budaun the pile spacing was increased to 3.5 pile diameter.

In a single test a minimum of eight load increment was used. The deformation of the footing/pile group plain or skirted was measured with the help of baty dial gauges 4 numbers placed on the footing at corners. The least count of the gauges was 0.01mm tier division and the travel was 50mm. The dial gauges were mounted on magnetic bases fixed on independent datum bars. The datum bars were fabricated out of m.s. channels with two supports at the end. The length of the datum bars were kept long enough so as to avoid any effect of movement within the influence zone (generally larger than 3 times the size of the footing). The each loading increment was maintained for a period of one hour or when the rate of increase in the deformation was less than 0.02mm per minute. A close up view of three [illegible] granular pile group with all details is shown in Fig. 4.38. In the case of large pile group the reaction was obtained from a loading platform besides the hold fast and spikes arrangements. The loading platform was 120t. capacity and the kentledge in the form of sand bags were stacked systematically Fig. 4.39. The settlement was taken as average of the four dial gauges. The load testing was stopped only when the footing clearly punched into the ground when the rate of deformation was high and was not readable.

A four pile group collectively skirted granular pile foundation after load testing is shown in Fig. 4.40. The site plan showing the layout of the various test facilities at this site is given in Fig. 4.41 and the typical load-deformation behaviour of single and groups of 2,3 and 4 plain and individually/collectively skirted granular piles are shown in Figs 4.42 and 4.43. The load-deformation curves for various footing tests on virgin soil is also shown in Figs. 4.42 and 4.43. Similarly a 25cm dia. 3.5m deep collectively skirted granular pile group was also load tested

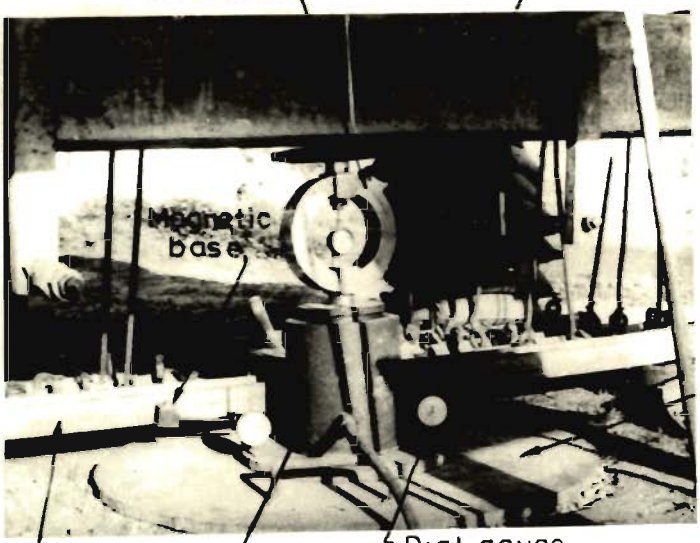


FIG.4.38. IN SITU LOAD TESTING OF 3 PLAIN PILE GROUP

FIG.4.39. LOADING PLATFORM SHOWING SAND BAG KENTLEDGE AND USING HOLD FASTS AS ANCHORS

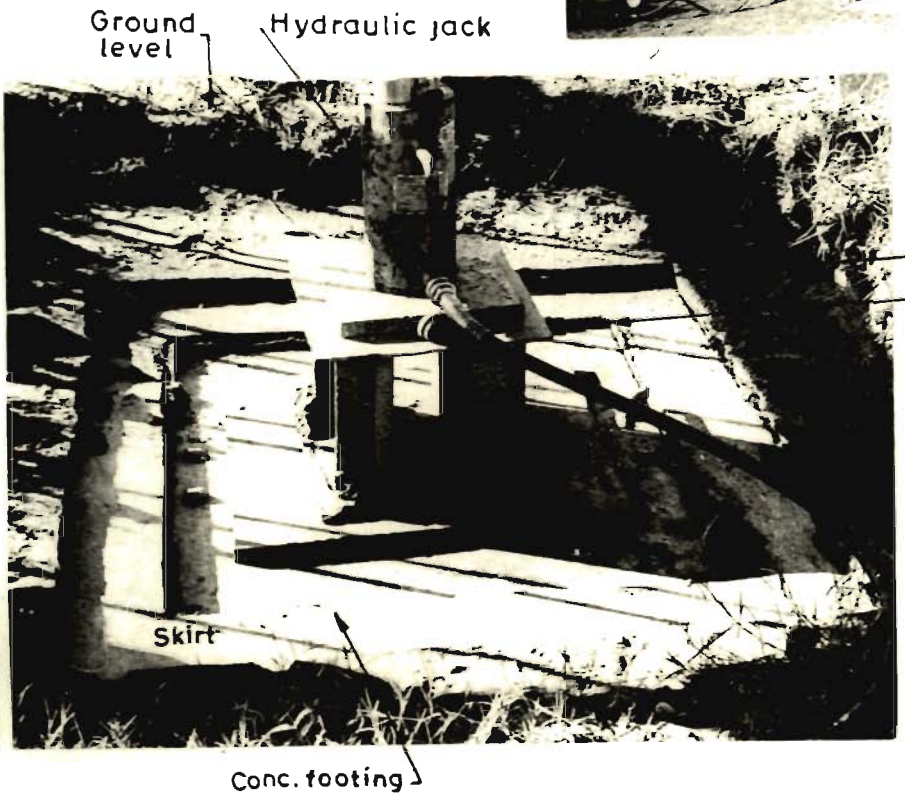


FIG.4.40. A 4 PILE GROUP SKIRTED FOOTING AFTER IN SITU LOAD TESTING

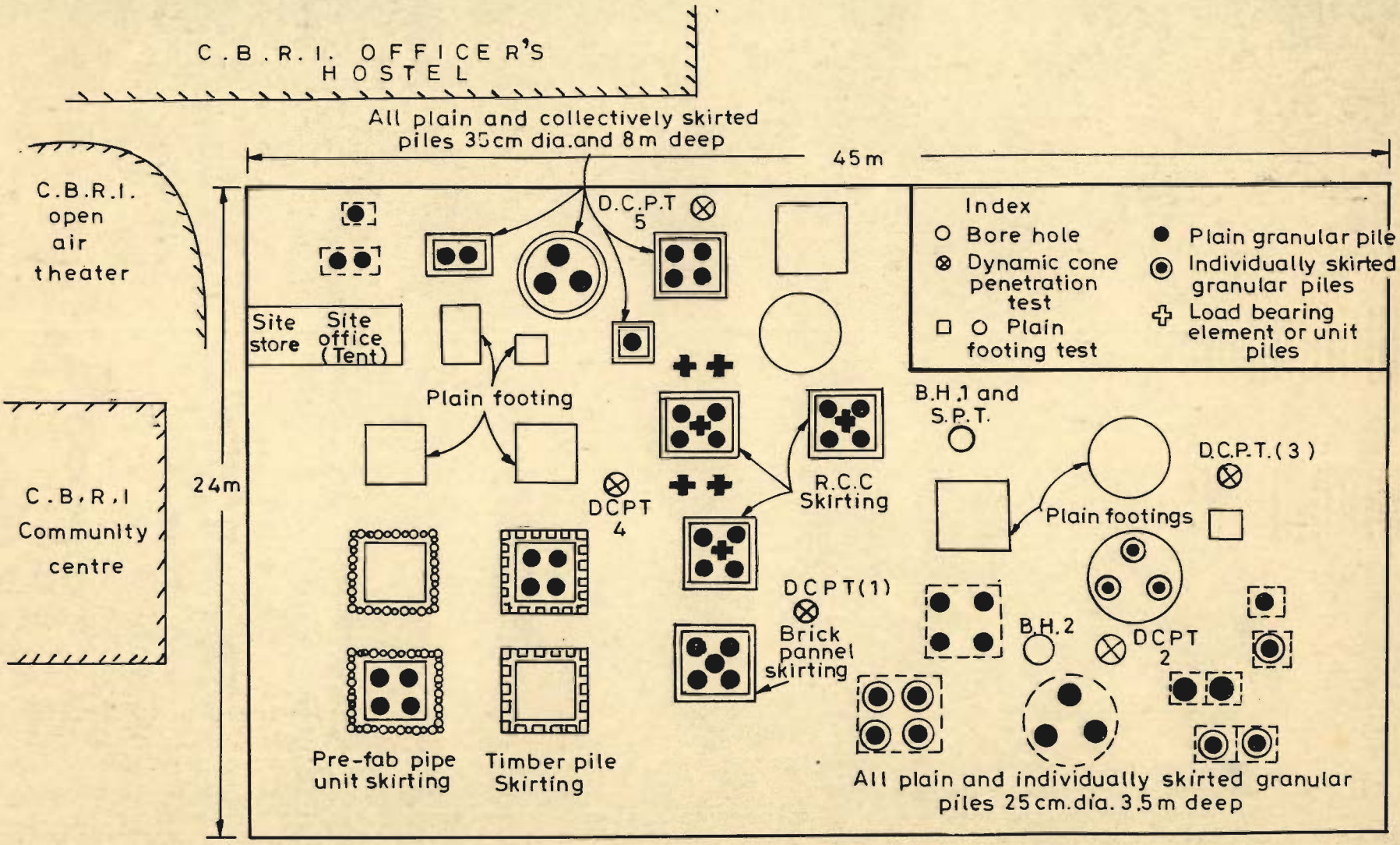


FIG.4.41. TEST FACILITIES C.B.R.I. OFFICER'S HOSTEL

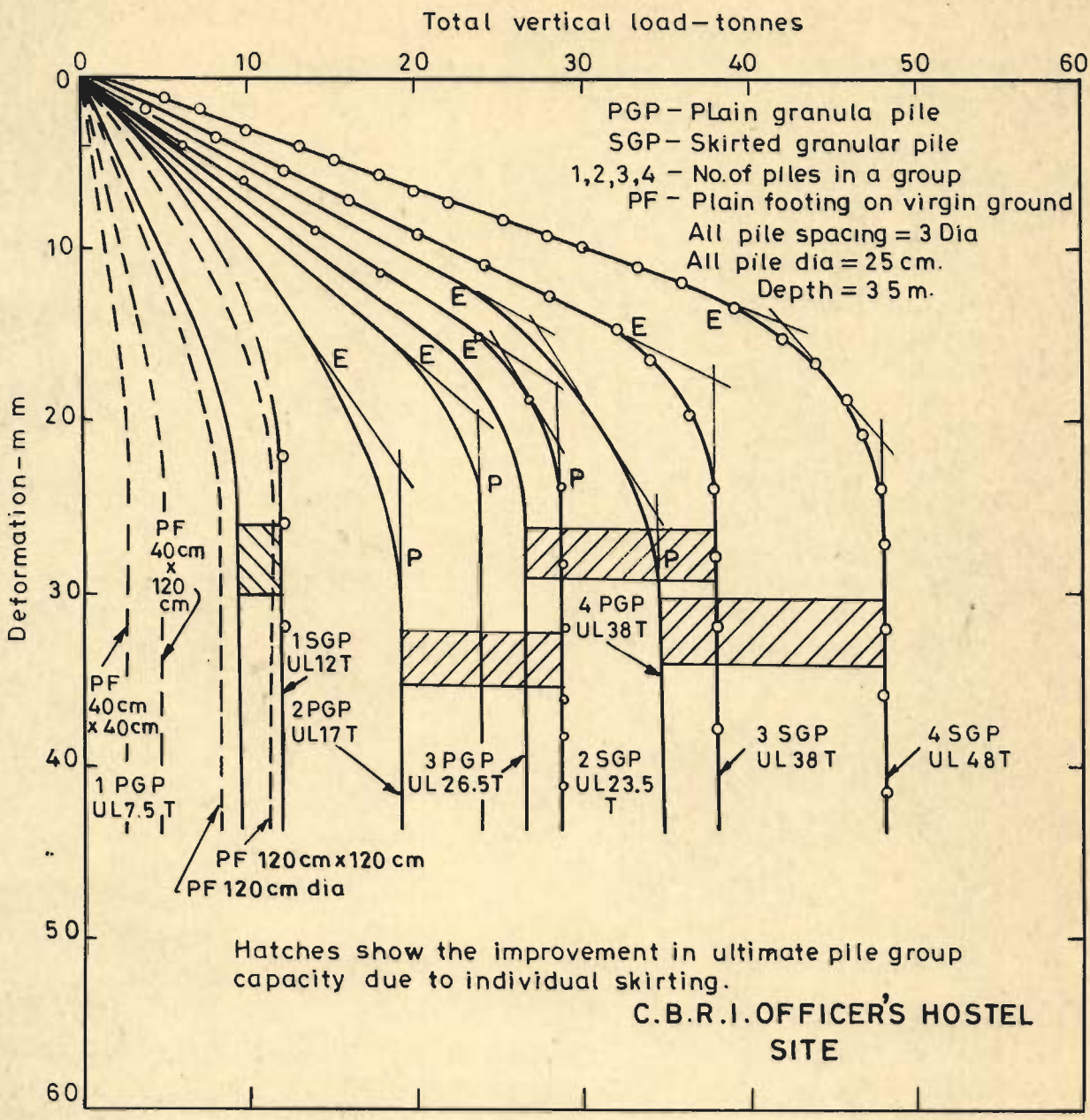


FIG.4.42. LOAD DEFORMATION BEHAVIOUR OF SINGLE AND GROUPS OF 2,3 & 4 PILE GROUPS OF PLAIN AND INDIVIDUALLY SKIRTED PILES

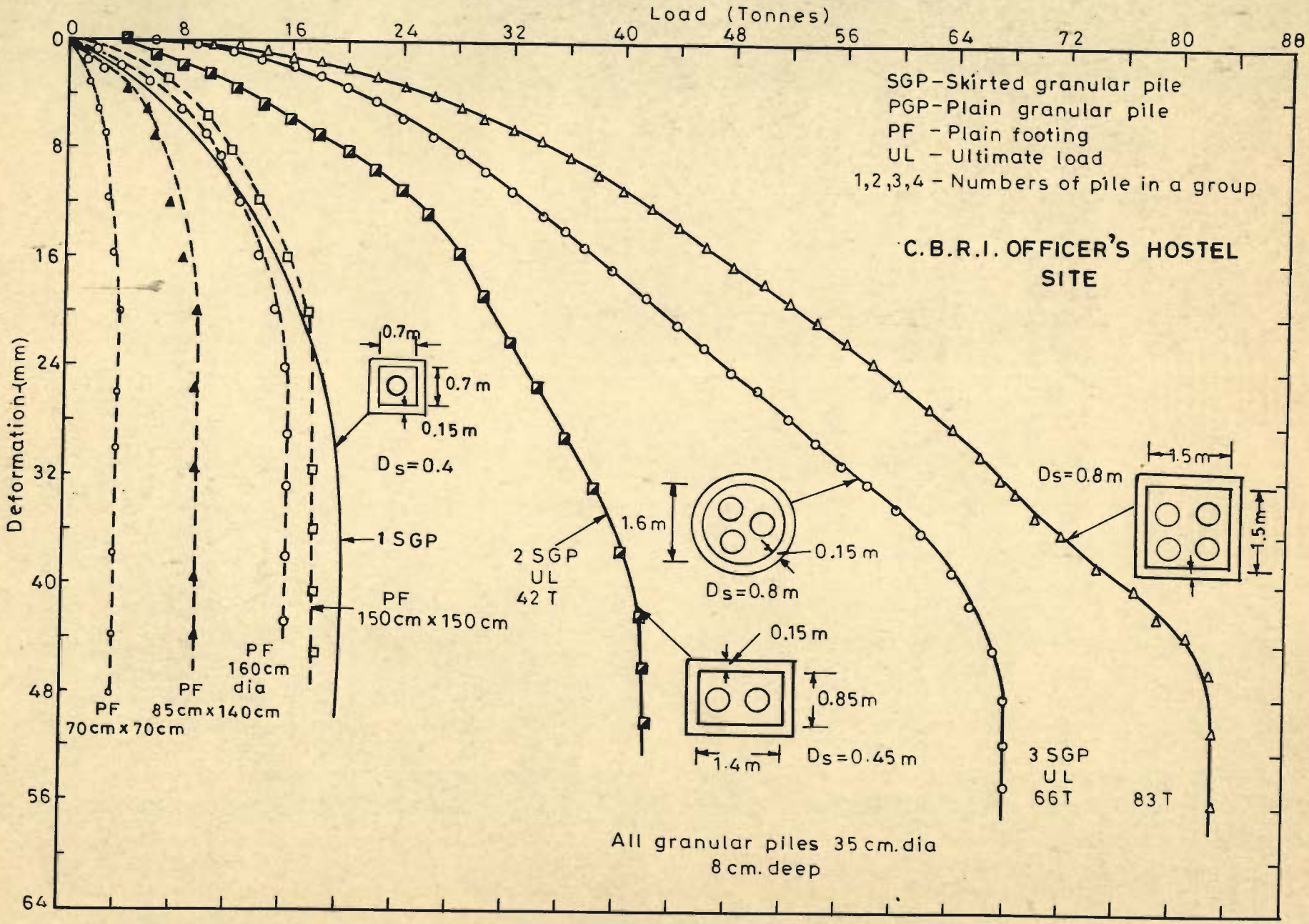


FIG.4.43- LOAD DEFORMATION BEHAVIOUR OF SINGLE AND GROUPS OF COLLECTIVELY SKIRTED GRANULAR PILE.

besides a r.c.c. unit pile (15cm thick 45cm long two cross units, 3m deep) collectively skirted. The load-deformation curves for these groups and the corresponding plain footing on virgin ground is shown in Fig. 4.44. The test facilities at Budaun site (UP) showing site plan, Fig. 4.45 and the load displacement curves for two plain granular piles group and a 4-pile group collectively skirted using r.c.c. skirt is shown in Fig. 4.46.

4.7.2 Plain and skirted granular pile groups with load bearing element (unit pile) subjected to uplift loads

Single and groups of granular piles were provided with a central mild steel bar 16mm in dia. (art. 4.5.2) which was threaded at the end. These bars were extended by another meter length with the help of couplers and m.s. bars of the same dia. These bars were finally bolted in a frame through the holes already provided (Fig. 4.31). Two rolled steel joists coupled together with the help of the clamps at four places (same as in compression tests) was placed centrally through the pile group. This beam was supported at the end by two wooden blocks leaving a clear gap of atleast 10cm from the average ground surface. The level of the reaction beam was checked before the start of the test. A 50 tonnes capacity hydraulic jack was placed on the reaction beam centrally. Between the reaction beam and the base of the jack a thick mild steel plate was also introduced with a view to distribute the load. Also packing plates were used to fill the gap between the top plate of the reaction frame and the ram of the jack. Four dial gauges (0.01mm and 50mm lead) were placed through the baty magnetic bases on the strips welded in the extension rods.

The magnetic bases in turn were fixed on two independent datum bars as in the compression tests. All the dial gauges readings

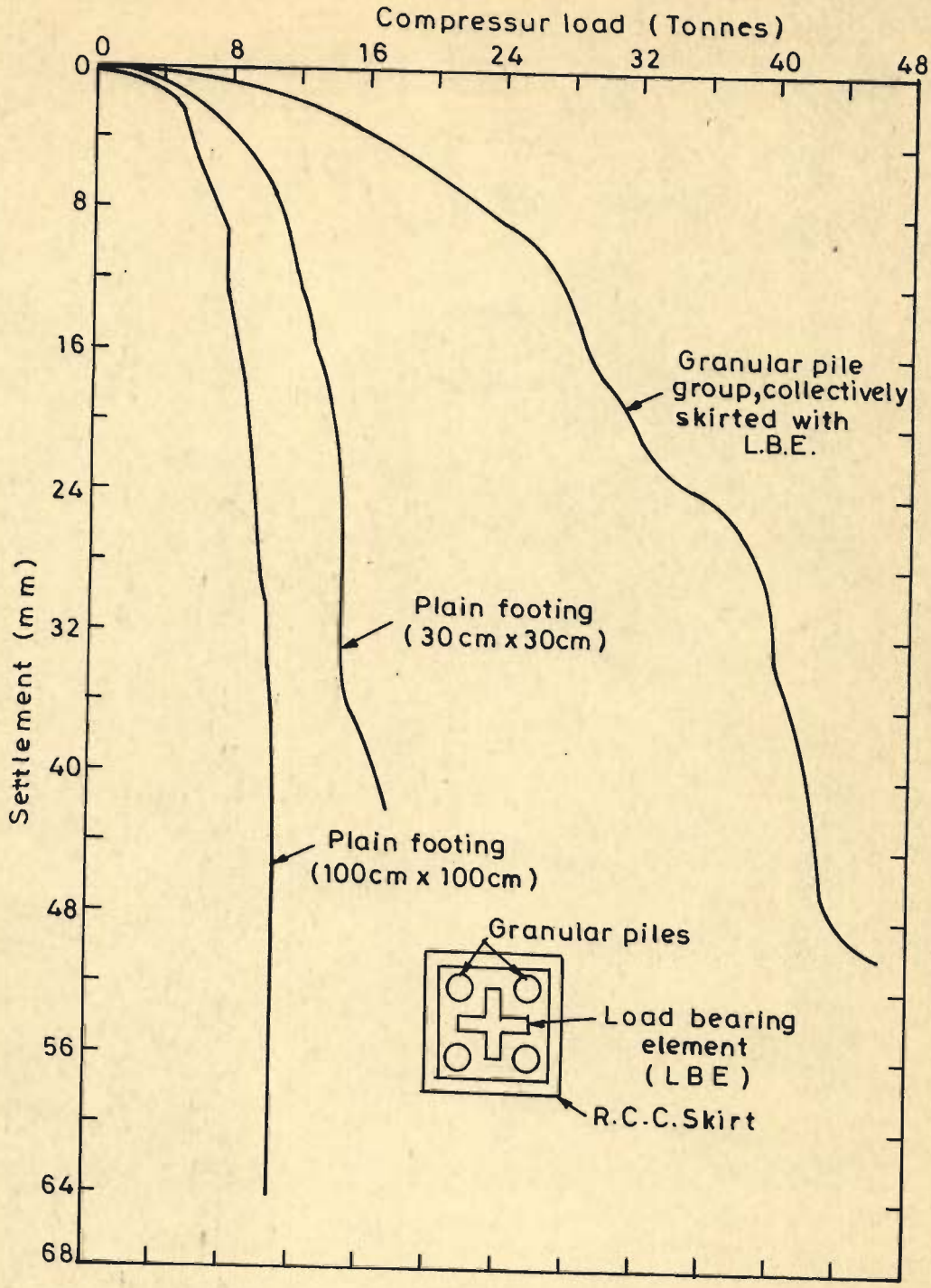


FIG.4.44- LOAD DEFORMATION BEHAVIOUR OF S.G.P. FOUNDATION WITH L.B.E. AND PLAIN FOOTING UNDER COMPRESSIVE LOAD

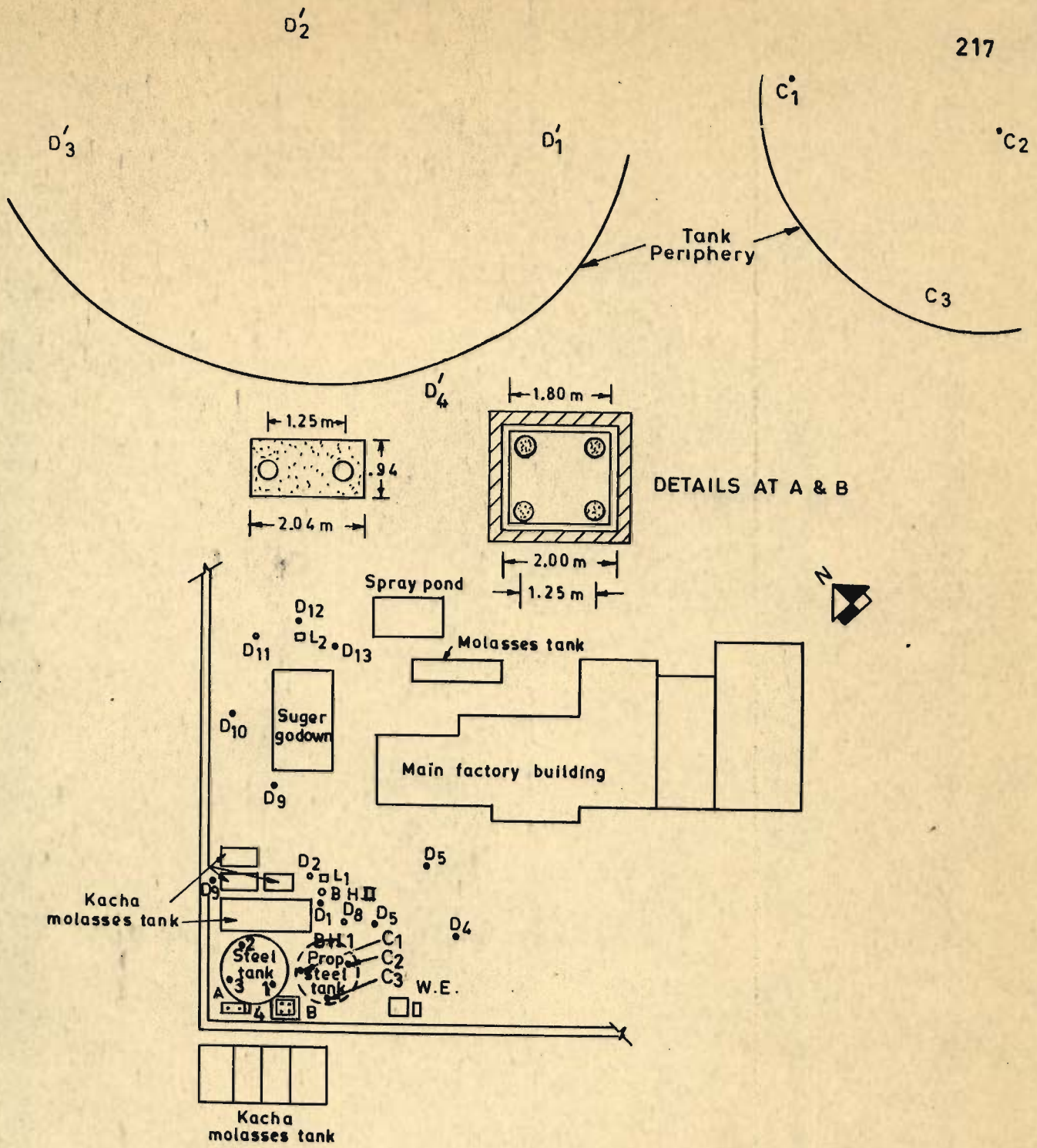


FIG.4.45- SITE PLAN MOTASSES TANK
SITE BUDAUN

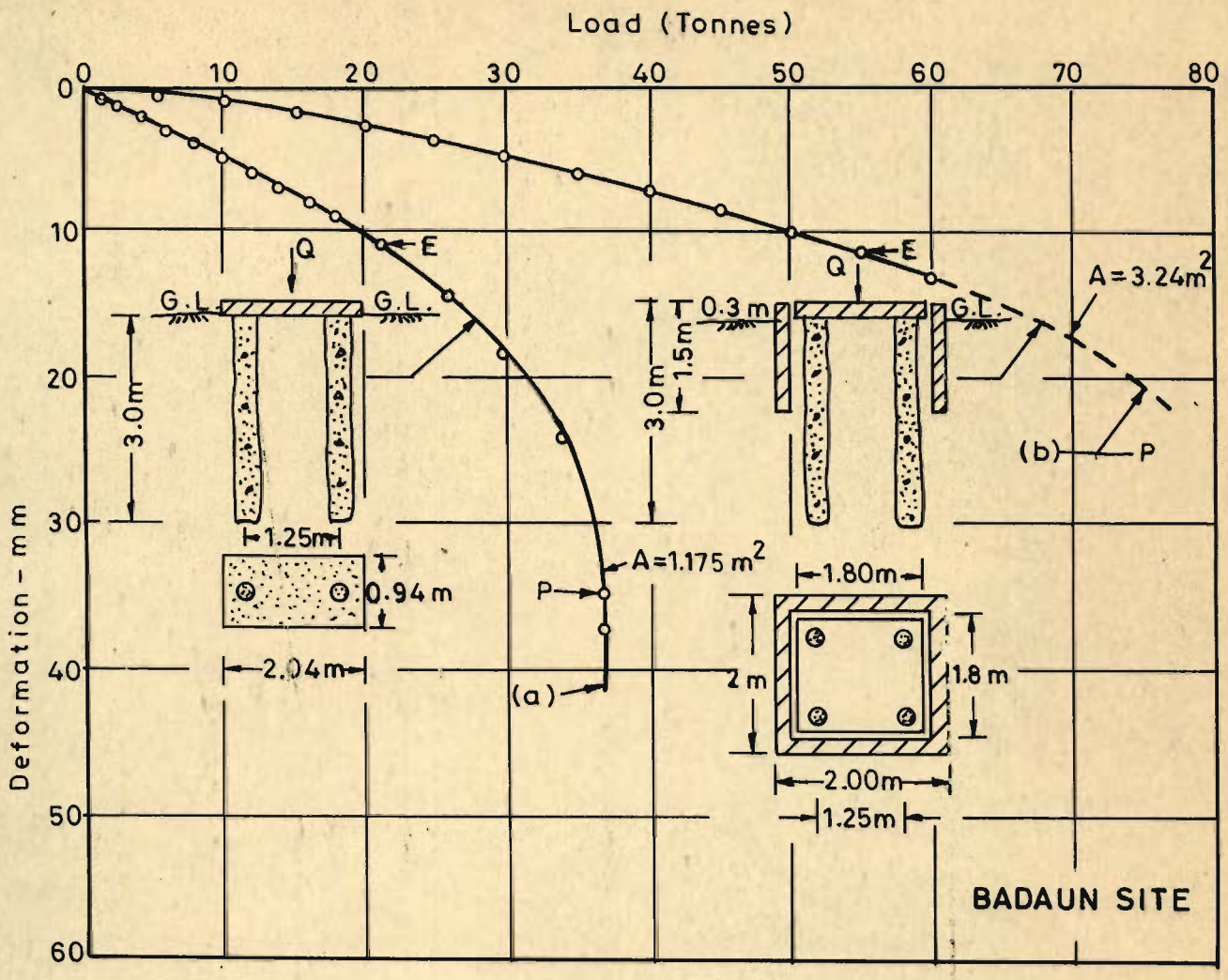


FIG. 4.46. LOAD SETTLEMENT CURVES FOR GROUP OF 2 PLAIN PILES AND GROUP OF 4 SKIRTED PILES

were made at zero point and so also the pressure gauge reading in the hydraulic -jack.

The loads were applied through the jack by a remote control pump in equal increments to achieve at least eight points on the uplift load - displacement curves. The recording of the readings and the test procedure was the same as in the case of compression tests. All the pile groups were tested under pulling loads till the rate of pull of the pile was large and was not readable from the dial gauges. This was clearly indicated by the continuous movement of the dial gauge readings. Fig. 4.47 shows the test set-up during the pull out test of four pile group. A typical uplift load vs pull movement of single and groups of 2,3 and 4 piles 25cm dia. and 3.5m deep at CBRI campus site is shiwn in Fig. 4.48.

Similarly a r.c.c. unit pile (cross load bearing element), a 4-pile group with a central load bearing element and a 4-pile group with central load bearing element both collectively skirted was subjected to uplift loads. The isolated load bearing element was load tested under pull alone while in the case of two skirted granular pile groups with load bearing element in one case, the pull load was applied only to LBE and in the other case the pull load was applied to both at a time i.e. the central LBE and the 4-granular pile group through the tie bars simultaneously. The uplift load vs the vertical pull movement of all these three pull out tests are shown in Fig. 4.49.

4.7.3 Load Bearing Element (unit pile) and the Skirted Granular Pile Group with Central LBE Subjected to Lateral Load.

In the case of insitu lateral load tests the horizontal loads

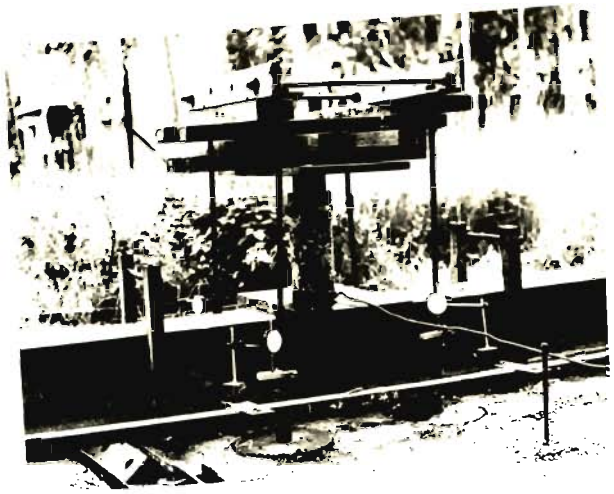


FIG. 4.47. INSITU LOAD TEST SETUP FOR FOUR PILE GROUP SUBJECTED TO UPLIFT LOADS AT C. B.R.I. CAMPUS SITE.

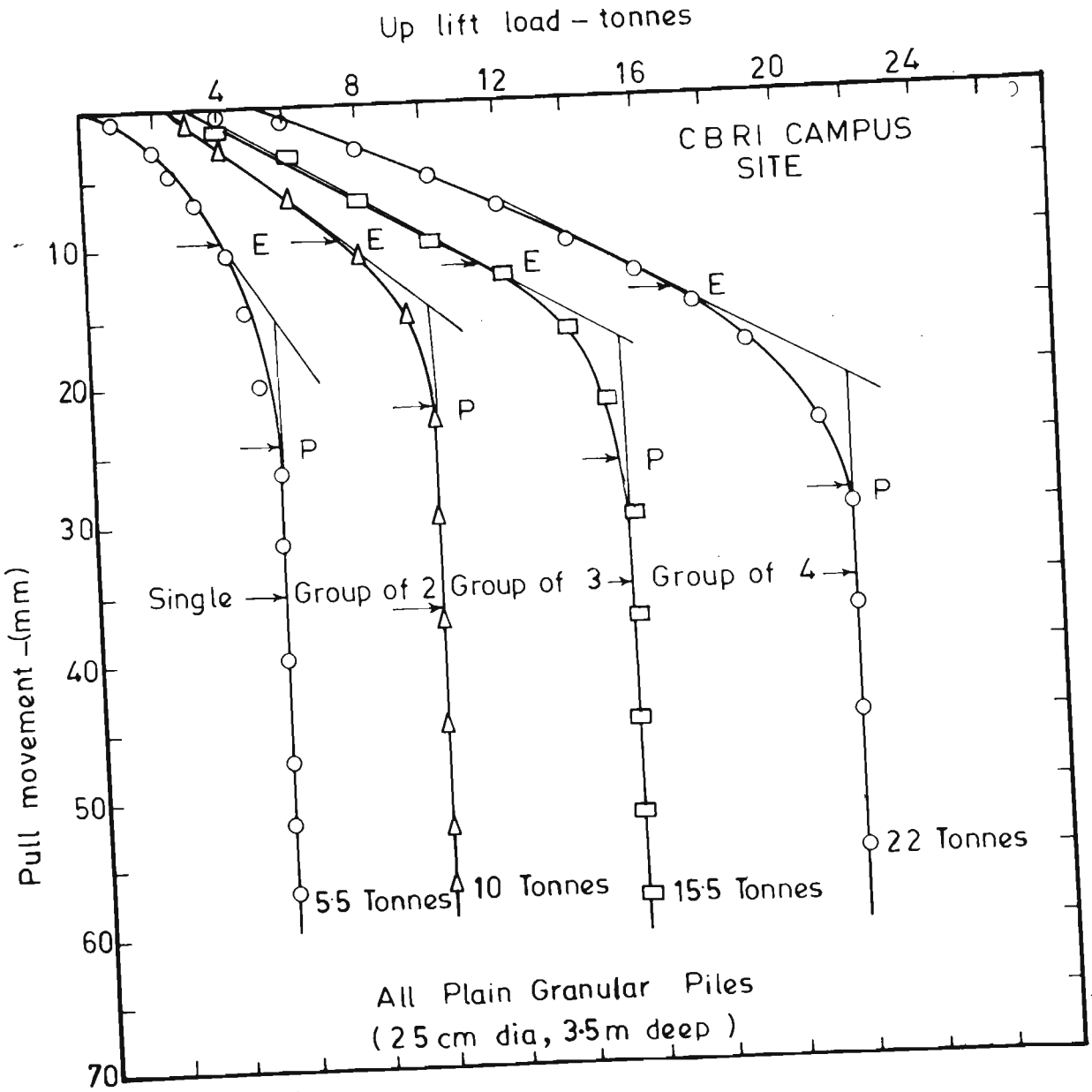


FIG 4·48 UP LIFT CAPACITY OF GRANULAR PILE GROUPS

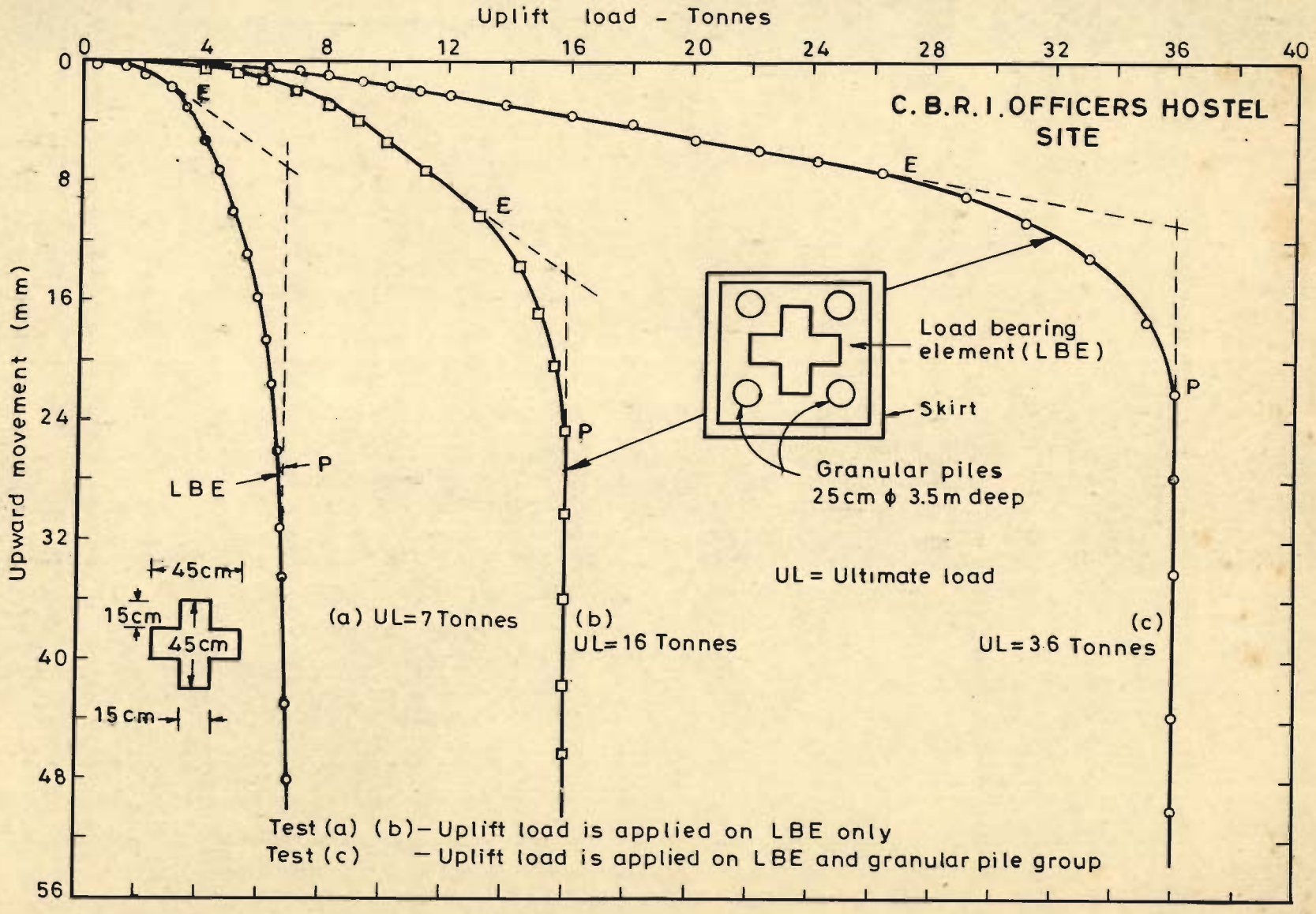


FIG.4.49. UPLIFT LOAD V/S PULL MOVEMENT. BEHAVIOUR OF LOAD BEARING ELEMENT AND SKIRTED GRANULAR PILE FOUNDATION ON VIRGIN AND IMPROVED GROUND

were applied with the 100 tonnes capacity remote control type hydraulic jack by placing the jack horizontal on the wooden support especially made for the purpose with a semi-circular cylinderical notch to allow the seating of the jack properly. The wooden support was placed at such a depth so that the central axis of the jack and the top of the LBE fall in the sameline. Care was taken to fix up the level of the jack so that the horizontal load is applied at the ground level. The lateral displacements at the ground level only were taken with the help of dial gauges (0.01mm) having 50mm lead, fixed at the two ends of the load bearing elements in the same line horizontally at the opposite side of the load application face. The dial gauges were supported through magnetic bases fixed on rigid m.s. long channel used as the datum bar. The reaction for the hydraulic jack was obtained either from a wooden sleeper held rigidily by large number of hold fasts and spikes driven into the virgin ground or other load bearing elements/skirted granular pile foundations used in compression tests. The horizontal loads were applied in increments of 1 tonne each and the corresponding lateral displacements were measured. The testing procedure in the lateral test and the measurements of the load and displacement was the same as in compression tests. The lateral load tests in the case of skirted granular pile group with a LBE was carried out in the same way. Typical lateral load-displacement curve of LBE and the skirted granular pile foundation with LBE is shown in Fig. 4.50.

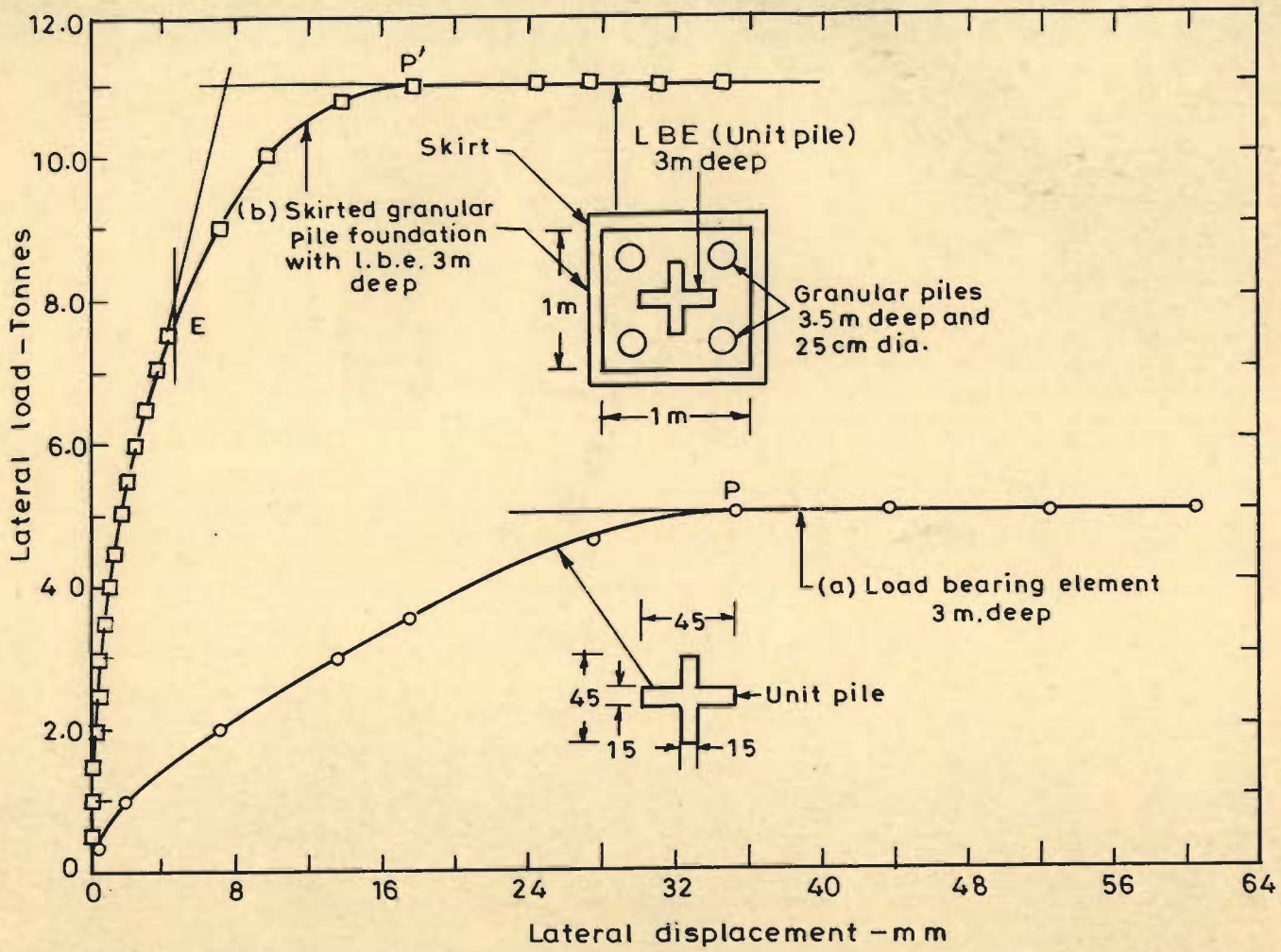


FIG.4.50-LATERAL LOAD V/S LATERAL DISPLACEMENT RELATIONSHIP OF L.B.E. AND S.G.P. WITH L.B.E.

CHAPTER V

TEST RESULTS AND DISCUSSION

5.1 Introduction

The sub-soil conditions and in-situ load test results of single and groups of granular piles (plain, individually skirted and collectively skirted) as detailed in Table 4.1, at different sites, are presented and discussed in this chapter. Also justification for various assumptions made in the proposed design methodology have been given. Comparision of ultimate loads (computed analytically) with and the experimental values has been made. The computed total settlement of plain and skirted granular piles have been compared with the actual insitu load tests data at various sites and discussed.

5.2 Sub Soil Conditions

As indicated earlier (Table 4.1), field studies were carried out at four different sites viz. (i) CBRI Officers' Hostel, Roorkee, UP (ii) CBRI Campus, Roorkee (iii) Molasses tank site Budaun (UP), (iv) Drilling Rig site, Silchar (Assam) and (v) Furnace Oil Tank site Haldia (W. Bengal).

5.2.1 CBRI Officers's Hostel Site (Roorkee)

Fig. 5.1 shows the borelog upto 10.5m depth, variation of N_s values and cone penetration values with depth. The soil has been classified as Indian Standard (IS: 1498-1970). The sub-soil investigations data reveal that the soil profile at this site consists of sandy silt and silty soil deposit (SM-ML) upto 2m depth with an average standard penetration N_s value of 4. This was followed by a ML-CL layer upto a depth of 6m with N_s value

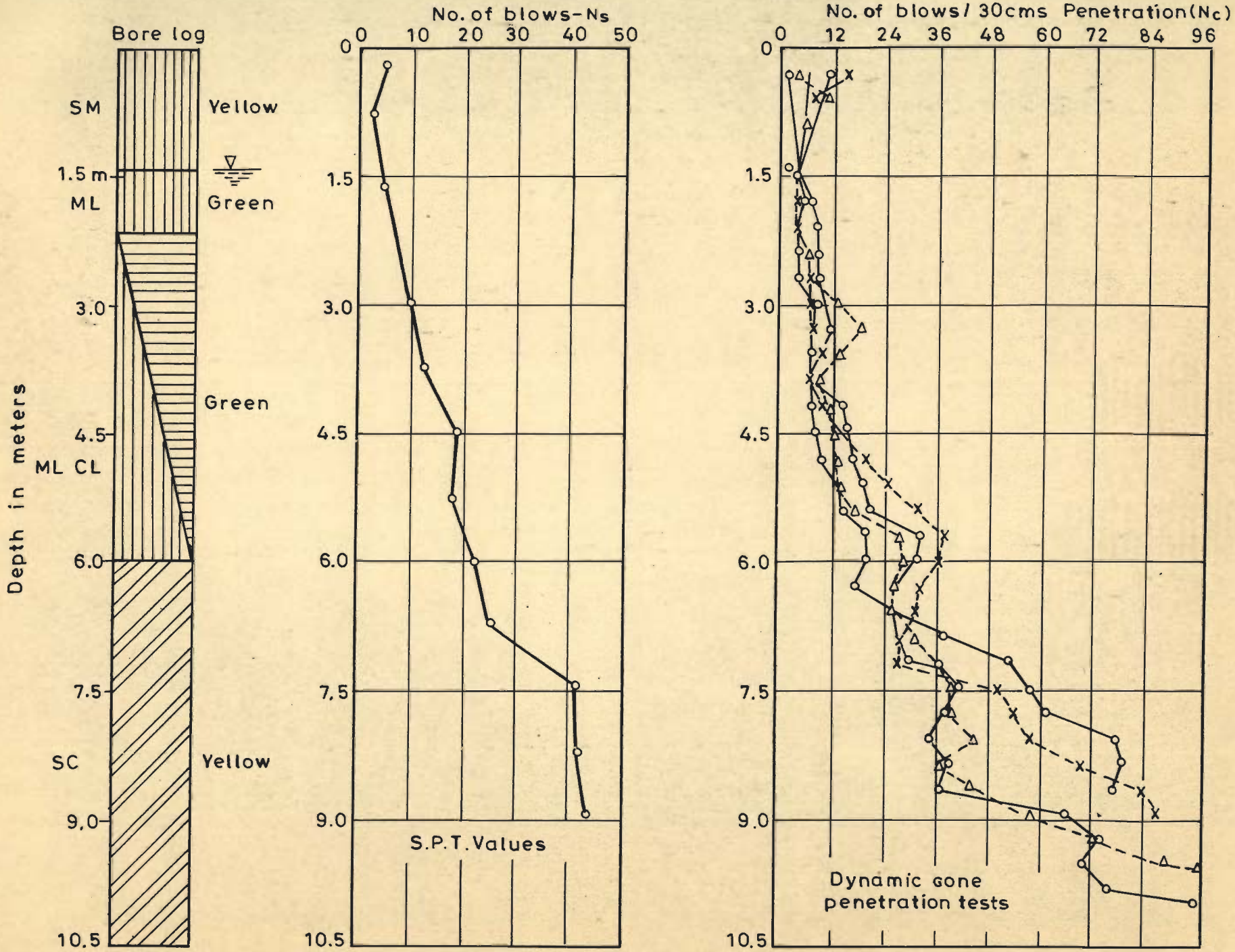


FIG.5.1. SOIL PROFILE AND FIELD TEST RESULTS (C.B.R.I. OFFICER'S HOSTEL SITE)

of 10 at 3m depth increasing to 19 at 4.5m and remaining constant at 20 upto 6m depth. Between 6m to 10.5m depth a yellowish sandy clay layer, (SC), was noted. The average N_s value between 6m to 6.0m and 6.75m to 7.5m was found as 22 and 32 beyond which it increased to about 40 or more. The sub-soil water level at the time of investigation (during the month of December) was found to be at 1.5m depth. The corresponding dynamic cone penetration test results indicated an average N_c value of 6 upto about 3m depth which increased to 12 to a depth of 4.5m. Beyond this depth the N_c values were found to show a consistant increase with about 30 at 7.5m depth and increasing to more than 100 at a depth of 10.5m.

The sub-soil conditions thus reveal that the soil deposit is predominantly cohesionless upto about 10.5m depth with loose upto about 3m depth and medium dense between 3 to 7m. The angle of shearing resistance as inferred from penetration test results upto a depth of 3m was found as 30°. The value also corroborated by isotropically consolidated drained triaxial tests on undisturbed cylinderical soil samples.

5.22 CBRI Campus Site (Roorkee)

The soil investigation data for the CBRI site is given in Fig. 5.2. The soil profile consisted of soil deposit having predominantly sandy silt (SM) upto a depth of about 3.4m with an average N_s value of 4 at 0.5m depth decreasing to 3 at 1.5m and again increasing to 4 at 3m depth. This was followed by a CI layer upto 7.5m showing an increase in N_s value to about 8 at 7.5m depth. The corresponding static cone penetration resistance (Fig. 5.2c) at 0.5m depth was found to be as 20 kg/cm^2 which

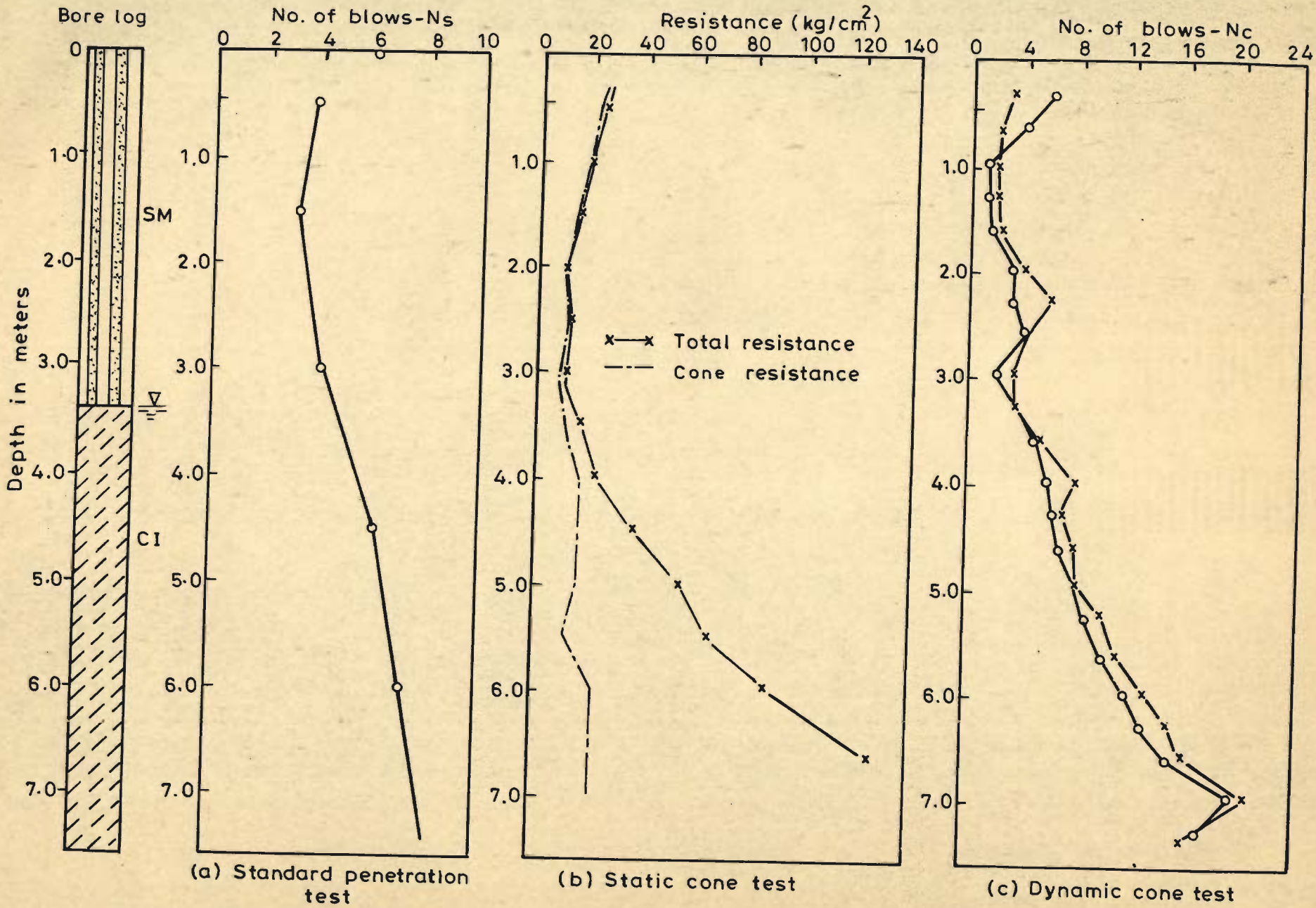


FIG.5.2. SOIL PROFILE AND FIELD TEST RESULTS (C.B.R.I. CAMPUS SITE)

decreased to 10 kg/cm^2 at a depth of 2m. The average static cone (point) resistance between 2 to 3m depth was noted to be 10 kg/cm^2 which again increased to 20 kg/cm^2 at 4m depth and remained constant upto a depth of 7.5m. The average dynamic cone penetration values, N_c was found to be 3 upto 2m depth and 4 betweeen 2 to 3.5m, beyond which it increased to 16 at 7m depth. The water table at the time of investigation (during the month of January) was noted to be at a depth of 3m. The angle of shearing resistance both inferred from standard penetration tests and isotropically drained triaxial tests indicated a value of 29°. The soil stratum is classified as predominantly loose cohesionless soil.

5.2.3 Molasses Tank Site (Budaun)

Fig. 5.3 shows the borehole and penetration tests data from the molasses tank site at Budaun, UP. The data reveals a sandy clay (SC) layer upto a depth of 3.5m with an average standard penetration value N_s equal to 7. This was followed by a soil deposit of poorly graded sand and sandy silt strata (SP-SM) with an average SPT value N_s of 9. The N_s value was noted to increase to 10 at 3.5m depth decreasing to 9 at 5m and again increasing to 20 at 6.5m. Further extension of the bore hole revealed the presence of a poorly graded sandy soil deposit (SP) between 6.5m to 12m with a decreasing N_s values from 20 at 6.5m depth to 12 at 11m depth. The dynamic cone penetration tests (Fig. 5.3 c and d) confirmed in general the strata and the strength indicating a loose strata upto 3m depth and slightly better between 3m to 5m with an average N_c value of 18. Further beyond 5m and upto 8m depth, an average N_c value of 26 was found which increased to 30 at 12m depth. The water table during the month of February was

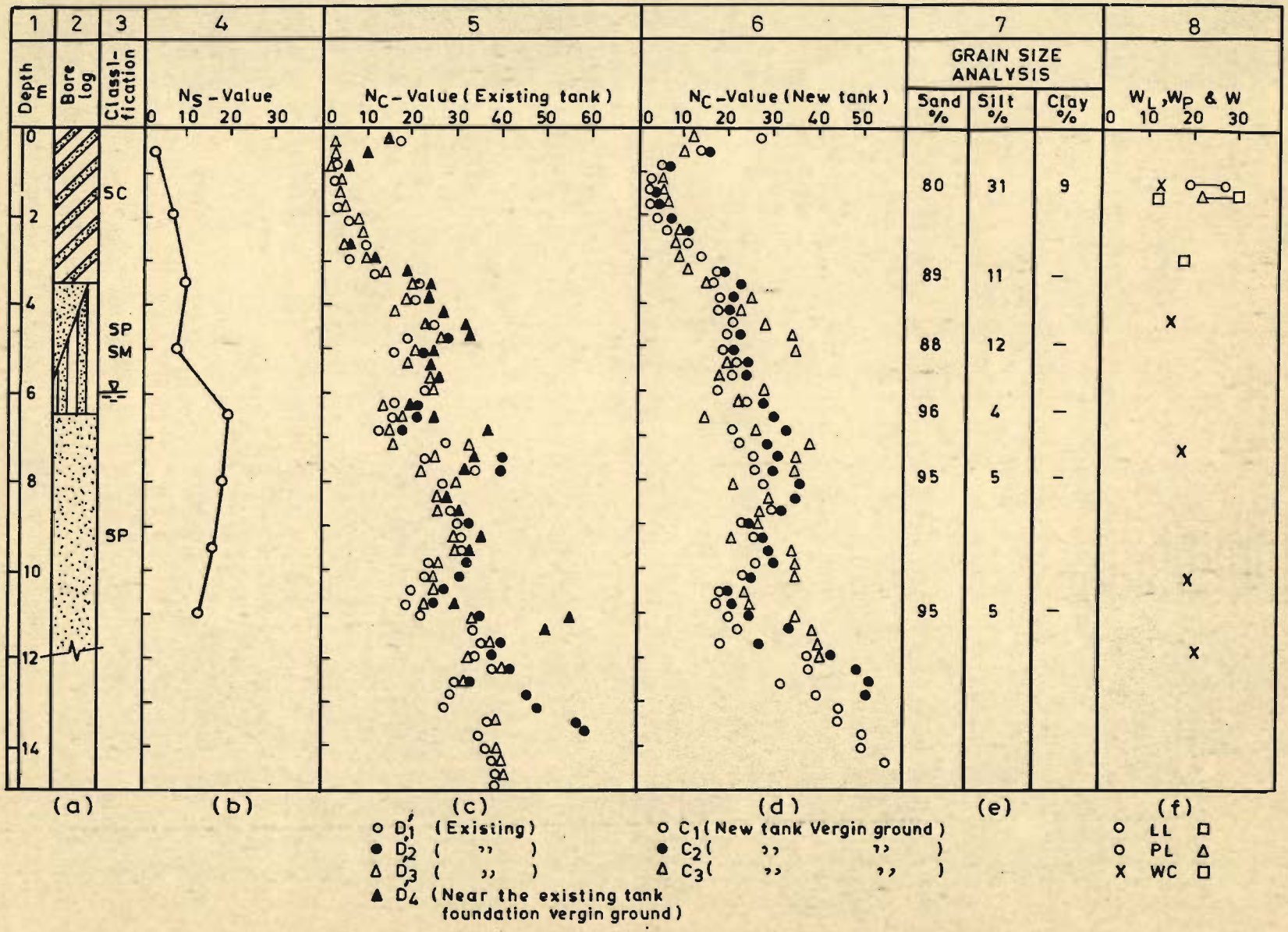


FIG.5.3. BORE HOLE AND SUB-SOIL CHARACTERISTICS (BUDAUN SITE)

noted at 6m depth.

5.2.4 Drilling Rig Site Silchar (Assam)

The location of two boreholes and dynamic cone penetration tests for the drilling rig site at Silchar (Assam) is shown in Fig. 5.4. The site plan also indicate the position of pilot 2 pile groups collectively skirted and a single plain pile 60cm in dia. 9m deep for insitu load testing. The bore log and the standard penetration test value with depth is shown in Fig. 5.5. The dynamic cone penetration test results in virgin soil at four locations is given in Fig. 5.6.

The location of the water table at the time of investigation (during the month of March) was found at 1.6m and 1.7m below ground level in boreholes 1 and 2 respectively. The bore hole data in general reveals a clayey silt (ML) layer upto a depth of about 2m, followed by 8m thick deposit of grey bluish with brown silt intrusions and pocket of sand (ML). Beyond this depth a 0.6-0.7m thick layer of grey sandy silt (SM) is noticed. Further extension of the boreholes demonstrated the presence of stiff grey clayey silt (ML) layer with sand. This continued upto a depth of 13.75m. The standard penetration test N_s values in borehole 1 from a depth of 1.75m to 5m showed a decreasing trend from 8 to 4, and then increasing from 4 to 20 at a depth of 11.3m beyond which it was found to remain constant. However, in borehole 2, the standard penetration N_s values were found to be slightly higher in comparision to borehole 1. The bulk density was found to vary from 1.90 to 2.07 g/cc while the plasticity index varied between 8.5 to 14.5 per cent. The undrained shear strength of the deposit varied from 0.48 kg/cm^2 to 0.51kg/cm^2. The compressibilities ($C_c/1+e_o$) value with depth was found to increase from

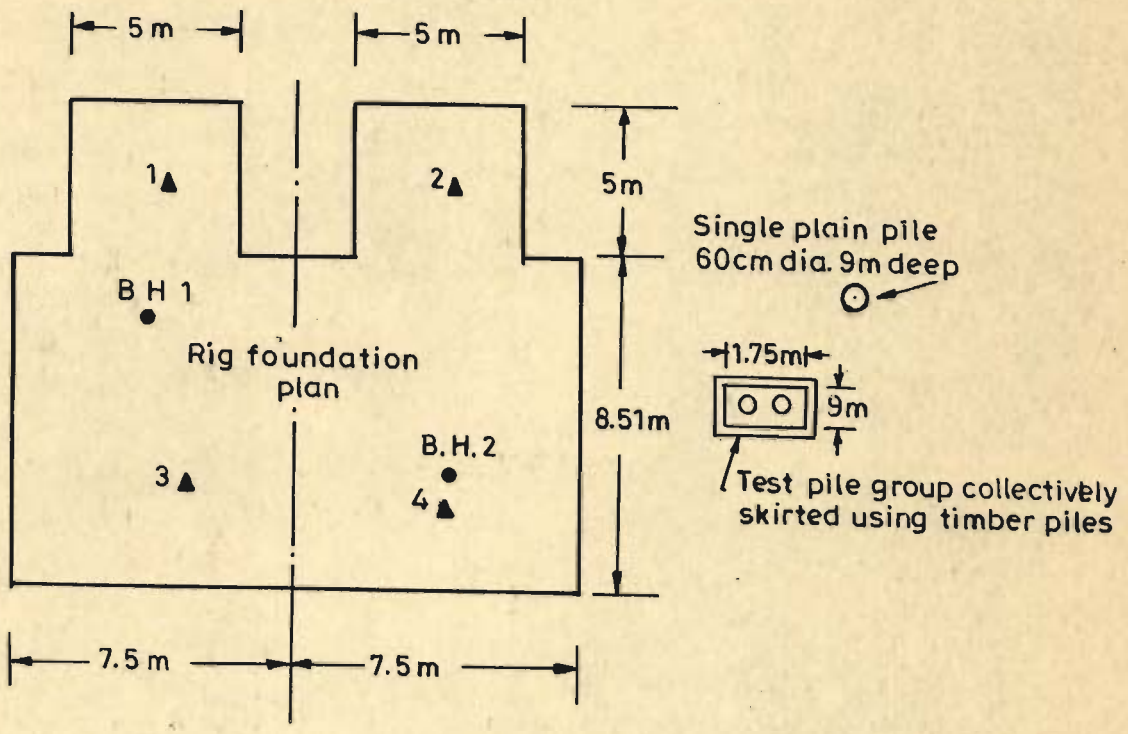


FIG.5.4. DRILLING RIG FOUNDATION PLAN SHOWING POSITION OF BORE HOLES AND DYNAMIC CONE PROBING LOCATIONS (SILCHAR)

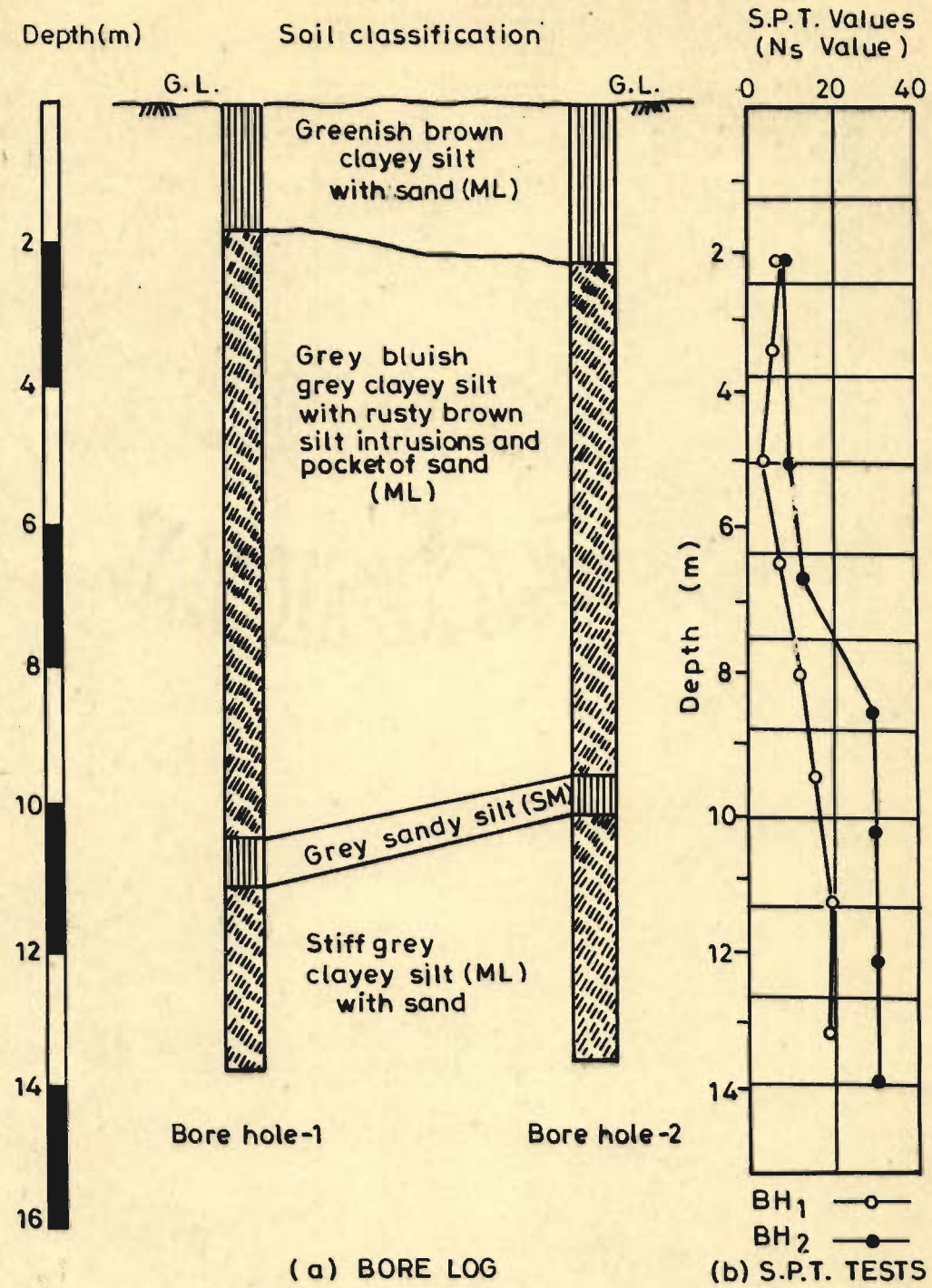


FIG.5.5. SUB-SOIL STRATUM WITH STANDARD PENETRATION TEST RESULTS DRILLING RIG SITE (SILCHAR)

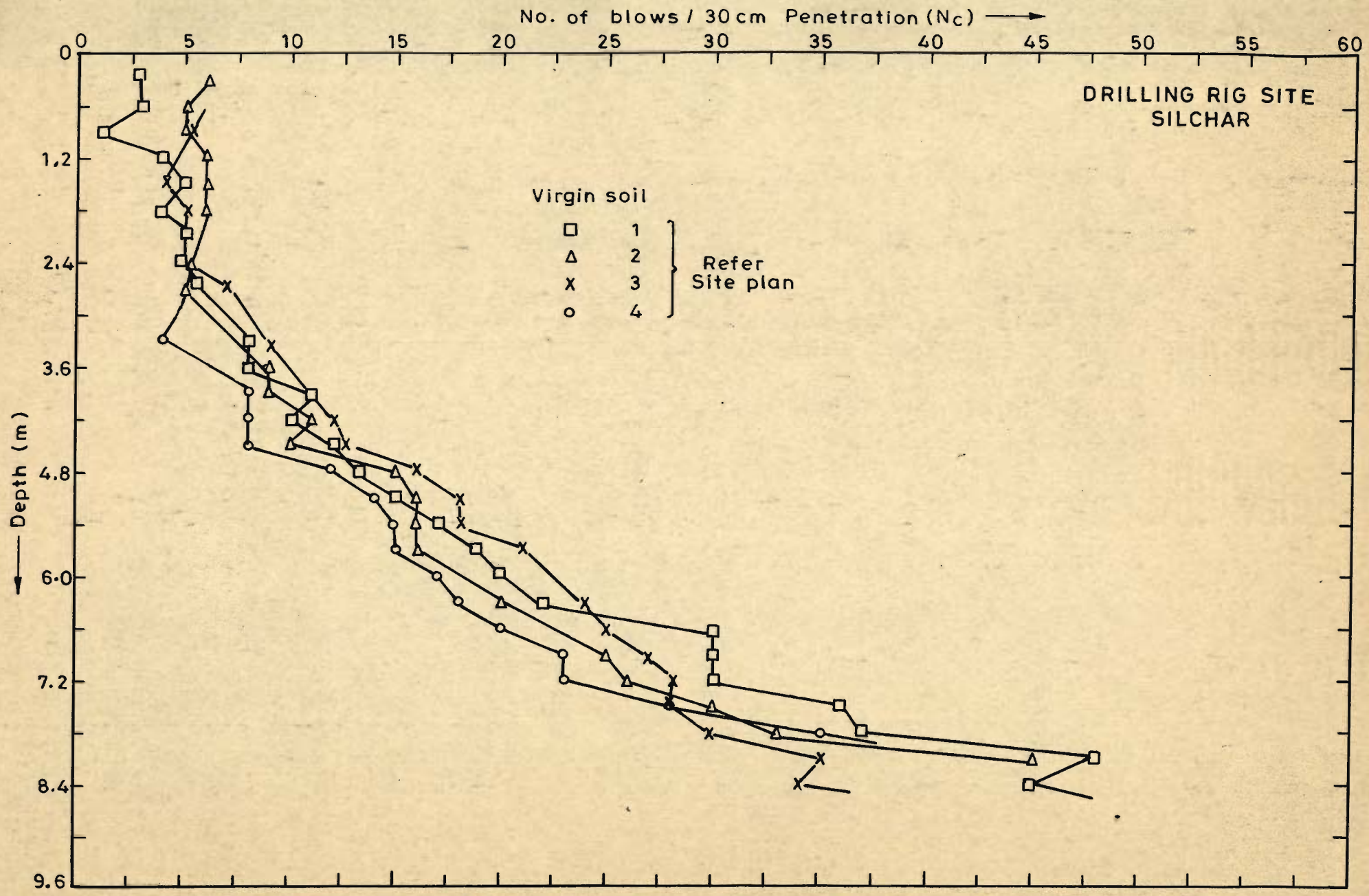


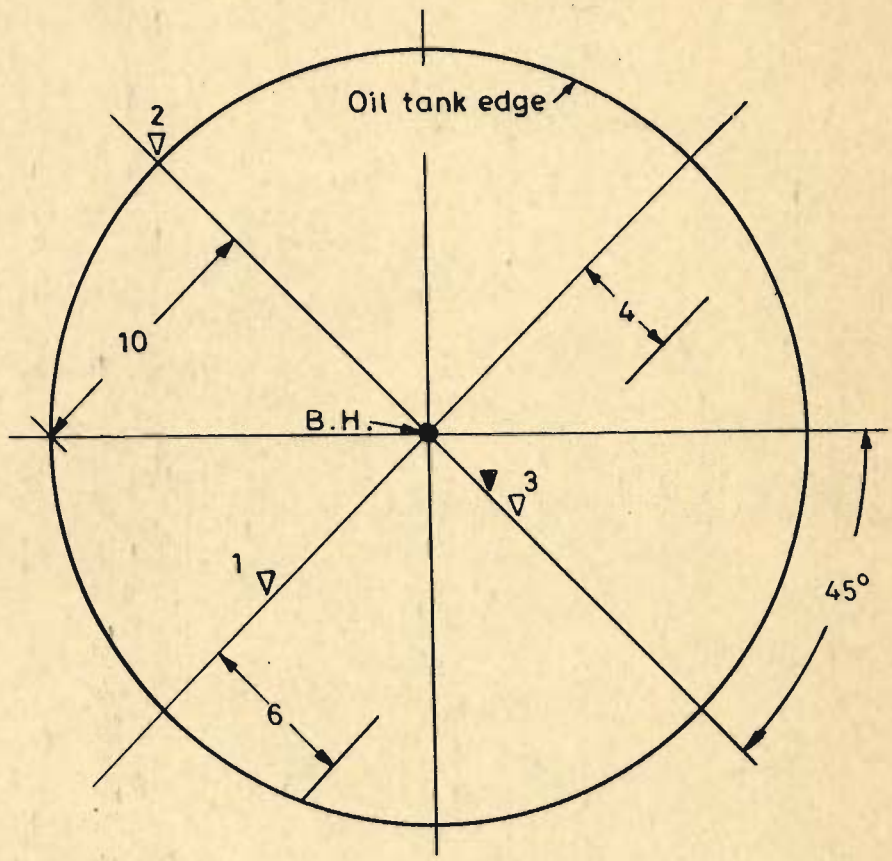
FIG.5.6. DYNAMIC CONE PENETRATION TEST RESULTS ON VIRGIN SOIL STRATUM

0.08 to 0.25 from 1.75m depth to 8.8m depth. Beyond this depth, the compressibility was noted to drop significantly reaching to a value of 0.1 only at 12m depth. In view of the above, soil stratum upto 12m depth is classified as compressible clayey silt cohesive deposit with low plasticity values having low to medium consistancy.

5.2.5 Furnace Oil Tank Site Haldia (W. Bengal)

Fig. 5.7 shows the location of borehole, dynamic and static cone penetration tests carried out at the Furnace oil tank site. The bore log, standard penetration test, static and dynamic cone penetration test results are shown in Figs. 5.8 through 5.10. The bore log, Fig. 5.8 indicates predominance of cohesive soil deposit having clay of low to high compressibilities upto a depth of 30m. The standard penetration resistance N_s values upto 9m depth average out to be equal to 6 which increased from 6 to 12 between a depth of 9m to 12m. Beyond this depth the value decreased slightly to an average value of 10 number of blows upto 20m depth. The N_s value was found to increase to 12 number of blows.

Comparision of the three dynamic cone penetration test results (Fig. 5.9) confirm the similarity of the strata at the site. It was further indicated that the dynamic cone penetration test resistance N_c values was found to be an average of 5 number of blows upto 3m depth and attained a value equal to 15 numbers of blows at 6m depth. The N_c values from 7m depth increased steadily and reached a value of 20 number of blows at 10m depth. Further penetration of the cone demonstrated the presence of harder strata corroborated by a high N_c - value , equal to 42 number



▽ Dynamic cone penetration test

▼ Static cone penetration test

● Bore hole

FIG.5.7- FIELD TEST LOCATIONS AT F.O.T. SITE HALDIA

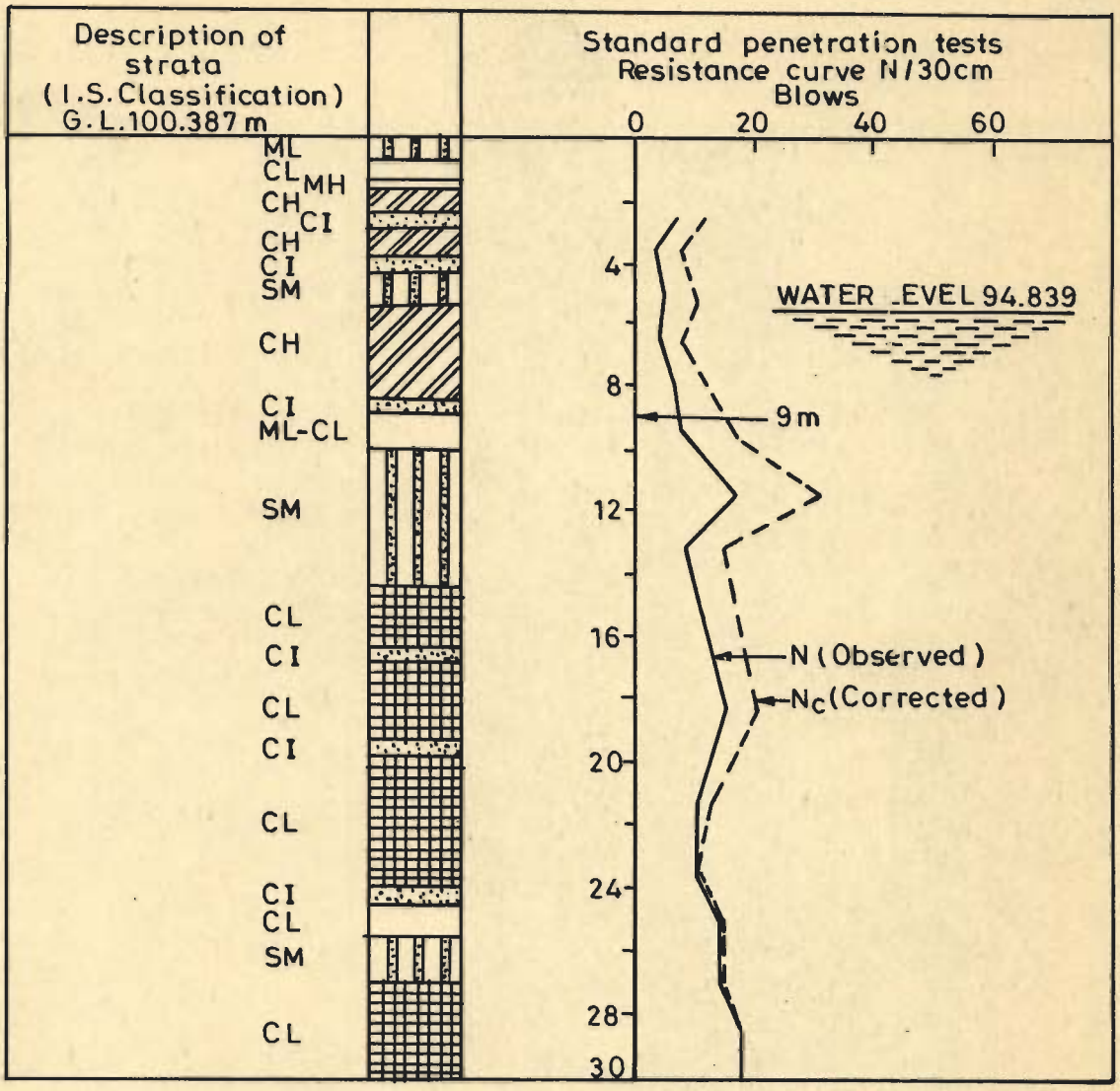


FIG.5.8. LOG OF BORE HOLE NO.1 AND STANDARD PENETRATION RESISTANCE CURVE (F.O.T. SITE HALDIA)

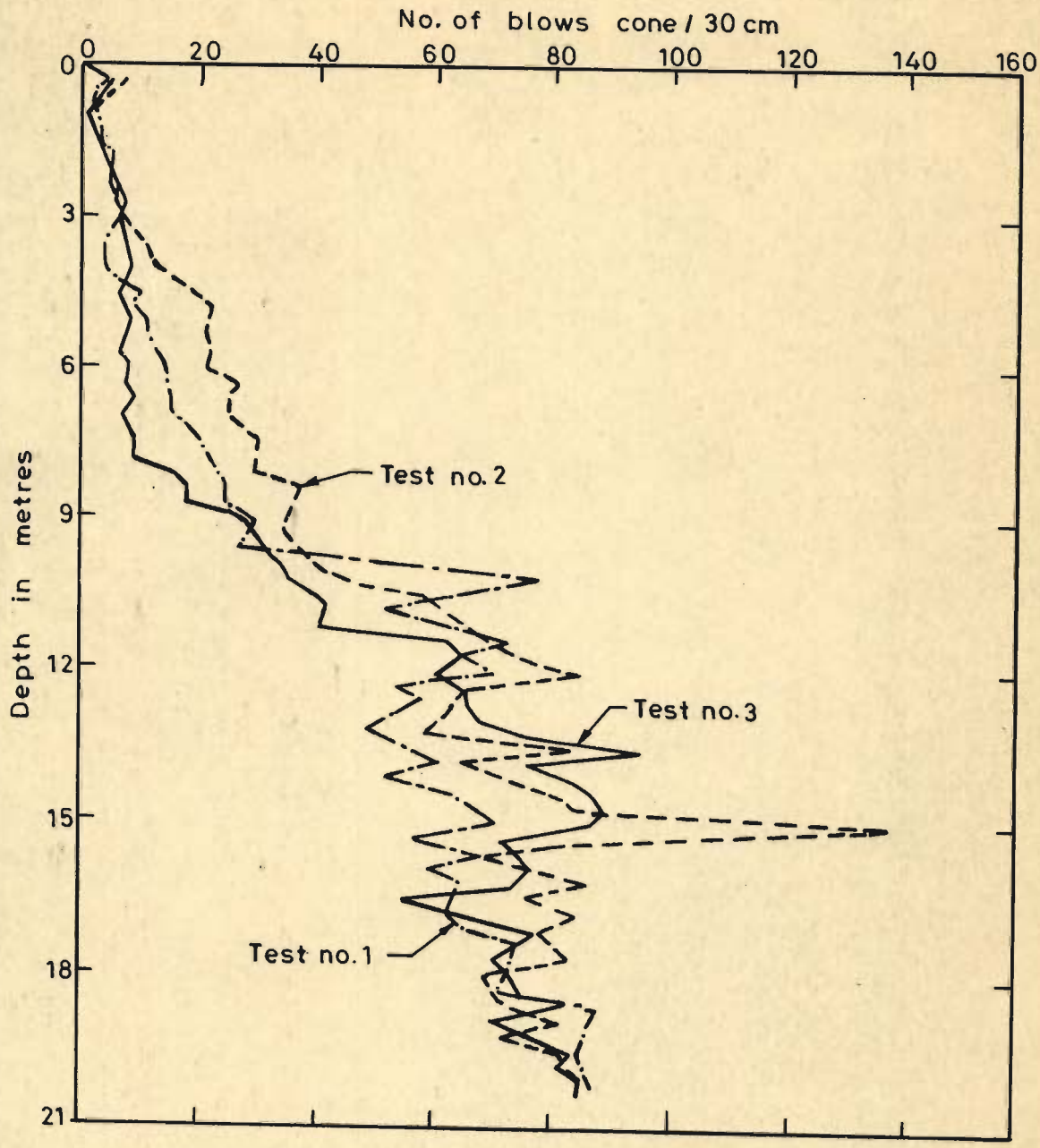


FIG.5.9. DYNAMIC CONE RESISTANCE CURVE
(LOCATION FOT. SITE HALDIA)

of blows at 12.5m depth. Beyond this depth the N_c value was observed to increase significantly. An average N_c values greater than 60 at 20m depth was observed. The static cone penetration resistance q_c values were found to vary between 12 to 25 kg/cm^2 upto 9.6m depth (Fig. 5.10). Beyond this depth the static cone resistance was found to increase significantly and attained a value greater than 100 kg/cm^2. The soil at the site thus indicated fully saturated sub-soil deposit with variation of bulk density from 1.56 to 2.06 gm per cc. The verage undrained shear strength was found to be as 0.27 kg/cm^2. The data thus indicated the presence of soft saturated cohesive subsoil deposit with high compressibility.

5.3 Failure Mode of Piles

The granular piles are noted to follow the bulging failure modes when loaded to their ultimate capacity. A 2 pile group 25cm in dia. 3.5m long was exhumed after load testing at CBRI officers's hostel site by carefully removing the soil from the side Fig. 4.5. The diameter of the bulged pile at depths 30cm, 50cm and 100cm below pile top were noted as 30cm, 34cm and 32cm respectively. Observation were also made on two similar pile group individually skirted as shown in Fig. 4.7. It was noted that the diameter of the bulged pile from immediately below the skirt was 35cm and was practically uniform upto about 75cm below the depth of skirt. Excavation beyond 75cm below the base of skirt was not possible due to the presence of the water table. Hughes and Withers (1974) also observed the bulging failure mode in the laboratory by loading a 38mm diameter sand pile installed in Kaolin clay (Fig. 2.2c). Mokhashi et.al (1976) also confirmed such a failure mode by laboratory model tests where bees-wax was

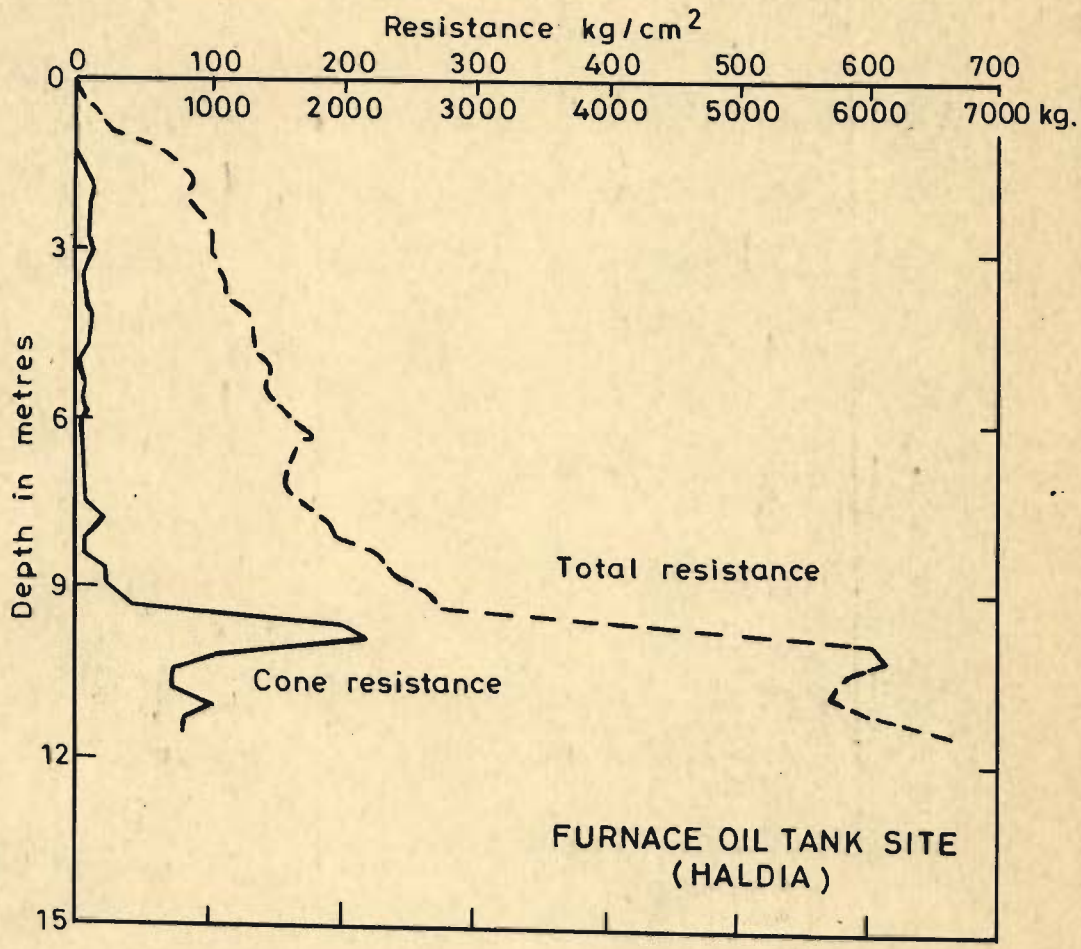


FIG.5.10. STATIC CONE PENETRATION CURVES

used as clay and sand as granular pile. Similar deformed shape of granular pile was also noted by Hughes Withers and Greenwood (1975) after excavating an actual pile (Fig. 2.2.d). Study of the shape of the pile revealed that the upper meter or two was often found to shape like a bucket resting on a cylinderical granular pile stem.

5.4 Critical Pile Length

Critical pile length, L_c which is depth of bulging is assumed to be equal to 4-5 times the installed pile diameter.

Figs. 3.1a and 3.1b show typical piles plain and individually skirted after load testing. For the installed pile dia. of 25cm the critical length of piles were noted to be as 102cm in the case of plain pile from the pile top. Wherein in case of individually skirted pile, it was found to be as 117cm. This indicates that the depth of the bulging extends to about 4-5 times the pile diameter. Theoretically for a granular pile installed in clay, Hughes and Withers (1974) have shown L_c/D ratio as 4.1. They also indicated that if the length of the pile is less than the critical length and the stress at the base exceeds the bearing capacity of the clay (q_c), the pile will fail in bearing. Williams (1969) stated that under typical loading conditions the pile will be weakest at a depth equal to 1.9 to 2.33 times its diameter, under triaxial equal all round stress. In the model study this value was found to 2.85 times the pile diameter by Mokhashi et.al (1976). Beyond this depth the pile was not found to be stressed.

5.5 The Influence of Pile Characteristics

The important pile characteristics influencing the behaviour of pile are: pile diameter, pile length, pile material and spacing

between piles in the group.

5.5.1 Influence of Pile Length and Pile Diameter

Unlike concrete piles the ultimate load bearing capacity of granular pile is not dependent on the length of the pile as the pile fails in bulging and the stresses in the pile are not transferred beyond the critical length of the pile at failure. Mokhashi et.al (1976) carried out model studies with partially penetrating piles with depth of piles equal to 33, 50, 75 and 100 per cent of the full pile lengths in soft clay deposit (Fig. 2.15). The test results demonstrated that the stress-deformation behaviour was independent of the pile length. However, the influence of pile penetration on the settlement is found to be significant for soil deposits having pile modulus E'_p and soil modulus E'_s constnat with depth Fig. 2.27 (Balaam et.al 1977). Similar trend was also observed in soil with modulii varying with depth (Fig. 2.28). Green Wood (1970) also reported that the depth of granular pile is generally limited to depth of the compact strata at shallow depth otherwise determined by the depth of the pressure bulb predicted by elastic theory which is usually about twice the plan dimension of the footing. However, in case where rafts or closely spaced footings stress deeper zones, it is found in practice that the depth of the pile between 6 to 8m below foundation level may be adequate. This is found sufficient to develop arching in the treated zone to bridge loose pockets in untreated zone.

Based on the observations made with respect to bulging failure mode (art. 5.3) and critical pile length (art. 5.4) it is noted that the ultimate load carrying capacity of granular pile is not dependent on the pile length provided the length of the pile is not

less than the depth of significant stress (which is equal to twice the foundation width). Though large depth may be needed for reducing settlement in deep soft cohesive deposits.

The present day practice of installing the granular piles is by using vibroflot either by vibrofloatation technique in cohesionless soil or by vibro replacement method in cohesive soil (Greenwood 1970) The procedure is already known for its application and is a established method of granular pile installation (Boer and Greenwood 1967). The diameter of the vibroflot generally varies from 0.3m to 0.5m and thus the pile diameters varies between 0.6m (stiff soil) to 1.1m (very soft cohesive soil). With the same diameter of vibrofloat Floss (1979) reported pile diameters between 0.75m to 1.05m. Watson & Thorburn (1966), Engelhardt & Krisch (1977), thus reported that Softer the soil layer larger will be the pile diameter. This suggests that since the variation in pile diameters installed by vibroflot is not significant (0.75m to 1m) the study on influence of pile diameter on the behaviour of granular pile has not attracted the attention. However, it may be noted that utilising the proposed indeqimeous construction method (utilising auger and internal operating hammer art. 4.5), one has the flexibility in choosing the pile diameter suiting the design requirements.

With a view to examine the influence of pile diameter, piles with 25cm and 35cm diameter were tested at CBRI officers hostel site. The load settlement curves for single piles and group of two plain granular piles are shown in Fig. 5.11. The ultimate load defined as the load at which the load settlement curve goes into a steep straight tangent, for the piles, are given in Table 5.1.

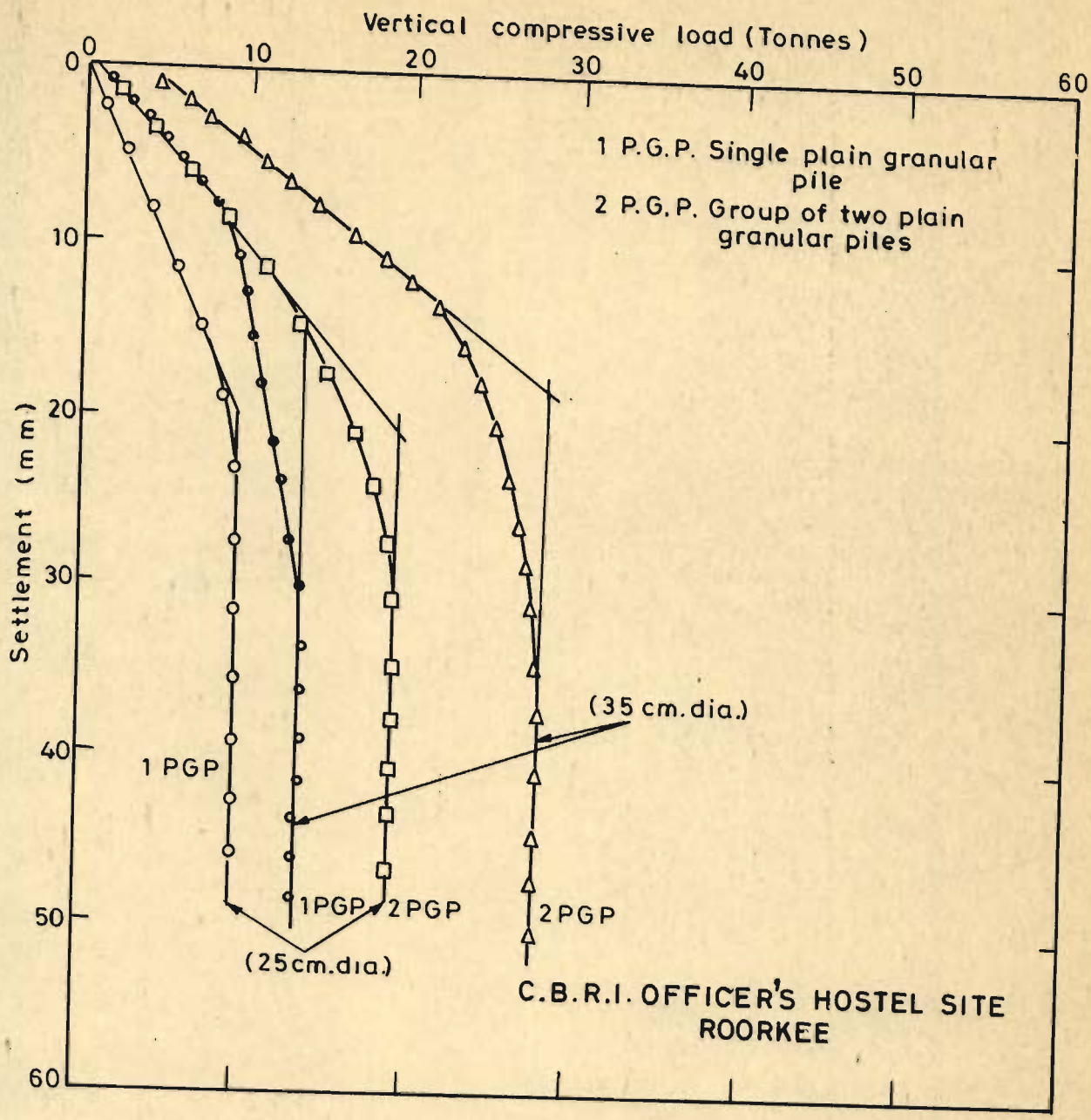


FIG.5.11–LOAD DEFORMATION BEHAVIOUR OF PLAIN GRANULAR PILE SHOWING INFLUENCE OF DIAMETER

Table 5.1 - Influence of Pile Dia. on Ultimate Load & Settlement at safe load of granular pile*

Pile Diameter (cm)	Pile length (m)	Ultimate load (tonnes)		Settlement (mm) corresponding to safe load (FS = 2)	
		Single pile	2 pile group	Single pile	2 pile group
25	3.5	9.5	19.0	9.5	10.0
35	8.0	14.0	28.0	7.0	8.25

*CBRI Officers Hostel Site

Since the ultimate load capacity of plain granular pile is independent of the pile length beyond critical length (art. 5.5.1), comparision of ultimate loads for 25 and 35cm dia. piles are shown in Table 5.1. It is observed that with the increase in pile diameter from 25 to 35cm (40 per cent), the ultimate load capacity in case of both single and group of 2 pile increases by 47 per cent which is found to be in direct proportion of per cent increase in pile diameter. However, no significant influence on the settlement at safe loads is noticed with increase of pile diameter for both single and group of 2 piles.

The limited test data indicates that with increase of pile diameter the load carrying capacity increases and settlement decreases. This is in order.

5.5.2 Granular Pile Spacing

Pile installed through vibrofloatation or vibrofot process, vary in dia. from 0.6 to 1.1m depending upon relative density in cohesionless soil deposit and consistency in cohesive deposit. These piles have been used in the past at a spacing of 1.65 to 2.73 times the pile dia. (Watt, Boer and Greenwood 1967, Engelhard and Krisch, 1977 Floss, 1979). The spacing of granular piles is generally determined

by settlement tolerances for the loads to be applied and to provide overlapping zones to cover a wide area of ground (Greenwood 1970). Pile spacing is also dependent on the degree of improvement required for providing a satisfactory foundation under the applied design load (Engelhardt and Krisch 1977). The settlement ratio of the reinforced ground and un-reinforced ground is a function of pile spacing and method of pile installation, Fig. 2.31. In a triangular pattern of pile installation, the zone of influence may be considered to have a hexagonal zone of influence Fig. 2.24, and an equal area of influence is given by a circular zone of dia. $d_e = 1.05\ s$, where s is the pile spacing. Balaam et.al (1977) have indicated on the basis of finite element analysis that significant reduction in settlement occurs only when the granular piles are closely spaced ($S \leqslant 5d$) and piles are installed to the full depth of consolidating layer. It was further indicated that too close spacing ($s/d \leqslant 2$) may not be feasible from construction point of view and also economy considerations and if the pile spacing is kept at s/d more than 4, the overall desired reduction in settlement may not be possible. Thus a pile spacing (s/d) between 2.5 to 4 may be adopted with reasonable accuracy. Also it has been recognised in practice that closer spacing are preferred under isolated footings than beneath large rafts (Greenwood 1970). Because of this fact in the present series of field tests relating to sites having cohesionless sub-soil deposits on all the three sites, a pile spacing (s/d) was kept as 3 except at one site (namely molasses tank site) where it was increased to 3.5 due to economy considerations.

5.5.3 Influence of Soil & Loading Characteristics & Pile Installation

Apart from the pile characteristics, the load-settlement behaviour of

piles are known to be significantly influenced by soil type such as cohesionless and cohesive deposits. In case of cohesionless soil, the important parameter is the relative density of the deposit whether the soil is loose, medium or dense. However, in the case of cohesive soil deposit, the consistency of the clay i.e. if the deposit is soft or stiff. These characteristics are known to have a significant influence on the pile behaviour in the case of rigid concrete pile. In the case of granular piles, however, these parameters are not likely to have significant influence due to the fact that the granular piles are installed only as a reinforcement in the loose soil or soft deposits to improve the relative density or the consistancy of the weak sub-soil stratum and as such these piles are only used in weak sub-soil deposits. In view of these facts, the present study was made in loose to medium dense cohesionless subsoil deposits at three sites and two sites having soft clayey silt and soft clay deposits, Table 4.1.

Different modes of load applications such as vertical compressive loading, vertical pull out loads and lateral loads or their combinations, influence the load displacement behaviour of piles. In the present test programme a detailed study of the behaviour of single and groups of 2, 3, 4 and 5 piles have been made under sustained vertical compressive and also under uplift loads. A few tests have been carried out under lateral load on skirted granular piles with load bearing element Table 4.1.

5.5.4 Group Efficiency of Plain Granular Piles

Ultimate load carrying capacity of the single and groups of 2, 3 and 4 plain granular piles at a spacing of 3d, (CBRI officers hostel site) have been obtained, (Table 5.2). Figs. 4.42 and 4.43 show the ultimate capacities of these single and groups of piles.

The group efficiency factor η has also been computed (Table 5.2a) col. 6. It is seen from the table that the value of η varies in a very narrow range from 0.92 to 1.0 in the case of both 25 and 35cm dia., 3.5m and 8m long plain granular piles. This seems to be in order because the installation of granular pile reinforces the deposit and improves the overall capacity. Thus for all practical purposes the group efficiency in case of plain granular piles at a spacing of three times the pile diameter is unity.

Table 5.2a - Group Efficiency Factor for Plain Granular Piles*

In situ test	Pile Dia (cm)	Pile length	Ultimate load observed (t)	Ultimate load of single pile in a group (col 4/ no. of pile) (t)	Group Efficiency Factor (η) (Col. 5 / U.L. of single pile)
1	2	3	4	5	6
Single pile	35	8	14.0	14.0	1.0
2 Pile Group	35	8	28.0	14.0	1.0
Single pile	25	3.5	9.5	9.5	1.0
2 Pile Group	25	3.5	19.0	9.5	0.894
3 Pile Group	25	3.5	26.5	8.83	0.92
4 Pile Group	25	3.5	35.0	8.75	0.921

*CBRI Officer's hostel site

5.5.5 Determination of the Elastic Modulus of the Pile Material

The stress-strain behaviour of the installed granular pile material consisting of stone metal (20-70mm size) with 20-30 per cent sand can not be easily assessed. Laboratory tests have little relevance owing to sample disturbances. Tests on remoulded samples are likely to yield unreliable results as the original arrangement of the stone metal/sand mixture as in the installed pile can not be achieved.

In view of these facts, best results may be obtained from in situ load testing on single/group of installed piles.

The elastic modulii E_p for the pile material have been computed back from the load-settlement curves of single and groups of granular pile installed at CBRI officers hostel site (SM group). Similar results from other two sites were also obtained. All these values have been shown in Table 5.2(b). For the computation of E_p values from the equivalent soil modulii E_{eq}; eqn. 3.78 (art. 3.11.1) was utilised.

It may be noted that the granular piles at CBRI officers hostel site were installed by using an internal operating hammer weighing 125 kg only. However at Drilling Rig site (ML group) and oil tank site (CH group), the weights of the hammers were 300 kg and 500 kg respectively. The study of the Table 5.2(b) indicates that the value of E_p for piles installed varies from 240 kg/cm^2 to 516 kg/cm^2 for a hammer weight of 125 kg. On other two sites, the corresponding values of E_p are 326 and 538 kg/cm^2 respectively for 300 kg and 500 kg hammer. It has been reported that the E_p values for vibro-floated stone columns based on large number of field tests, is found to be equal to 600 kp/cm^2 (600 kg/cm^2).

Incidentaly, the values of E_p for artificially prepared gravelly soil deposit computed from load test results of Mohan et.al (1971) are found to be 270 and 299.9 kg/cm^2. Also these values were obtained from in situ load tests on footings 90cmx90cm cast on gravelly soil deposit with crushed rock and sand as filler material which is found to vary between 306 kg/cm^2 to 474 kg/cm^2 (Rao et.al 1981). Similar values are also obtained from confined compression tests carried out on the same site Table 5.2 (b). These test results

Table 5.2(b) - Elastic Modulus (E_p) of the Installed Pile Material

Site	Test No.	Pile cap/ footing size (m)	Material type	Filler Material	Observed Modulii (E_p)-kg/cm^2 from			Source
					In situ load tests on footing	In situ compression Tests	In situ Load Tests on Granular piles	
1	2	3	4	5	6	7	8	9
CBRI Officers Hostel site (Roorkee)[1]	Single pile	0.4	Stone metal 18mm to 22mm	Solani sand	-	-	341.3	Present Investigation
	Group of 2	0.4	-do-	-do-	-	-	423.1	
	Group of 3	1.2 dia	-do-	-do-	-	-	240.7	
	Group of 4	1.2	-do-	-do-	-	-	516.1	
Drilling site Silchar (Assam)[2]	Single pile	0.75	Stone metal (20-70mm)	Local coarse sand	-	-	326.0	
Oil Tank site Haldia (W. Bengal)[3]	Group of 2	1.0	-do-	Midnapur sand	-	-	538.3	
CBRI Lab	A	0.45	Pure gravels less than 50mm	Nil	270.0*	-	-	Mohan et.al (1971)
CBRI Lab.	B	0.45	-do-	Nil	299.9*	-	-	-do-
Muni ka Reti site(Rishikesh)	F_1(a)	-	Gravel 20-50 mm in size	Coarse sand/ crushed stone	-	570.7	-	Rao et.al (1981)

Table 5.2 (b) (Contd..)

1	2	3	4	5	6	7	8	9
	(b)	0.9			350.9	462.96	-	
	(c)	-			-	312.01	-	
	F_2	0.9			373.0	384.02	-	
	F_3	0.9			460.0	328.08	-	
	F_5	0.9			474.6	540.5	-	
	F_6(a)	0.6			306.0	1000.0	-	
	F_7(a)	0.9			483.2	509.6	-	

*Load tests in Large Lab. Model

1 - Using 125 kg internal operating hammer

2 - Using 500 kg internal operating hammer

3 - Using 500 kg internal operating hammer

Note: Dia. of compression test ring was 30cm for all tests except in F_1(b,c) which was 45 & 60 cms.

further confirm the validity of the method of computation of E_p values from the insitu load test results of singl/group of granular piles. These data suggest that, it may be reasonable to accept the value of elastic modulus of pile material from 240 kg/cm^2 to 500 kg/cm^2 for hammers weighing between 125 kg to 500 kg.

5.5.6 Validity of Assumption of K_p value

In the present series of tests on single and groups of granular piles, two tests on single plain granular piles were carried out by loading the pile top only (free standing pile case). The value of (q_{ult}) was obtained from the load settlement curves.

On the other hand the value of limit stress σ_{rL} was computed using the soil parameters obtained from the laboratory and field tests utilising equation 3.53. The average value of the coefficient (K_p) which is the ratio (q_{ult}/σ_{rL}) was obtained. This average value of K_p was found to be 5.72 from these two tests. The value of K_p is also dependent on angle of shearing resistance (Ø') of granular pile material constituting the pile. The value of Ø' has been reported to vary from 38° (Engelhardt and Golding, 1975) to 45°, 1967) based on field and laboratory tests. For Ø' equal to 45°, the K_p value works out to be 5.89 (Broms 1979) which is in good agreement with the experimental value of 5.72 from the present tests. In view of this fact a value of 6 for K_p was adopted for further computation.

5.6 Plain and Skirted Granular Piles

5.6.1 Load Deformation Characteristics

The load-deformation characteristics as obtained from in situ load tests at CBRI officers Hostel site for plain and skirted granular pile, single and groups of 2, 3 and 4 is shown in Fig. 4.42.

load-settlement behaviour of the virgin soil obtained from in situ footing tests by testing plain footings are also shown on the same figure. The piles used in the test were 25cm dia. and 3.5m deep at a pile spacing of 3 times the pile diameter. The piles are plain granular piles as well as individually skirted granular piles. The corresponding stress deformation behaviour of virgin soil, plain granular piles, single and in groups of 2,3 and 4 and individually skirted piles are shown in Fig. 5.12. The stress-deformation curves of the virgin soil indicates a non-linear behaviour upto the ultimate load (defined earlier) and beyond this point P'P' (Fig. 5.12) a plastic behaviour. Such a behaviour is indicative of the presence of loose medium dense non-cohesive sub-soil deposit. On the contrary the stress-settlement characteristics of plain and skirted granular piles whether single or in groups of 2, 3 and 4 demonstrate a elastic behaviour indicated by point (E) in the initial stages of loading followed by an elasto-plastic behaviour (EP) and when the ultimate load is reached a completely plastic behaviour (point P, Fig. 5.12). Such a behaviour is also noticed in the case of 35cm dia. 8m deep plain single and 2 pile groups and collectively skirted single pile and groups of 2, 3 and 4 piles, Fig. 5.13. The corresponding load deformation curves are shown in Fig 5.14.& 4.43. This is further confirmed by the load-settlement behaviour of a 25cm dia. 3.5m deep collectively skirted pile group on the same site, (Fig. 5.15 to 5.17) and also from tests on the same group with a central unit pile collectively skirted, (Fig. 4.44 and Fig. 5.18). Similar observations have also been noted from the insitu load test results at Molasses tank site Budaun, Fig. 4.48 consisting of loose-medium non cohesive sub soil deposits, Drilling rig site, Silchar, (Fig. 5.19) consisting of deep layer of clayey silt deposit with tests on a group of 2 piles, 9m deep having 60cm dia. collectively skirted. Even at Furnace oil

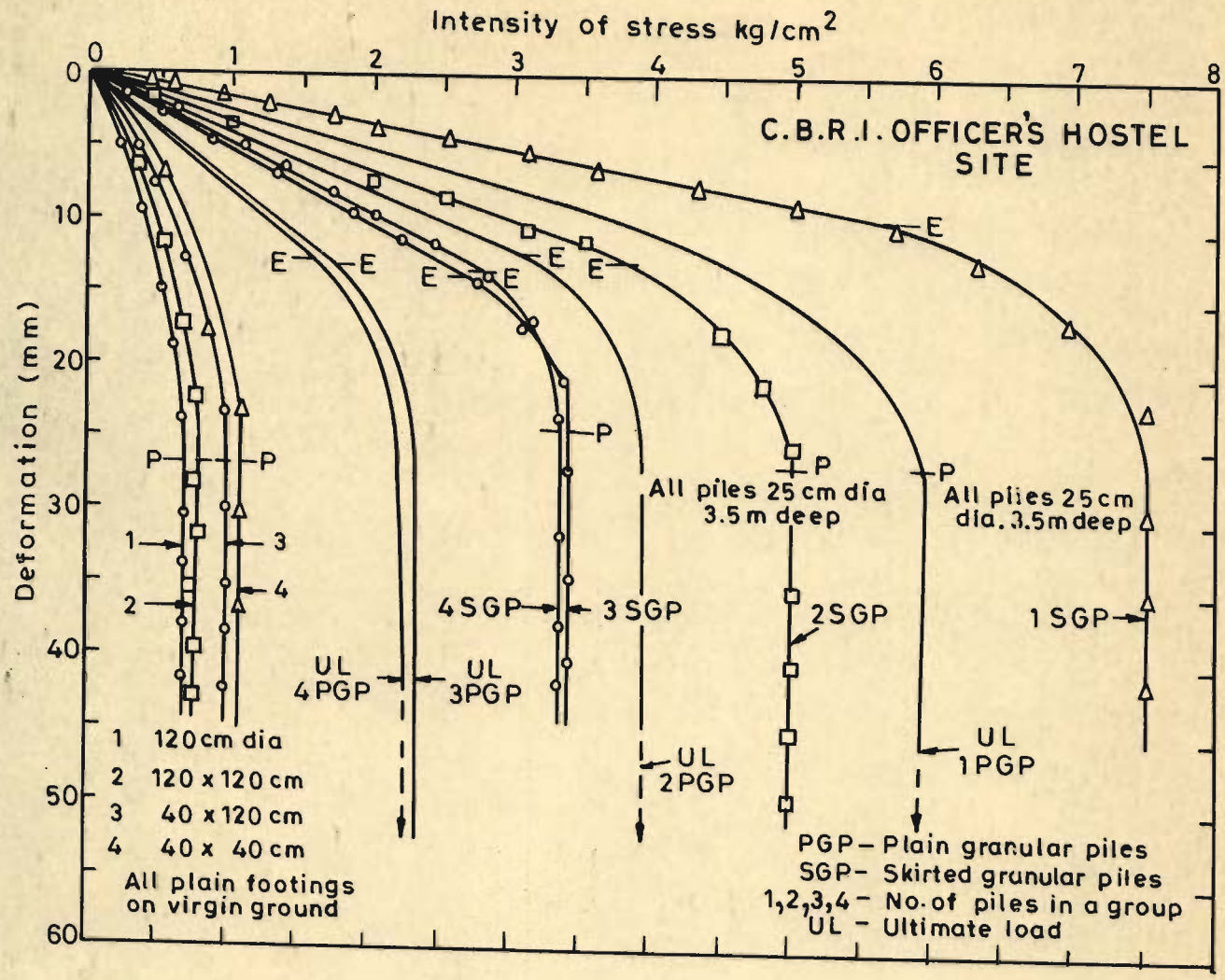


FIG.5.12- STRESS DEFORMATION BEHAVIOUR OF INDIVIDUALLY SKIRTED SINGLE AND GROUPS 2,3 & 4 PILES COMPARED WITH CORRESPONDING PLAIN GRANULAR PILES

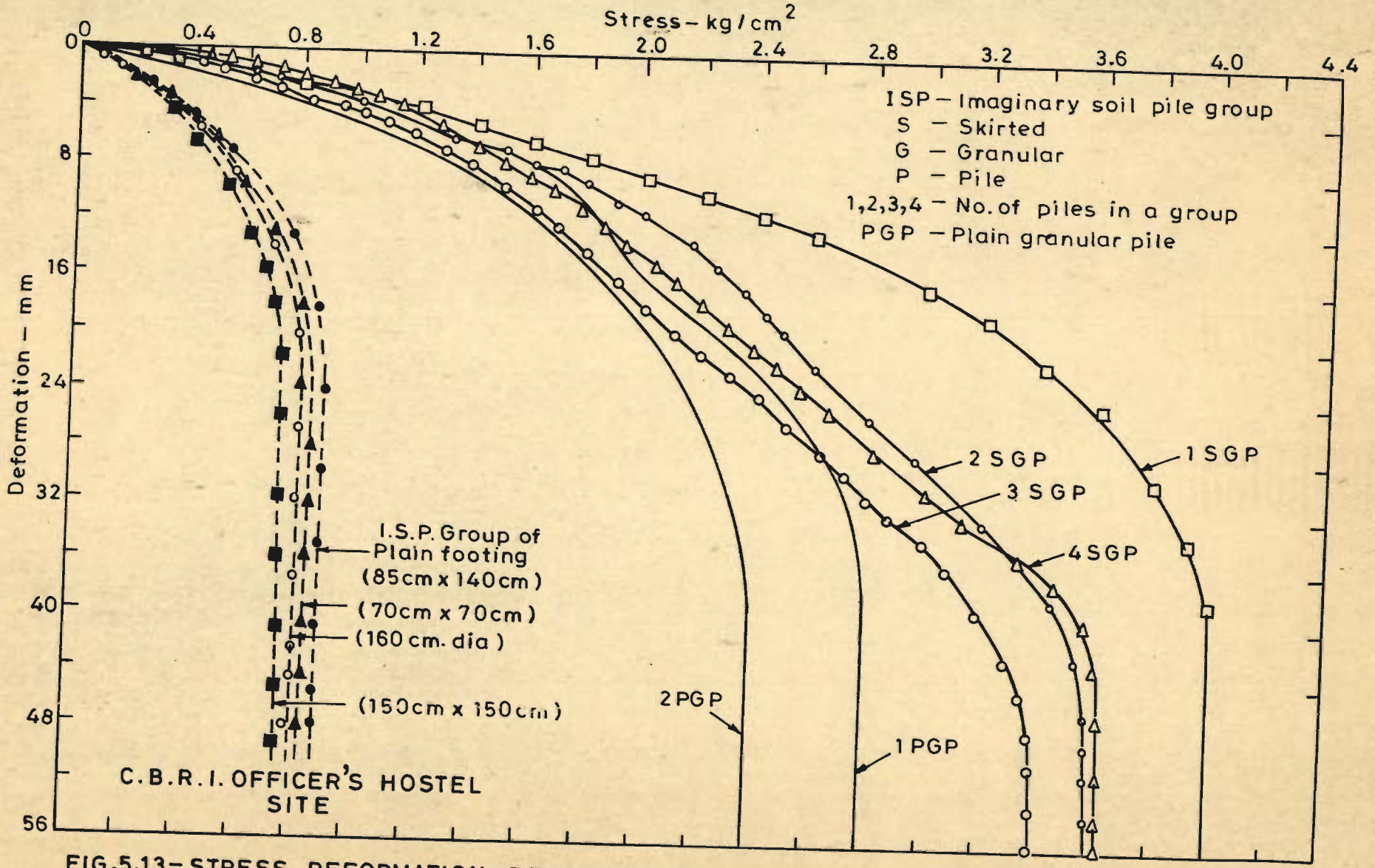


FIG.5.13 - STRESS DEFORMATION BEHAVIOUR OF COLLECTIVELY SKIRTED GRANULAR PILE FOUNDATION AND PLAIN FOOTING

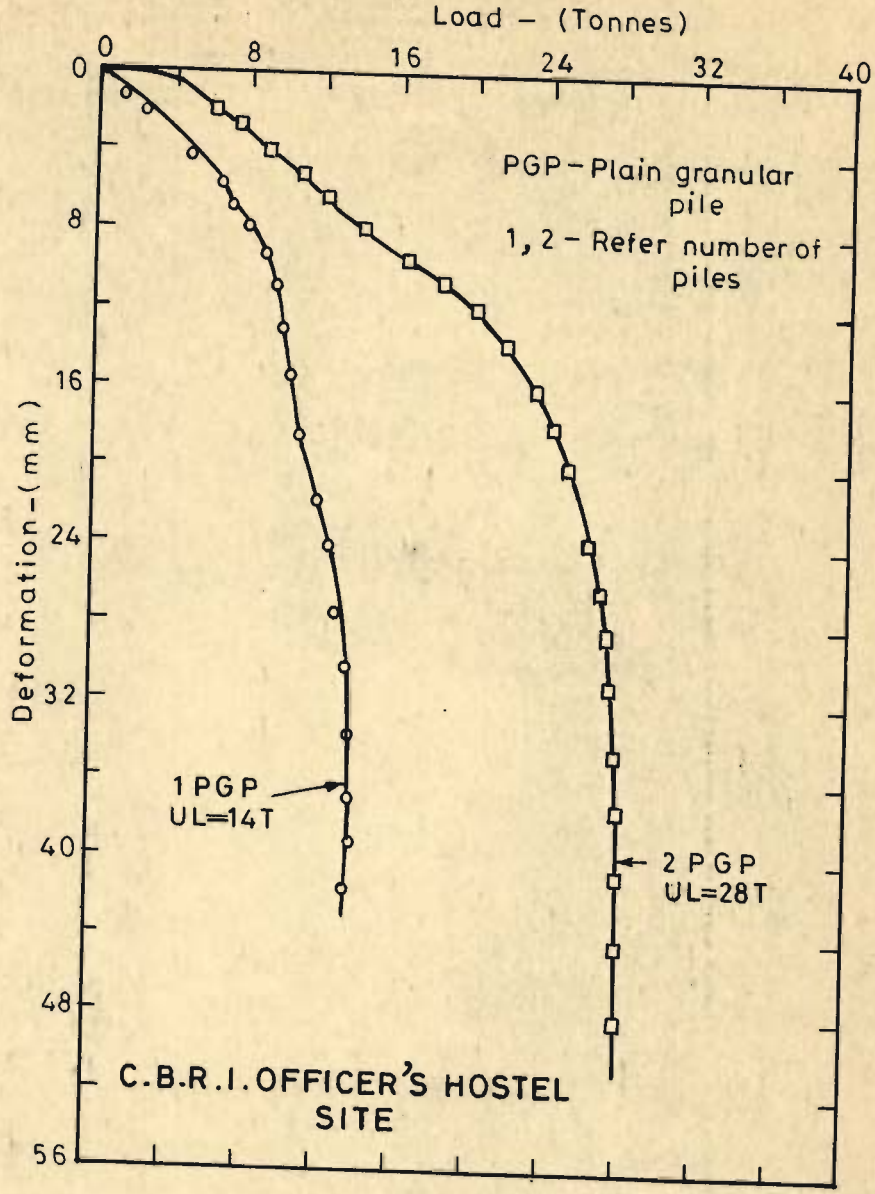


FIG.5.14-LOAD DEFORMATION BEHAVIOUR OF PLAIN GRANULAR PILES.

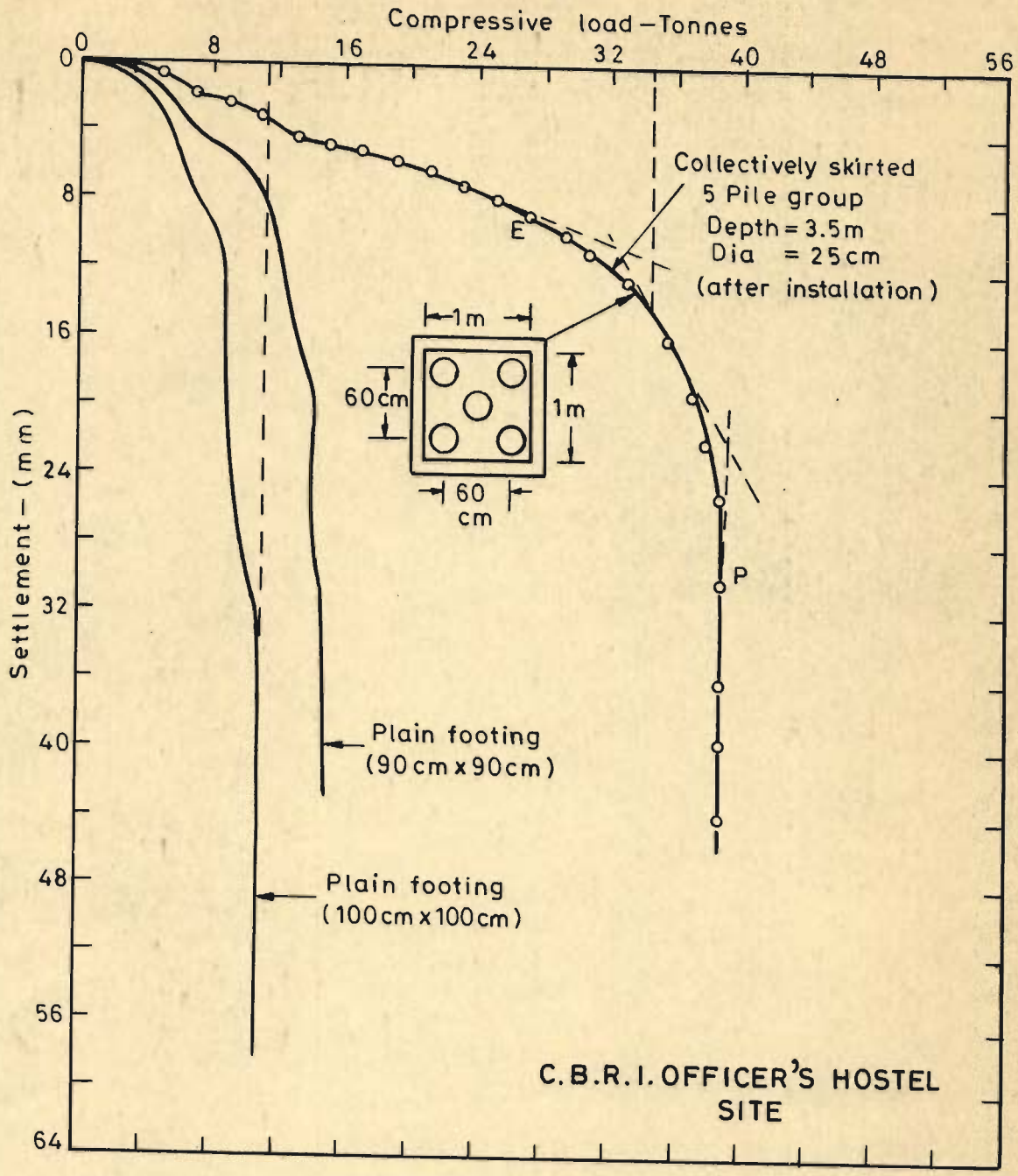


FIG.5.15 – STRESS DEFORMATION BEHAVIOUR OF SKIRTED GRANULAR PILE GROUP AND PLAIN FOOTING

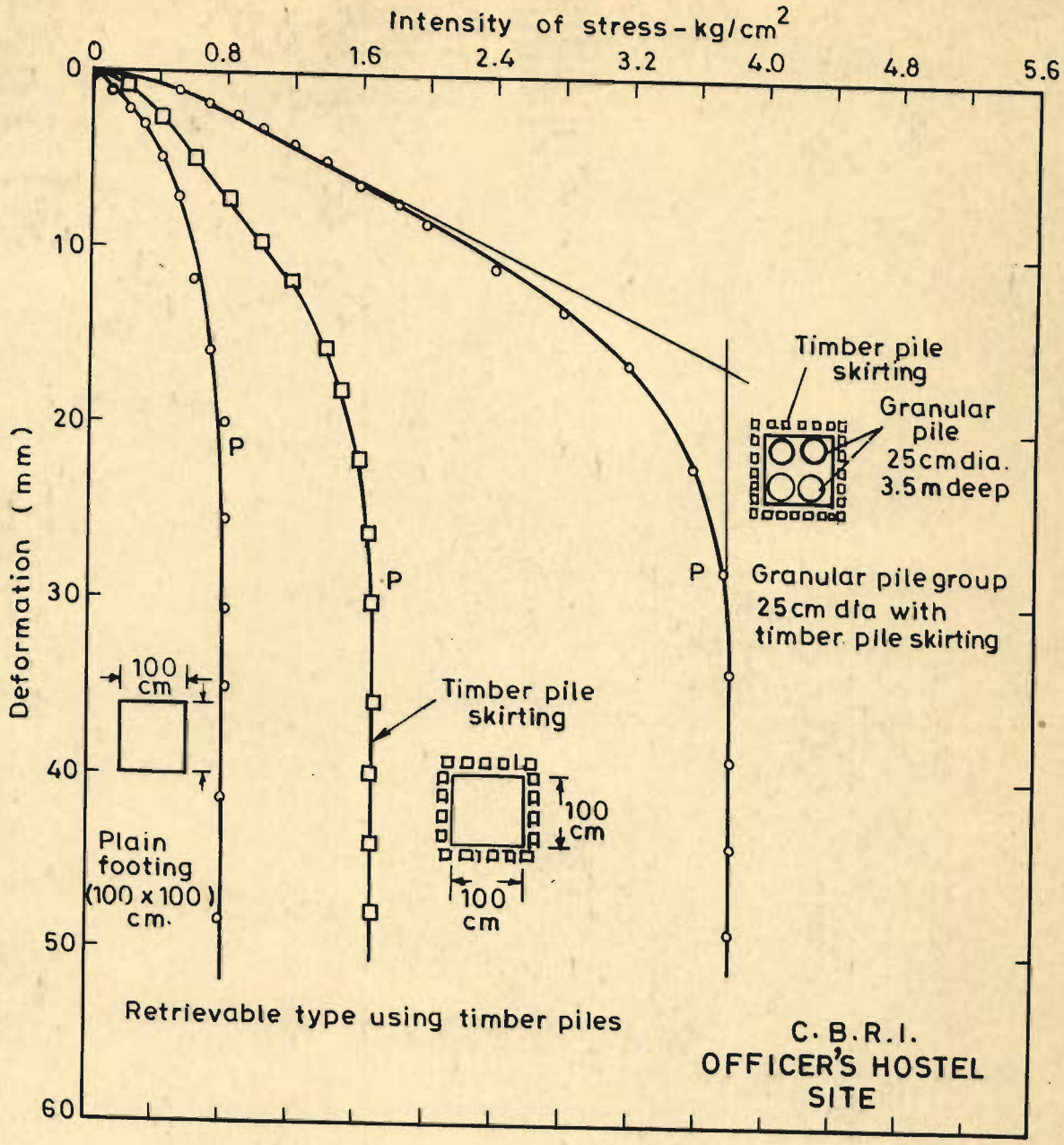


FIG.5.16 - STRESS DEFORMATION BEHAVIOUR OF PLAIN SKIRTED AND SKIRTED GRANULAR PILE FOUNDATION.

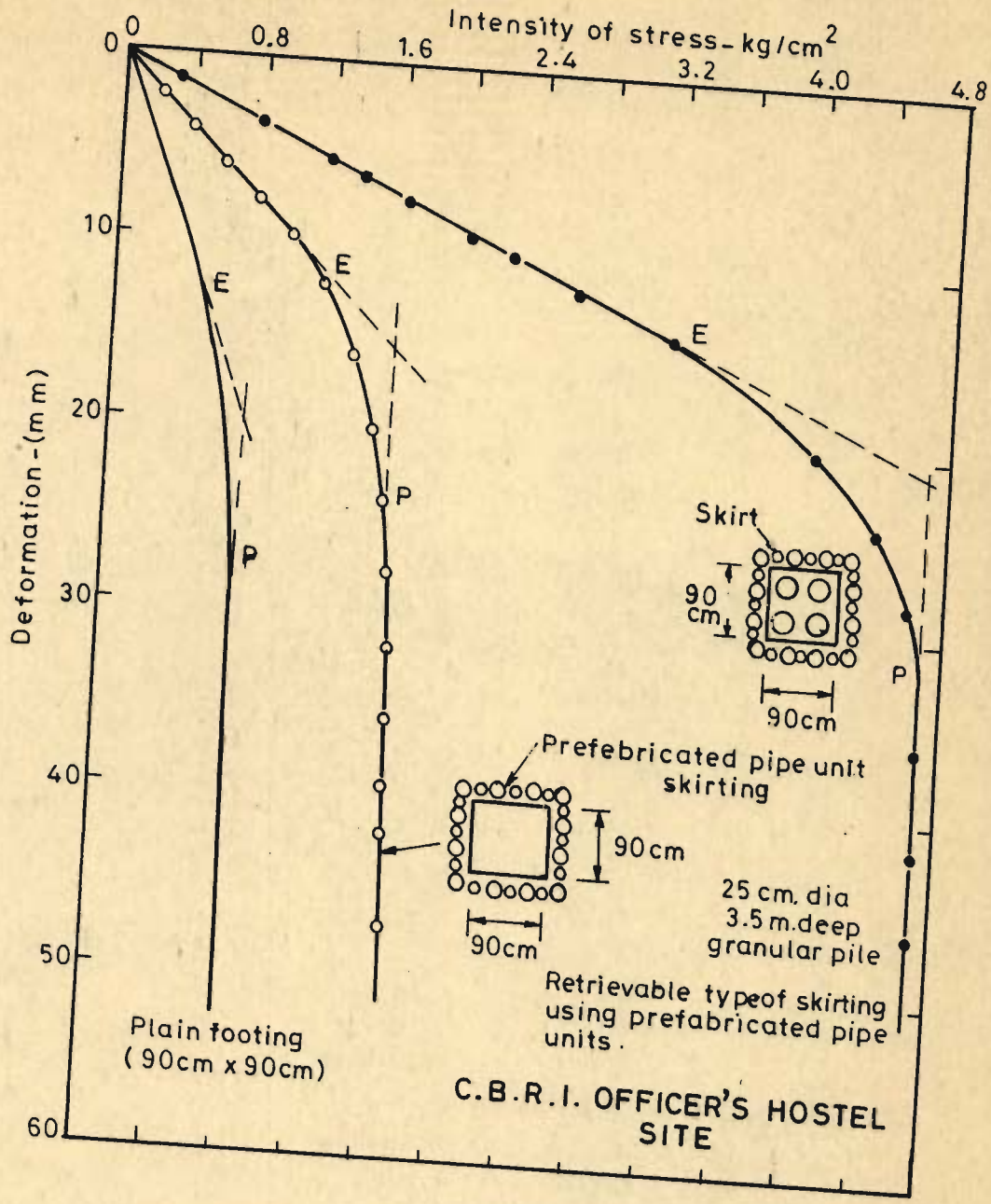


FIG.5.17– STRESS DEFORMATION BEHAVIOUR OF PLAIN FOOTING SKIRTED FOOTING AND SKIRTED GRANULAR PILE GROUP

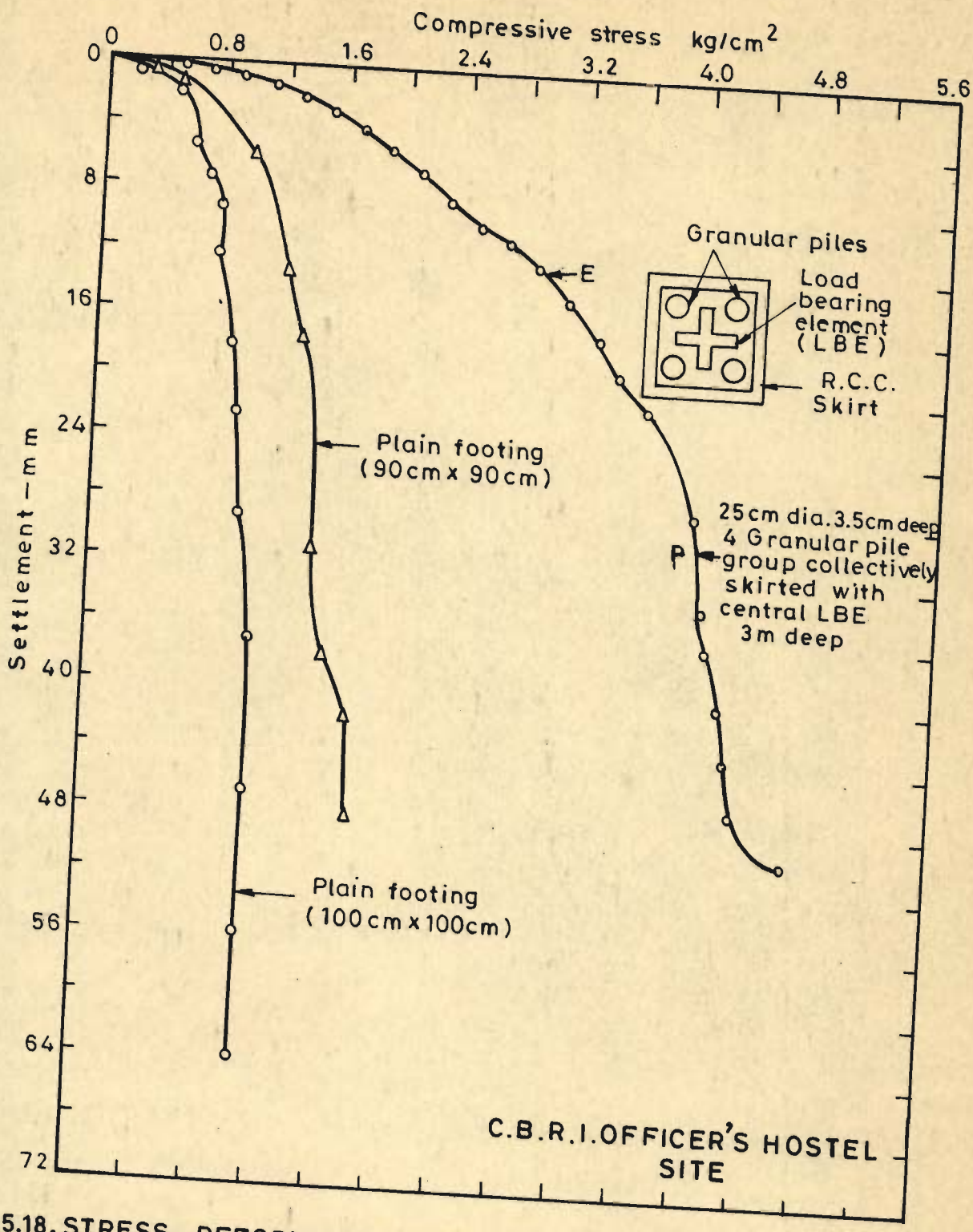


FIG.5.18. STRESS DEFORMATION BEHAVIOUR OF S.G.P. FOUNDATION WITH L.B.E. AND PLAIN FOOTING UNDER COMPRESSIVE LOAD

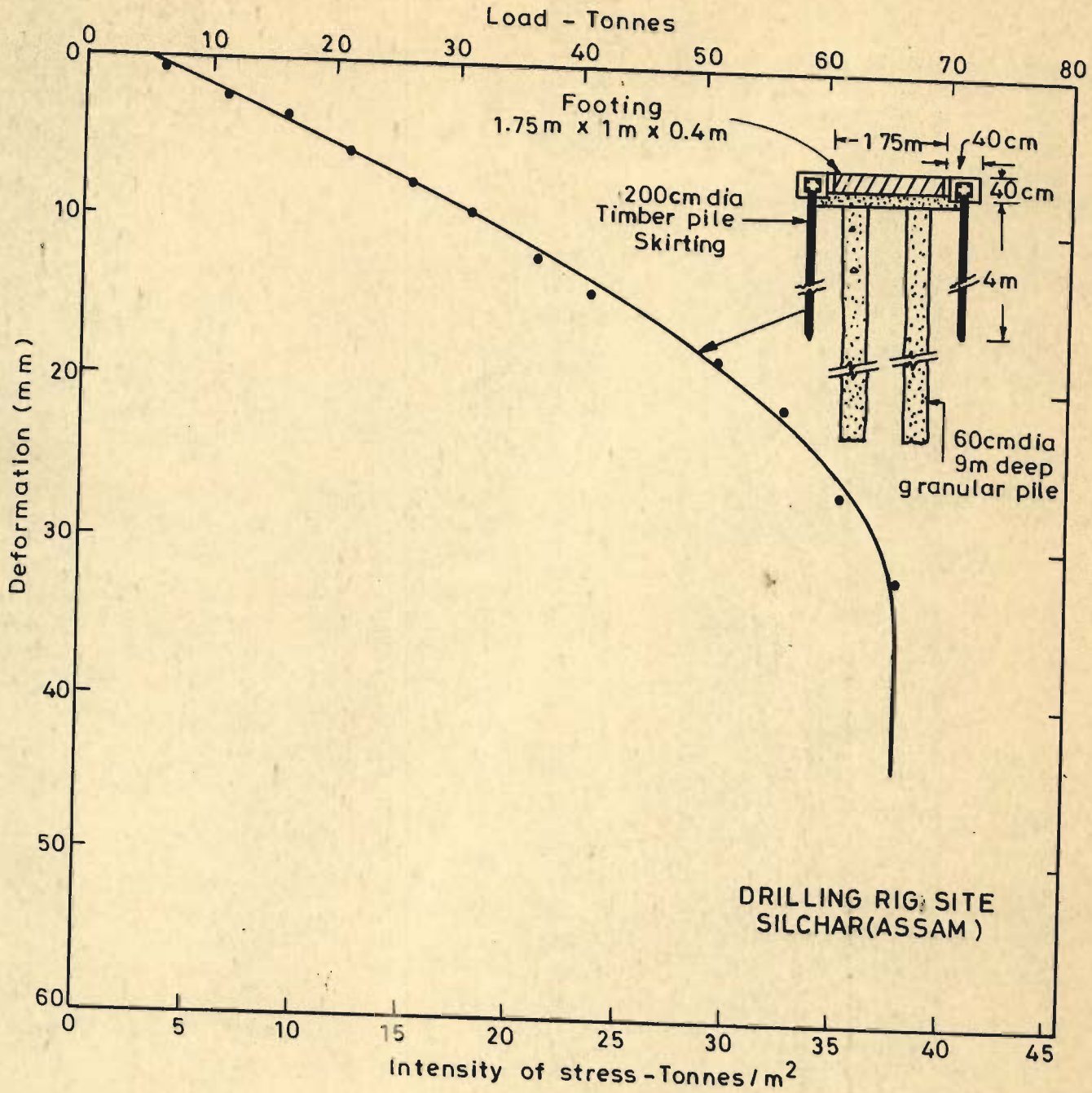


FIG.5.19. LOAD DEFORMATION BEHAVIOUR OF SINGLE GRANULAR PILE AND SKIRTED 2-GRANULAR PILE GROUP USING TIMBER PILE SKIRTING

tank site (Haldia) where deep layers of soft clay was encountered, the load-deformation characteristics of 2 pile groups without skirt having 55cm dia., 9.5m deep, when subjected to load testing indicated almost same behaviour Fig. 5.20.

The deformation corresponding to elastic yield point E and plastic yield point P (ultimate load) is shown in Table 5.3 for plain footing (PF), plain granular pile (PGP) and skirted granular piles (SGP) in Col. 9 and 10 and 3 to 5 respectively. These deformation as percentage to footing width/pile diameter for plastic yield point have also been shown in Col. 6,7 and 8 and for elastic yield point in Col. 11 and 12 (Table 5.3) respectively for all the sites. Glance of Table 5.3 indicates that the ultimate load (plastic yield point P) for the plain footing in virgin soil is mobilised at an average deformation of 2.75 (1.5-4) per cent of the footing dimension (Col. 6). However, this value is found to increase to 10 per cent (9.1 to 10.4) of the pile diameter in the case of plain granular piles and 11.5 per cent (9.6 to 13.6) (Col 8) for the skirted granular piles. It is further observed (Col. 11 and 12 of Table 5.3) that the elastic yield points (correspondint to point E) for plain and skirted granular piles whether single or in groups is mobilised at an average deformation of 4.5 per cent of the pile diameter. Thus it may be concluded that in loose to medium dense cohesionless subsoil deposit, the ultimate load carrying capacity of Plain/ Skirted granular piles single or in group mobilises at a deformation of about 10 to 12 per cent of pile dia. and the deformation corresponding to elastic yield point is found equal to 4.5 per cent of pile dia. Further it is noted that these values are independent of number of piles in a group. In case of soft clayey silt and clay deposits, these values are found to be 5 per cent and 2.5 per cent respectively which is almost half the values obtained in cohesionless soil deposit.

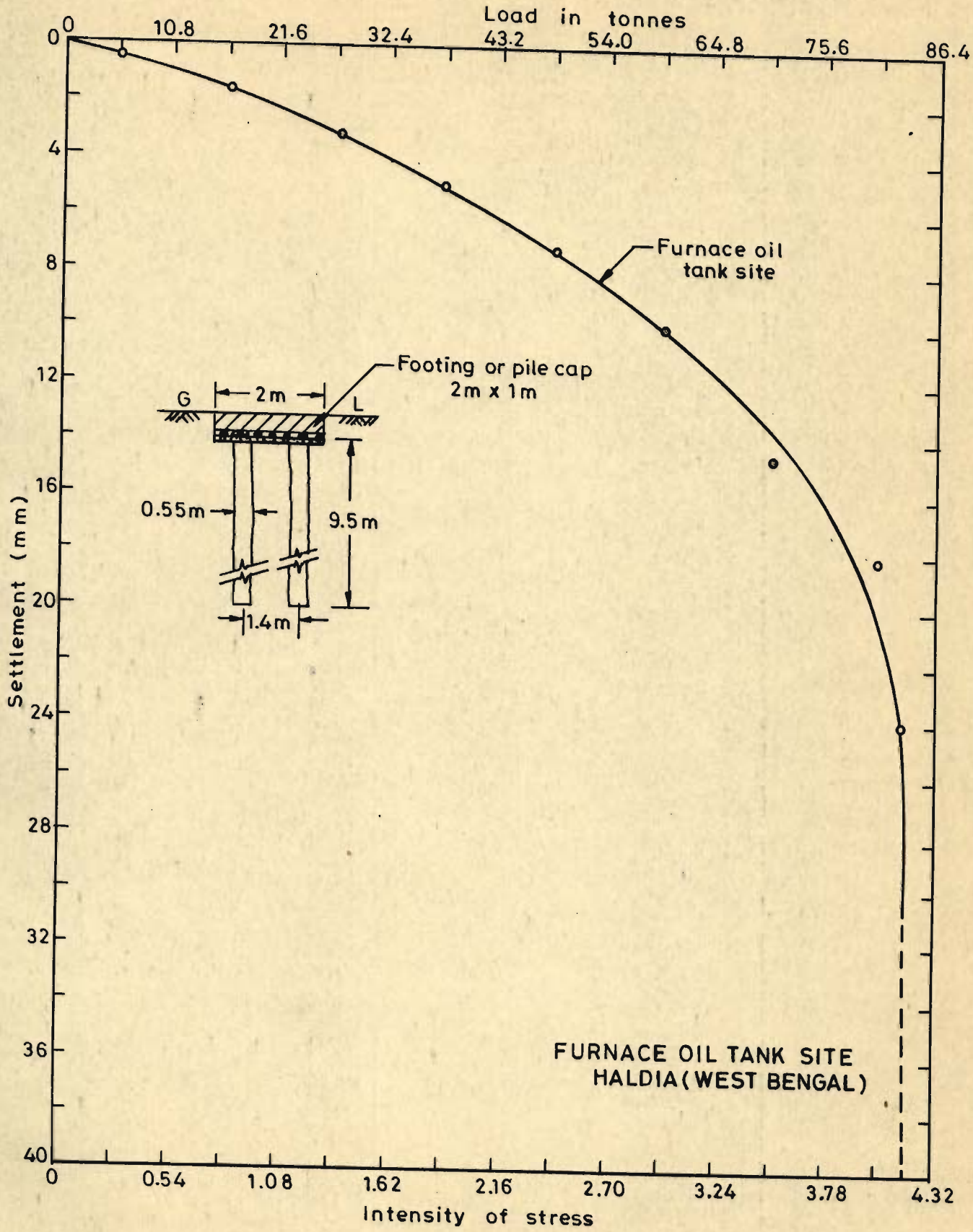


FIG.5.20.LOAD DEFORMATION BEHAVIOUR OF 2 PILE GROUP WITHOUT SKIRT

Table 5.3 - Deformation of Elastic & Plastic Yield Points of Plain/Skirted Granular Piles

Site	In situ Tests	Deformation at plastic yield point P (mm)			Deformation as % of Footing width	Deformation as % of Pile Dia.		Deformation of Elastic yield Point E (mm)		Deformation as % pile Dia.		Remarks
		PF	PGP	SGP	PF	PGP	SGP	PGP	SGP	PGP	SGP	
1	2	3	4	5	6	7 *	8 *	9	10	11	12	13
CBRI Officers Hostel Site Roorkee	Single	28	25	30	3.5	10.0	12.0	11.0	12.0=	4.4	4.8	Individually skirted granular piles, 25cm dia., 3.5m deep
	2 pile group	16	31	24	4.0	12.4	9.6	10.0	15.0	4.0	6.0	
	3 pile group	25	26	24	2.08	10.4	9.6	16.0	15.0	6.4	6.0	
	4 pile group	28	29	24	2.33	11.6	9.6	13.0	14.0	5.2	5.6	
	Single	20	32	36	2.85	9.1	10.2	11.0	12.0	3.1	3.4	Collectively skirted granular piles 35cm dia., 8.0m deep
	2 pile group	22	34	44	2.58	9.7	12.5	12.0	11.0	3.4	3.1	
	3 pile group	24	-	46	1.5	-	13.1	-	14.0	-	4.0	
	4 pile group	24	-	48	1.6	-	11.4	-	12.0	-	3.4	
	5 pile group	24	-	26	2.4	-	10.6	-	9.0	-	3.2	Collectively skirted granular piles 25cm dia., 3.5m deep
	4 pile group with unit pile	34	-	34	3.4	-	13.6	-	10.0	-	4.0	-do- with unit pile 3m deep

Table 5.3 Contd..

1	2	3	4	5	6	7**	8**	9	10	11	12	13
	4 granular pile group	-	-	32	-	-	12.8	-	12.0	-	4.8	Collectively skirted granular pile, 25cm dia., 3.5m deep using timber pile skirting
	4 pile group	-	-	34	-	-	13.6	-	14.0	-	5.6	-do- using prefab. pipe unit skirt
	Skirted Footing	26	-	34*	2.6	2.88	3.46*	-	12.0	-	1.2*	Using timber pile skirting without granular pile
	Skirted Footing	30	-	35*	3.0	3.33	3.88*	-	14.0	-	1.5*	Using prefab. pipe unit skirting without granular piles.
Molasses Tank site (Budaun)	2 pile group	-	35	-	-	10.0	-	12.0	-	3.42	-	35cm dia., 3m deep Plain Granular Pile
	4 pile group	-	-	Pile group did not fail (for 70t -8mm)			-		12.8		2.85	35cm dia., 3m deep collectively skirted granular piles
Drilling Rig site (Silchar)	2 pile group	-	-	30	-	-	5.0	-	16.0	-	2.7	60cm dia., 9.0m deep collectively skirted using timber pile skirting.
Furnace Oil tank Site(Haldia)	2 pile group	-	30	-	-	5.0	-	13.0	-	2.4	-	55cm dia., 9.5m deep granular piles without skirting.

*Skirted footing without piles.

**The deformation at ultimate load was found to be equal to 58% of the pile dia. (Hughes & Withers 1974). However, Thorburn (1975) based on field tests has reported a value equal to 10-12 per cent of pile diameter.

5.6.2 Effect on Ultimate Load Carrying Capacity

From the insitu test results at CBRI officers hostel site the ultimate load carrying capacity of (a) 25cm dia. 3.5m deep plain and individually skirted granular piles single and group (Fig. 4.42) (b) 35cm dia. 8m deep piles single and in groups plain and collectively skirted granular piles (Fig. 4.43) and (c) 25cm dia. 3.5m deep collectively skirted granular pile group, (Fig.5.15) (d) 4 pile group having same pile dimensions as earlier but using prefabricated timber pile and pipe unit skirting (Fig. 5.16 and 5.17) and (e) 25cm dia., 3.5m deep collectively skirted granular piles with a central concrete unit pile (Fig. 5.18) is obtained corresponding to point P. The same is tabulated in Table 5.4. The ultimate loads from in-situ tests on 35cm dia. 3m deep pile group and 4 pile group collectively skirted (Fig. 4.46) at Molasses tank site, Budaun. Drilling Rig site, Silchar (Fig. 5.19) and Furnace oil tank site, Haldia (Fig. 5.20) have also been tabulated in Table 5.4 with data from tests on plain footings on virgin soil. The per cent improvement in ultimate load capacity of the virgin soil Q_s due to plain granular piles, Q_p are shown in Col. 5. Col. 6 shows the per cent increase in capacity of skirted granular piles Q_{sk} over the ultimate load of plain piles Q_p and Co. 7 shows the per cent improvement in ultimate capacity of skirted granular piles, Q_{sk} whether single or in groups of 2,3,4 and 5 over the ultimate capacity of virgin soil, Q_s.

The test data (Table 5.4, col. 5) indicates that the ultimate load capacity of single plain pile and plain pile groups of 2,3 and 4 pile, increases from 164 per cent to 427 per cent over the ultimate load of virgin soil. The ultimate load is further increased when the piles are skirted either individually or

collectively. The increase in ultimate load is found to vary from 317 per cent to 566 per cent which is significant.

In case of drilling Rig site Silchar the 2 plain granular pile group, 60cm in dia., 9m deep collectively skirted under a load of 66.5 tonnes had a settlement of 28mm only while a single isolated plain pile 60cm in dia. 9m deep under a load of 33 tonnes had a settlement of 23mm only, when loaded through a pile cap of 0.75mx0.7m size. This gives an intensity of stress equal to 6.28 kg/cm^2. Using a factor of safety of 3.0, the safe intensity of stress is 2.07 kg/cm^2. The safe bearing capacity as calculated from undrained shear strength (0.48 kg/cm^2) gave a value of about 1 kg/cm^2. This exhibits that due to the reinforcement on account of granular piles, there is an increase of 209 per cent in load carrying capacity of virgin soil. The increase in ultimate load is observed to vary between 26 to 50 per cent by providing skirt around the granular pile individually or collectively (Col. 6, Table 5.4).

In the case of soft clay deposit at Furnace oil tank site, Halida footing 2mx1m on 55cm dia. piles, 9.5m deep, 2 pile group without skirting reached an ultimate load of 81 tonnes giving an intensity of stress as 4.05kg/cm^2. Thus with a safety factor =2, the safe capacity is 2 kg/cm^2. The undrained shear strength of the soft clay was 0.27 kg/cm^2 giving the ultimate bearing capacity of 0.55kg/cm^2(FS=3) This shows an increase in safe capacity of 363% over the soft clay capacity by installing plain granular pile. Thus, it may be noted that even in soft clayey silt and clay deposit, there is a significant improvement in load carrying capacity.

5.6.3 Group Efficiency

In the case of pile and pile groups when collectively or individually

Table 5.4 - Improvement in Ultimate Load Capacity of the Virgin Soil Due to Granular Pile Reinforcement

In-situ tests on Granular piles	Ultimate load (tonnes)			Per cent improvement in ultimate load capacity due to reinforcement			Remarks
	Plain footing equivalent to pile cap on virgin soil Q_s	Plain Granular Pile Q_p	Skirted Piles Q_{sk}	By plain pile over virgin soil $(Q_p-Q_s)/Q_s$	By skirted pile over plain pile $(Q_{sk}-Q_p)/Q_p$	By skirted pile over virgin soil $(Q_{sk}-Q_p)/Q_s$	
1	2	3	4	5	6	7	8
CBRI Officers Hostel site	Granular piles 25cm dia., 3.5m deep individually skirted						
Single pile	1.8	9.5	12	427.7	26.3	566.66	
Group of 2	4.9	19	24	287.75	26.31	389.79	
Group of 3	8.5	26.5	38	211.7	43.4	347.0	
Group of 4	11.5	35	48	204.3	37.1	317.0	
	Granular piles 35cm dia., 8m deep collectively skirted						
Single	3.6	14.0	20	288.8	34.3	455.5	
Group of 2	9.2	28.0	42	204.3	50.0	356.52	
Group of 3	16	-	68	-	-	325.00	
Group of 4	18	-	83	-	-	361.11	

Table 5.4 Contd..

1	2	3	4	5	6	7	8
	Granular piles, 25cm dia., 3.5m deep collectively skirted using Brick Pannel Skirting						
Group of 5	10	-	39	-	-	290	
	Granular piles, 25cm dia., ,3.5m deep collectively skirted using RCC skirt						
Group of 4 piles with load Bearing Element	10	-	43	-	-	330	
Steel Tank Budaun site	Granular piles 35cm dia., 3m deep collectively skirted						
Group of 2	14.00	37	-	164.2	-	-	Plain granular pile
Group of 4	37.68	-	Greater than 100t	-	-	Greater than 167	Skirted granular pile not tested to failure
CBRI Officers Hostel Site							
Skirted footing without granular piles	6.48	13.77*	-	112.5**	175.9***	-	Prefabricated pipe unit skirting retrievable type 1m deep
Group of 4	6.48	-	38	-	-	486.4	-do-
Skirted footing without piles	7.5	17.0*	-	126.6**	123.5***	-	Timber piles skirting 1m deep, held rigidily by clamps at foundation level
Group of 4	7.5	-	38	-	-	406	-do-

*Ultimate load of skirted footing without piles, **Improvement due to skirting over plain footing, ***Improvement due to skirted granular piles over skirted footing.

skirted having 25cm dia. and 3.5m length, the value of η varies between 0.82 to 1.05. Table 5.5. Further study of the table 5.5 indicates that for 35cm dia., 8m deep granular piles, single and in groups of 2,3 and 4 the value of η is found to vary between 1 to 1.13.

Table 5.5 Group Efficiency Factor For Skirted Granular Piles*

In situ tests	Pile dia. (cm)	Pile length (m)	Ultimate load observed (t)	Ultimate load of single pile col. 4/No. of pile (t)	Efficiency factor η	Remarks
1	2	3	4	5	6	7
Single pile	35	8	20.0	20.0	1.0	Collectively skirted granular pile single and in group
2 pile group	35	8	42.0	21.0	1.05	
3 pile group	35	8	68.0	22.6	1.13	
4 pile group	35	8	83.0	20.75	1.03	
4 pile group	25	3.5	38.0	9.5	1.0	Collective skirting using Timber piles
5 pile group	25	3.5	39.0	77.8	0.82	Collective skirting using brick pannels
4 pile group	25	3.5	38.0	9.5	1.0	Collective skirting using pipe units
Single pile	25	3.5	12.0	12.0	1.0	Individually skirted piles using m.s. pipes
2 pile group	25	3.5	24.0	12.0	1.0	
3 pile group	25	3.5	38.0	12.66	1.05	
4 pile group	25	3.5	48.0	12.0	1.0	

*CBRI OFFICERS HOSTEL SITE (ROORKEE)

These observations show that collectively or individually skirted pile when used in groups of 2, 3, 4 and 5 at a pile spacing of 3 times the pile diameter, the group efficiency factor for all practical purpose may be taken as unity.

5.6.4 Effect on Settlement

The ultimate load carrying capacity of virgin sub-soil stratum consisting of loose-medium cohesionless deposits at CBRI officers hostel site is found from insitu load tests on plain footings as shown in Figs. 5.12 with corresponding value marked on the figure. The settlement corresponding to the load equal to the ultimate bearing capacity of the virgin soil found from these figures is shown in Table 5.6, Col. 3. Similarly, settlement of 3 and 4 pile groups plain and skirted individually and collectively is shown in Col. 4 and 5 respectively. The reduction in settlement due to plain granular pile installation over the settlement of plain footing is shown in col. 6. Similarly reduction in settlement due to skirted granular pile over plain granular pile is shown in col. 7. Column 8 shows the reduction due to skirting around granular pile over plain footing.

It is observed that a significant reduction in settlement is achieved at a load-corresponding to the ultimate bearing capacity of the soil when the sub-soil is reinforced with plain granular pile. This reduction in settlement in the present tests is shown to be as 76 per cent for 3 and 4 pile groups. The settlement is further reduced to 86 per cent (with a variation from 84-87.5 per cent) when the piles groups are individually or collectively skirted col. 8, Table 5.6. When the plain granular piles are collectively skirted with a unit pile cast in the centre, the reduction is found to be as high as 94 per cent. It is further observed from col. 4 and 7 of the same table that by a mere provision of skirting around a footing the reduction in settlement may be as high as 53 to 64 per cent (see single star and double star marks).

Table 5.6 - Settlement reduction due to Plain & Skirted Granular Pile Installation

Site	Insitu Tests	Settlement at Ultimate load of virgin soil (mm)			Reduction in settlement per cent due to:		
		PF	PGP	SGP	PGP over PF	SGP over PGP	SGP over PF
CBRI Officers Hostel site	Individually Skirted Granular Piles						
	Group of 3	25	6	4	76.0	33.3	84.0
	Group of 4	25	6	3.5	76.0	41.6	86.0
	Collectively Skirted Granular Piles						
	Group of 3	25	-	4	-	-	84.0
	Group of 4	25	-	2	-	-	92.0
	Group of 5	32	-	4	-	-	87.5
	Group of 4 Timber Piles	24	8*	3.0	-	62.5**	87.5
	Group of 4 with Prefab. pipe unit	30	8*	3.75	-	53.12**	87.5
	Group of 4 with LBE	36	-	2	-	-	94.4

*Settlement of Skirted Footing without piles

**Reduction in settlement due to Skirted Granular pile over skirted footing without pile.

In addition to the above observation, the present series of tests have further shown that the settlement of 3 and 4 pile groups plain and individually skirted does not exceed 14mm and 10mm respectively at a load corresponding to 2 times the ultimate bearing capacity of the soil (2 kg/cm^2), Fig. 5.12. Also, when the load is increased to 3 times the ultimate load of the virgin soil (3 kg/cm^2), the settlement of 3 and 4 pile groups individually skirted does not exceed a value of settlement more than 16mm.

Similar observations have been made in case of 35cm diameter, 8m deep plain and collectively skirted granular pile. The study of the figure 5.13 shows that at 2 and 3 times the ultimate bearing capacity of the virgin soil (1.6 kg/cm^2 to 2.4 kg/cm^2) the corresponding settlement does not exceed 11.5mm and 24mm respectively with a variation of (6mm to 11.5mm) and (11mm to 24mm). The settlement for a 25cm dia. 3.5m long 5 pile group collectively skirted is found to be equal to 6 and 14mm only at 2 and 3 times the ultimate bearing capacity of the soil, Fig. 5.14. Observations in case of 4 granular piles, 25cm dia., 3.5m deep collectively skirted using timber pile skirting and prefabricated pipe unit skirting, Figs. 5.16 and 5.17 also indicate the same trend. The settlement at 2 and 3 times the ultimate bearing capacity is shown to be 6mm and 11mm and 7mm and 10mm respectively. It may also be seen from the same figures that the settlement at a load equal two times the ultimate bearing capacity of the soil, the settlement of skirted footings without granular piles is not exceeding a value of 22mm and 28mm respectively.

The observations from the tests reveal that the virgin soil stratum consisting of loose-medium dense cohesionless sub soil stratum as low value of ultimate bearing capacity and the observed settlements were noted to be high. The installation of plain granular piles in

the virgin soil stratum resulted in increasing the strength and hence the ultimate bearing capacity of the plain granular pile-soil system was found to increase significantly and settlement was reduced considerably. Thus the increase in overall bearing capacity and reduction in settlement may be attributed to high shear strength of pile material and compaction of surrounding soil during pile installation which leads to improved stress deformation behaviour of composite mass.

When the plain granular piles are collectively skirted, the applied stresses are transferred deep below the base of the skirt resulting into larger resistance against bulging failure due to increased effective mean normal stress. In addition to the above facts, when the soil plug reinforced with plain granular piles is subjected to vertical stress, the rigid skirt provides confinement to the soil plug which in turn results in further reduction of overall settlement.

These observations suggest the potentiality of plain/skirted granular pile group system to provide a suitable foundation for structure which are sensitive to settlements and is required to sustain high loads. The test data reveal that the settlement at a load equal to 2 to 3 times the ultimate bearing capacity of the virgin soil is found to be negligible.

5.7 Types of Skirting

As discussed in chapter IV (art. 4.5.3 through 4.5.7) different types of skirt were used in the present test programme (Table 4.1). These are:

(a) Reinforced cement concrete skirt with mild or tor steel bar reinforcement.

(b) Reinforced cement concrete skirt with thin GI or mild steel sheet reinforcement.

(c) Pre-fabricated brick pannel skirting.

(d) Pre-fabricated pipe unit skirting, retrievable type

(e) Precast cement concrete piles steel hume pipe piles or timber pile skirting.

The choice of a particular type of skirt is dependent upon its functional utility based on design requirement, availability of local material and equipment for the construction of skirt and the local experience. The skirt may be designed as rigid member providing sufficient confinement to the soil plug. Steel reinforcement may be replaced by mild steel sheets depending upon cost and availability consideration, (Fig. 4.32 and 4.33). Similarly pre-fabricated brick pannel skirting (Figs. 4.17 and 4.18) may provide as an economical alternative to r.c.c. skirt where availability of stone metal is scarce and bricks of better quality are costly.

Besides the availability of the local material, there are certain structures which require foundations for a certain specific period such as foundations for a on-shore oil drilling rig. To provide an economical and better foundation, granular pile group with prefabricated pipe unit skirting (Figs 4.21 and 4.26) may be used under such situations. Though the initial cost in procuring such a type of skirting may call for a heavy investment however, because of its reuse value (by retrieving from one site and installing on the other) the overall cost may considerably be reduced.

In the retrievable type of skirting, precast reinforced concrete piles, steel on hume pipes the main draw back is that the skirt made of these piles is not rigid. Thus to provide sufficient rigidity, the pile tops at the foundation level need be clamped together suitabily by providing rigid steel members on outside the skirt. (Fig. 4.27 and 4.28). Alternatively, the flexibility of such a type of skirting system can be reduced - significantly by connecting all the pile heads at foundation level by casting in situ reinforced concrete beam suitably designed for resisting lateral stresses.

A series of test results using various types of skirting at CBRI officers hostel site having loose to medium dense non cohesive soil deposit around granular pile group is shown in Fig. 5.21. Curves 1 shows the test results of 4 granular pile with a central unit pile having 25cm dia., 3.5m deep collectively skirted by a reinforced cement concrete skirt using mild steel as reinforcements. Curve 2 gives the test results of 5 granular pile groups of same dimension collectively skirted by prefabricated brick panel skirting whereas curves 3 and 4 correspond to the prefabricated pipe unit and timber pile skirting.

Fig 5.21 shows that for a cohesionless sub soil deposit footing has an ultimate bearing capacity of 0.8 kg/cm^2. It is further noted that for a collectively skirted granular pile group having different type of skirts (curves 1 to 4) settlement ranges between 9mm to 13mm for an intensity of 2.4 kg/cm^2, 3 times the ultimate soil bearing capacity of footing on virgin soil. Also at 4 times the ultimate soil capacity (3.2 kg/cm^2), the settlement does not exceed a value of 20mm.

Similar type of behaviour was noticed at Silchar site (Table 4.1) also where 20cm dia., 4m deep timber pile skirting was used with

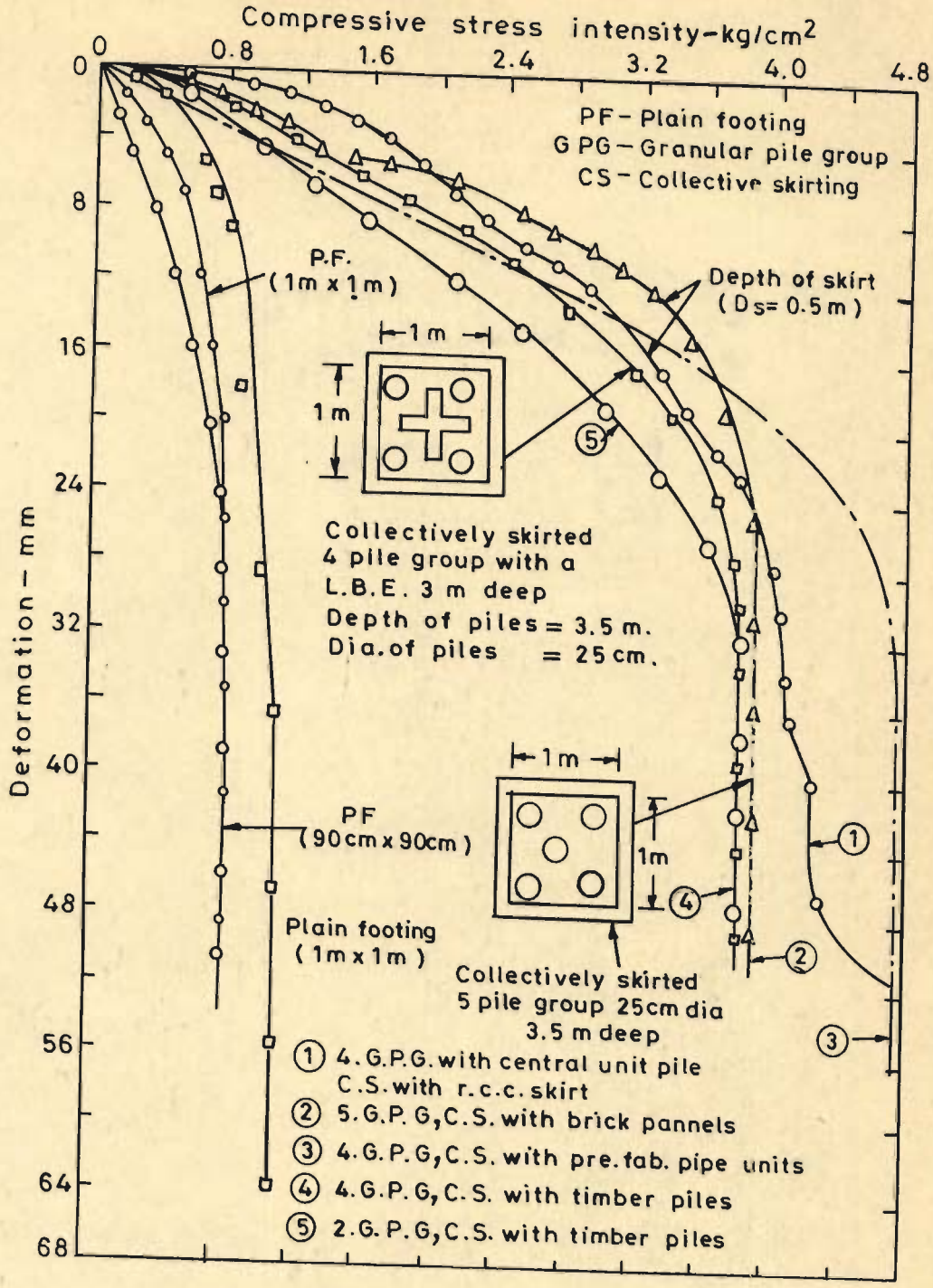


FIG.5.21–STRESS DEFORMATION BEHAVIOUR OF COLLECTIVELY SKIRTED 4-PILE GROUP WITH LOAD BEARING ELEMENT AND 5-GRANULAR PILE GROUP WITH DIFFERENT TYPES OF SKIRTS.

a r.c.c. edge beam at the foundation level, Fig. 5.21 (5).

During the pull out tests, the skirt was also found to move up alongwith the unit pile and group of granular pile showing the system to act as one unit.

5.8 Up-lift Resistance of Plain & Skirted Granular Piles

5.8.1 Up-lift Load Capacity of Single and Groups of Plain Granular Piles

Pull load-movement relationship of single pile and pile in groups of 2,3 and 4 having 25cm dia., and 3.5m deep cast at a spacing of 3 pile dia. at CBRI campus site having predominantly loose non cohesive deposit is shown in Fig. 4.48. It is noted that the nature of curve in all the four cases is practically the same. The pull out load-movement behaviour of these granular piles soil system single or in groups comprises basically of three parts, viz, the initial straight line portion demonstrating elastic behaviour upto point E the middle part EP showing elasto-plastic behaviour and a vertical steep straight tangent beyond point P, exhibiting completely plastic behaviour.

The ultimate uplift capacity of single and group of granular piles is taken as the load at which the uplift load-pull movement curve goes into a steep straight tangent. These values have been tabulated in Table, 5.7. The ultimate uplift capacity of a single pile in a group is obtained by dividing the number of piles in the same group and is indicated in col. 6. The group efficiency factor is shown in col. 7. It is noted from col. 7 that the group efficiency factors as obtained from the insitu pull out tests vary in a narrow range of

0.9 to 1, and are independent of number of piles in a group. Hence for all practical purposes the group efficiency factor for pile groups subjected to uplift load may be taken as unity. It is also interesting to note that the group efficiency factors for plain granular piles single and in groups of 2, 3 and 4 subjected to vertical compressive load was also noted as unit.(art. 5.6.3 and Table 5.5)

Table 5.7 - Group Efficiency of Plain Granular Piles

Site	Insitu tests	Granular pile dimen-sion(m) Dia.	Length	Ultimate uplift load observed (t)	Ultimate uplift capacity of single pile (col 5/ no. of piles) (t)	Group Efficiency
1	2	3	4	5	6	7
CBRI campus site	Single pile	0.25	3.5	5.5	5.5	1.0
	2 pile group	0.25	3.5	10	5.0	0.9
	3 pile grpup	0.25	3.5	15.5	5.16	0.93
	4 pile group	0.25	3.5	22	5.5	1.0

5.8.2 Pull-movement of Plain Granular Piles

The pull movement corresponding to point E elastic range for single and groups of piles ranges only from 4.4 to 5.6 per cent of the granular pile diameter with an average value of 5 per cent. However, the plastic yield point P (ultimate uplift-load capacity) of these piles occurs at an average pull movement of about 10 per cent of the pile dia. with a minor variation of 8.4 per cent to 12 per cent. These observations are similar to those granular pile single and in

groups under compressive loads.

5.8.3 Skirted Granular Piles with a Central Unit Pile

The pull load-movement behaviour of a group of 4 granular piles 25cm in dia., 3.5m deep with a central unit pile (Load Bearing Element) consisting of two 15cmx45cmx3m reinforced concrete unit cast monolithic at right angles to each other giving a cross shape unit pile, at CBRI officers hostel site is shown in Fig. 4.49(c) Also the pull load-movement relationship of a similar unit pile alone and a unit pile surrounded by 4 granular pile group collectively skirted, on the same site is shown in Fig. 4.49. Curves (a) and (b) respectively. Curves (a) and (b) correspond to the case when the unit pile alone was subjected to uplift load while curve (c) correspond to the case where the unit pile alongwith 4-granular pile group was pulled together.

It is observed from Fig. 4.49(a) that the ultimate uplift capacity of a unit pile is 7t only when cast in the virgin soil consisting of loose-medium dense cohesionless sub soil stratum. The same unit pile when cast in the same sub-soil, in the centre of a 4 granular pile group collectively skirted, the ultimate pull load increases to 16t. when the unit pile alone is subjected to pull load. This indicates an increase in the uplift load of 128 per cent, (Fig. 4.49b) Further when the unit pile and the 4-granular piles are subjected to pull load, the ultimate uplift capacity is observed to be 36t (Fig. 4.49c) which is an increase in the ultimate uplift load of 414 per cent over ultimate uplift capacity of unit pile cast in virgin ground and 125 per cent over the ultimate uplift capacity of the similar unit pile when cast centrally in the centre of a 4 granular pile group collectively skirted and the unit pile is pulled alone. These observations indicate that the unit skin friction between the shaft

and the surrounding soil is increased significantly by casting granular pile group around it. This is obvious as the installation of the granular pile is likely to increase the relative density of the virgin soil due to compaction and hence the shear strength. Also in the case of curve (c) fig. 4.49 when the unit pile and the 4 granular pile group is pulled together the ultimate uplift capacity of 36t is noted. This uplift load is sum total of (a) ultimate uplift capacity of unit pile in the improved ground which is 16t (Fig. 4.49b) (b) ultimate uplift capacity of 4 granular pile group which may be taken as 20t based on the test results on similar strata (Fig. 4.48). Thus it may be concluded that the ultimate uplift capacity of unit piles or r.c.c. concrete piles may significantly be improved by installating granular piles at adequate spacing around it and collectively skirting them. It is observed from Fig. 4.49 that the ultimate load capacity of a unit pile is more than double (7-16t) when the unit pile is cast in the centre of the 4 granular pile collectively skirted and the pull movement at elastic yield point E, is increased from 2mm to 9mm. It may further be noted that the ultimate uplift capacity of the unit pile and skirted granular pile together is further doubled (16t to 36t) when both are pulled together. However, the pull movement at elastic yield point remains almost constant. The limited test data reveals that uplift load of a skirted granular with a unit pile in the centre increases significantly. However, the pull movement at failure remains practically same.

5.8.4 Ultimate Load Capacity of Skirted Granular Piles with a Central Unit Pile Under Compressive & Uplift Vertical Loads

Pull load-pull movement behaviour of 4 granular pile groups 25cm in dia. and 3.5m long at the CBRI officers hostel site with a central

unit pile as obtained from the pull-out test is shown in Fig. 5.22 Curve (1) is compared with the load-deformation behaviour of the same foundation load tested under compressive vertical load shown in Fig. 5.22(2). Also the load-settlement curve of a 5 granular pile group collective skirted, obtained from compressive load test on the same site having similar condition is also placed on the same graph Fig. 5.20(3).

It is interesting to note from Fig. 5.20 that the pull load-movement and load-deformation behaviour of 4-granular pile group collectively skirted with a central pile under uplift load and compressive load is practically similar (curve 1 & 2). Also the behaviour of a 5 granular pile group-collectively skirted and load tested under compressive vertical load curve 3 when compared with curves 1 and 2 exhibit the same behaviour.

Such a similarity in behaviour of skirted granular pile foundation with central unit pile cast in the centre, further confirms the earlier observation that the granular pile group, the skirt and the central unit pile behaves as a single block or unit after a certain stage of initial load application, till the ultimate load is reached. This statement was also confirmed during in situ tests. The footing cast on the soil plug reinforced with granular piles central unit pile and the skirt settled together under the load. This observation during compression testing is noticed due to the fact that during granular pile installation, the overall shear strength of the composite mass increases significantly because of high shear strength of pile material and compaction of surrounding soil. Further, the collective skirting around the group helps in transferring the stresses below the base of the skirt resulting into higher restraining stresses due to increase in the effective mean normal stress. Thus, when the soil plug is subjected to vertical compressive load, the overall bearing

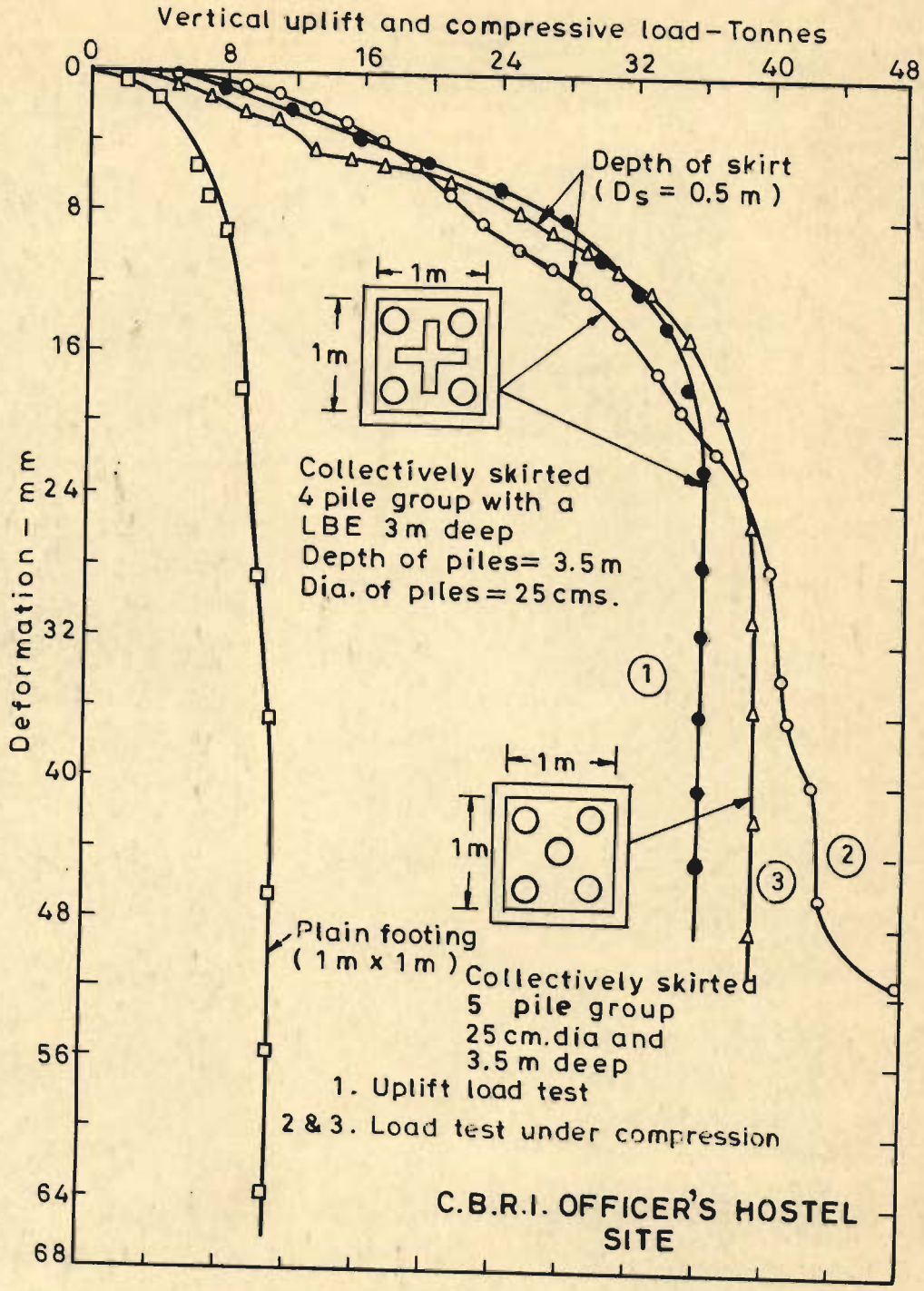


FIG.5.22. LOAD DEFORMATION BEHAVIOUR OF COLLECTIVELY SKIRTED 4-PILE GROUP WITH LOAD BEARING ELEMENT UNDER VERTICAL COMPRESSIVE AND UPLIFT LOADS

capacity and the unit skin friction on the shaft of the unit pile increase significantly. On the other hand, during the uplift testing when the pull load is applied on the granular pile group and the unit pile, the resistance to pull is offered by the restraining pressure at near the toe of the granular pile due to high effective mean normal stress and the increased unit skin friction on the shaft of the unit pile besides the friction provided by the interface of the skirt.

It is further noted (Fig. 5.20) that in a loose to medium cohesionless sub-soil deposit having ultimate bearing capacity of 11t/m^2 as virgin soil (curve 4), the skirted granular pile foundation when placed in such a deposit, does not exceed a pull movement or deformation more than 3mm at the ultimate load capacity of the deposit. Even at two times the ultimate bearing capacity this value does not exceed 5mm and at two and a half times the ultimate capacity of the soil, the value is limited to 18mm only. Thus in situations where the foundation is to sustain high loads and stringent settlement or pull movement criterial is to be followed such a foundation may provide an economical alternative.

5.9 Lateral Load Capacity

5.9.1 Lateral Load Capacity and Lateral Displacement

The unit pile and the 4-granular pile, 25cm in diameter, 3.5m long with a central unit pile cast in the centre and collective skirted were also load tested by applying lateral loads at the ground level in small increments as discussed earlier (art. 4.7.3). The units already used in uplift load testing were subjected to lateral load testing. The lateral load vs lateral movement behaviour of unit pile cast in the virgin ground and the skirted granular piles with centatral unit pile (LBE) is shown in Fig. 4.50. The lateral load-displacement curve indicate a non-linear behaviour till the ultimate lateral load (4.9t) is reached (Point p. Fig. 4.50). For the pile tested, the deformation at the ultimate load is noted to be 34mm. On contrary in the case of skirted granular pile group with a central unit pile the ultimate lateral load (11.0t) is attained at a lateral displacement of 17mm only (point P', Fig. 4.50). In the case of skirted granular pile, the load-displacement curve is transformed into elastic (Point E) and elasto plastic EP system as in the cases of compressive and pull load tests. Comparision of curves (a) and (b) Fig. 4.50 further reveals that in the case of skirted granular piles, while the ultimate lateral load carrying capacity increases almost two times, the lateral displacement at ultimate reduces to half.

5.9.2 Safe Lateral Load from Insitu Tests

Despite the considerable advancement in the treatment of the problem of lateral load capacity of soil-pile system, in situ load tests on piles continues to be the only reliable method of assessing lateral load capacity of pile and pile group.

The safe lateral load of a concrete pile is estimated from the load displacement behaviour as the least of the following IS:2911 Part (IV)-1979.

(a) Fifty per cent of the final load corresponding to total displacement of 12mm.

(b) Final load at which the total displacement corresponds to 5mm

(c) Load corresponding to the specific permissible displacement.

The criteria listed above are applicable for concrete piles. However, in the absence of any other criteria, the same have been used for analysing the results of unit pile. The safe lateral load is evaluated from Fig. 4.50 and tabulated in Table 5.8. From Table 5.8, it is noted that the safe lateral load capacity of skirted granular pile group with a central unit pile is found to increase almost by 400 per cent over the unit pile alone cast in the virgin sub-soil stratum.

Table 5.8 - Safe Lateral Load Capacity* from IS:2911 (IV)-1979

Type of piles	Safe Lateral Loads			Safe load as least of three
	Criteria 1 (t)	Criteria 2 (t)	Criteria 3 (t)	
Unit pile	1.5	1.5	1.5**	1.5
Skirted Granular pile with central unit pile	5.9	7.5	7.5**	5.9

*CBRI Officers Hostel site, Roorkee

**Assuming permissible displacement as 5mm.

The significant improvement in the lateral load - displacement behaviour and in the safe lateral load capacity of skirted granular piles with central unit pile over the lateral load behaviour of unit pile alone may be attributed to the following factors:

(a) The improved relative stiffness provided by the compacted soil due to granular pile installation which has higher strength in comparision to virgin soil.

(b) Higher values of the EI of the pile material consisting of skirt, composite mass of granular pile soil plug and the concrete unit pile.

In view of the above, the overall stiffness of the top crust of the soil increases significantly. As the property of the surrounding top soil around the pile subjected to lateral load is known to have maximum influence on the later-load-displacement-behaviour of soils, the increase in stiffness of top soil layer results into high lateral load capacity and reduced horizontal displacement.

5.10 Comparision of Observed & Predicted Ultimate Loads and Settlements

The number of insitu tests carried out at different sites, the type of the tests and the nature of loads whether compression, uplift or lateral, whether plain footing,plain granular piles or individually/collectively skirted pile and pile groups and the different types of the skirt material have been given in Table 4.1. The ultimate loads obtained from the various insitu load tests and the settlement of the plain/individually and collectively skirted granular piles single and in groups for a particular design load intensity of stress have been compared with the corresponding predicted ultimate loads and settlement based on insitu soil

properties. The details of these have been discussed in this section.

5.10.1 Ultimate Load in Compression Tests

The ultimate load carrying capacity of plain and skirted granular piles, (skirted individually or collectively) obtained from insitu load-settlement curves (defined as the load at which the load settlement curve becomes steep straight tangent) have been tabulated in Table 5.9 (Col. 4). The ultimate load carrying capacity of the plain granular piles single and groups and also, of individually skirted granular piles in cohesionless soils have been predicted by using equations 3.46 through 3.60 (art. 3.8.2) and in cohesive sub-soil deposits equations 3.70 to 3.77 (art. 3.10). Appendix-A shows the typical computation for the ultimate load carrying capacity of a plain granular pile under vertical compressive load. In case of single and groups of skirted granular piles equation 3.46 has been used. The soil strength parameters namely insitu density, cohesion, angle of shearing resistance have been chosen from the laboratory tests. The computed values of the ultimate loads based on soil properties have been shown in col. 5 (Table 5.9) and the ratio of observed and predicted capacities have been shown in Table 5.9 (col. 6).

Study of the Table 5.9 (col. 6) reveals that the ratio of observed and predicted ultimate load capacities varies between a narrow range of 0.9 to 1.18 indicating thereby a good degree of correspondence.

5.10.2 Ultimate Loads in Uplift Tests

Similarly, as discussed in art. 5.10.1 in case of uplift capacity

Table 5.9 - Comparision of Observed and Computed Ultimate Loads

Site	Test No.	Insitu tests	Ultimate loads observed (tonnes)	Ultimate loads 'computed' (tonnes)	Ratio (Observed)/(Comp)	Remarks
1	2	3	4	5	6	7
CBRI Officers Hostel site (Roorkee)	1	Single pile	14.0	15.53	0.90	Plain Granular Piles from test no. 1 to 7
	2	Group of 2	28.0	31.06	0.90	
Molasses Tank site (Budaun)	3	Group of 2	37.0	34.88	1.06	
CBRI Officers Hostel site (Roorkee)	4	Single pile	8.5	8.0	0.94	
	5	Group of 2	19.0	16.0	1.18	
	6	Group of 3	26.5	24.0	1.10	
	7	Group of 4	35.0	32.0	1.09	
-do-	8	Single pile	20.0	22.0	0.90	Collectively skirted granular piles from test no. 8 to 12
	9	Group of 2	42.0	42.72	0.90	
	10	Group of 3	68.0	64.64	1.05	
	11	Group of 4	83.0	87.50	0.94	
	12	Group of 5	39.0	51.60	0.755	
	13	Skirted footing	17.5	16.49	1.06	Using Timber pile skirting without piles.

Table 5.9 Contd....

1	2	3	4	5	6	7
	14	Group of 4	38.0	37.08	1.02	
	15	Skirted footing	13.77	13.44	1.02	Prefab. pipe unit skirting without piles.
	16	Group of 4	38.0	33.66	0.98	-do- with piles
Molasses Tank Site Budaun	17	Group of 4	70.0*	98.0	-	Collectively skirted using r.c.c. skirt loaded upto 70 tonnes for 16mm settlement. No further load was available
CBRI Officers Hostel (Roorkee)	18	Single pile	12.0	11.7	1.02	Individually skirted piles using m.s. pipes as skirting
	19	Group of 2	24.0	23.4	1.02	
	20	Group of 3	38.0	35.1	1.08	
	21	Group of 4	48.0	46.8	1.02	
Drilling Rig Site Silchar (Assam)	22	Single	33.0	36.0	0.91	Plain granular pile, 60cm dia., 9m deep
	23	Group of 2	66.5	72.0	0.92	-do-
Furnace Oil Tank Site, Haldia, W. Bengal	24	Group of 2	82.7			Plain granular pile group, 55cm diameter, 9.5m deep
Mandatory Crude Oil tank site Haldia, W. Bengal	25	Group of 2	82.7			Plain granular pile 55cm diameter, 14m deep

*The pile group could not be loaded beyond 70 tonnes due to shortage of kentledge.

Table 5.10 - Comparision of Observed and Computed Ultimate Up-Lift loads for Plain & Skirted Granular Piles

Test No.	In situ Up-lift load Tests	Granular Pile dimension (m)		Unit Pile(LBE) Dimension (m)			Ultimate uplift loads (tonnes)		Ratio Col 8 / Col 9	Angle of wall Friction (δ)	Remarks
		Dia.	Length	Length	Width	Depth	Observed	Computed			
1	2	3	4	5	6	7	8	9	10	11	12
1	Single pile	0.25	3.5	-	-	-	5.5	4.8	1.14	-	All Plain Granular from 1 to 4
2	Group of 2	0.25	3.5	-	-	-	10	9.6	1.04	-	
3	Group of 3	0.25	3.5	-	-	-	15.5	14.4	1.01	-	
4	Group of 4	0.25	3.5	-	-	-	22.0	19.2	1.14	-	
5	Unit Pile(LBE) in virgin soil	-	-	0.45*	0.15*	3.0	7.00	7.82	0.895	$\delta = \frac{2}{3} \phi$	Unit pile pulled alone
6	Unit Pile cast in the centre of 4 granular pile group collectively skirted	0.25	3.5	-do-	-do-	-do-	16.00	15.67	1.02	$\delta = 1.2 \phi$	-do-
7	-do-	0.025	3.5	-do-	-do-	-do-	36.00	36.18	0.995	$\delta = 1.2 \phi$	Unit Pile and Granular Pile group pulled together

Note: 1. Test number 1 to 4 at CBRI Campus Site (Roorkee). 2. Test number 5 to 7 at CBRI Officers Hostel site (Roorkee)
3. *Two rectangular units (45cmx15cm) cast in situ at right angles
4. ϕ is the angle of shearing resistance of virgin soil.

of single and groups of 2, 3 and 4 plain granular pile at CBRI campus site and also for the unit pile and skirted granular piles at the CBRI officers hostel site, have been taken from the pull load and pull-movement curves obtained from insitu tests and have been tabulated in col. 8(Table 5.10). The ultimate uplift load capacities of the plain granular piles in cohesionless soil deposit have been computed from the proposed design method as detailed in (art. 3.12.T), utilising equation 3.102. Appendix B shows the typical computation for the ultimate uplift load carrying capacity of a plain granular pile. The soil strength parameters have been obtained from laboratory and field tests. Also the ultimate uplift capacities of unit pile cast in the virgin cohesionless sub-soil stratum and the skirted granular pile with central unit pile have been computed by equations 3.108 & 3.112 respectively.

In the computation of ultimate uplift resistances the angle of wall friction δ, between the surface of the unit pile and the surrounding virgin soil have been taken as (2/3 $\emptyset$) where $\emptyset$ is the angle of shearing resistance of the virgin soil (Das et.al 1977). However, in case of unit pile cast in the centre of 4 granular pile group collectively skirted, δ has been taken as (1.2 $\emptyset$) with a view to incorporate the effect of compaction during granular pile installation which results into significant increase in shear strength of the composite mass consisting of granular piles and the compacted cohesionless soil surrounding the piles. The predicted uplift capacities of plain granular pile, single and in group of 2, 3 and 4 and also for those of unit pile cast in the virgin sub-soil and the skirted granular piles have been shown in col. 9 (Table 5.10). The ratio of the observed and computed ultimate uplift capacities (col. 10, Table 5.10) indicate close agreement.

Thus it is noted that the comparision of ultimate loads whether in compression or under uplift loads (Table 5.9 and 5.10) of the plain and collectively skirted granular piles (whether single or in groups) predicted from the proposed design approach based on modified cavity expansion approach agrees well with the ultimate capacities obtained from the full scale insitu tests carried out at various sites.

5.10.3 Lateral Load Capacity

The details of the insitu load tests of a unit pile cast in the virgin sub-soil stratum and a unit pile cast in the centre of a collectively skirted granular pile group on CBRI officers hostel site have been shown in Table 4.1. The lateral load versus lateral displacement curves have been shown in Fig. 4.50. For the prediction of lateral load capacity the non-dimensional elastic solution (Reese & Matlock 1956) for laterally loaded piles have been used. The lateral load for a 12mm lateral displacement have been computed. The value of coefficient of horizontal sub-grade reaction, η_h has been taken as 0.245 kg/cm^3 based on Mc Corcle (1969) for the loose cohesionless soil. Thus the lateral load, Q_{hg} for the unit pile is found to be as 3.9t for 12mm displacement by using Reese and Matlock (1956) approach. The observed value of lateral load, Q_{hg} from Fig. 4.50 is noted to be as 3.8t. Appendix C gives a typical computation for the Lateral Load Capacity of a skirted granular pile foundation.

In the case of skirted granular pile group with central unit pile the value of Q_{hg} is computed as 13.17t against the observed value of 11.0t. Here again the value of n_h is taken as 0.25kg/cm^3. Appendix C shows the typical computation for the lateral load capacity of a skirted granular pile group with central unit pile. Based on limited test data, it is observed that the lateral load capacity of unit pile is significantly influenced by collectively skirted granular pile group due to increased stiffness. In view of this, it may be

anticipated that the installation of 2 to 3 rows of granular piles at a spacing of 3 to 4 pile diameter may further reduce the horizontal deflection.

5.10.4 Settlement of Plain and Skirted Granular Piles

The details of the various insitu tests carried out at different sites, on plain and individually/collectively skirted granular piles, dimensions and areas of pile caps (concrete footings or steel plates) used in the tests and the type and material of skirtings have been given in Table 5.11

The stress-deformation behaviour of various insitu tests carried out (Table 4.1) shown in Figs 5.12 through 5.20 indicated an average safe bearing capacity of $20t/m^2$ (18 to $45t/m^2$) with a factor of safety of 2. Also in practice, generally the design loads for heavy structures ranges between 15 to $20t/m^2$. In view of these facts, the settlements corresponding to a stress intensity of $20t/m^2$, have been obtained.

These observed settlements have been compared with the computed settlements under the same intensity of applied stress. The prediction of settlements of the soil granular piles system having plain granular piles, single and in groups, under the base of the pile caps were carried out by the proposed concept of Equivalent soil modulus, E_{eq} (equation 3.78; art. 3.11.1).

Using the value of equivalent soil modulus, E_{eq} for the various full scale tests the value of coefficient of volume compressibilities, m_{veq} were found out. The calculated values of E_{eq} utilising relative pile area, α and the modulus of soil and pile materials are shown in Table 5.12. The computed values of coefficient of volume compressibilities, m_{veq} have been shown in col. 5. Table 5.13, and

Table 5.11 - Details of Granular Piles, Pile Caps and Types of Skirting Used in Various In-situ Tests

Site	Test No.	In situ Test	Granular Pile Dimensions (m)		Pile Cap or Footing dimensions (m)		Depth of Skirt (m)	Pile Cap or Footing Base Area 'A' (m^2)	Type of skirting
			Dia.	Depth	Width	Length			
1	2	3	4	5	6	7	8	9	10
CBRI Officer's Hostel Site (Roorkee)	1	Single	0.35	8.0	0.70	0.70	-	0.49	Plain granular piles (Test nos 1 to 7)
	2	Group of 2	0.35	8.0	0.85	1.40	-	1.19	
Molasses Tank site (Budaun)	3	Group of 2	0.35	3.0	0.94	1.25	-	1.175	
CBRI Officer's Hostel Site, (Rooree)	4	Single	0.25	3.5	0.4	0.4	-	0.16	
	5	Group of 2	0.25	3.5	0.4	1.20	-	0.48	
	6	Group of 3	0.25	3.5	1.2m Dia.		-	1.13	
	7	Group of 4	0.25	3.5	1.2	1.2	-	1.44	
	8	Single	0.35	8.0	0.7	0.7	0.40	0.49	Collectively skirted Granular Piles having Cement Concrete reinforced skirt with 1.5mm thick m.s. sheets was used for cast-the skirt in test nos. 8 to 11.
	9	Group of 2	0.35	8.0	0.85	1.4	0.45	1.19	
	10	Group of 3	0.35	8.0	1.6m dia.		0.80	2.0	
	11	Group of 4	0.35	8.0	1.5	1.5	0.80	2.25	

Table 5.11 (Contd..)

1	2	3	4	5	6	7	8	9	10
	12	Group of 5	0.20	3.5	1.0	1.0	0.50	1.0	Prefabricated brick pannel skirting (110cmx50cm) sandwitched with plain cement concrete
	13	Skirted footing without Granular Piles	-	-	1.0	1.0	1.0	1.0	10cmx10cm timber piles driven closely into the ground for test no. 13 & 14 was used as skirting The head of the piles were clamped together by m.s. channel clamps
	14	Group of 4 using timber pile skirting	0.25	3.5	1.0	1.0	1.0	1.0	
	15	Skirted Footing without Granular Piles	-	-	0.9	0.9	1.0	0.81	Prefabricated pipe unit skirting driven together into the ground around the pile cap was used for test no. 15 & 16.
	16	Group of 4 Granular Piles	0.25	3.5	0.9	0.9	1.0	0.81	
Molasses Tank site (Budaun)	17	Group of 4	0.35	3.0	1.8	1.8	1.5	3.24	Collectively Skirted Granular Pile having 20cm thick reinforced cement concrete skirting
CBRI Officer's Hostel (Roorkee)	18	Single	0.25	3.5	0.40	0.40	1.0	0.16	Individually skirted granular piles having 30cm diameter mild steel pipes skirting were used individually around each pile for tests nos. 18 to 21

Table 5.11 (Contd)

1	2	3	4	5	6	7	8	9	10
	19	Group of 2	0.25	3.5	0.40	1.20	1.0	0.48	
	20	Group of 3	0.25	3.5	1.1m dia.		1.0	1.13	
	21	Group of 4	0.25	3.5	1.20	1.20	1.0	1.44	
Drilling Rig Site Silchar (Assam)	22	Single pile	0.60	9.0	7.5	0.7	-	0.525	Plain Granular pile
	23	Group of 2	0.60	9.0	1.75	1.0	-	1.75	Collectively skirted 2 pile group using timber piles 20cm dia., 4m deep with r.c.c. edge beam
Crude Oil Tank site Haldia (W. Bengal)	24	Group of 2	0.55	14.0	2.00	1.0	-	2.00	Plain Granular pile without skirt at spacing 1.4m

Table 5.12 Equivalent Modulus of Deformation (E_{eq}) for the Reinforced Sub-Soil Stratum

Site	Test No.	Insitu Tests	Pile cap or Base area of footing 'A' (m^2)	Cross section area of single pile (m^2)	Total Cross-sectional area of the pile 'A_p' group (m^2)	Coefficient $\alpha = \frac{A_p}{A}$	Value of $(1-\alpha)$	For $E_p = 2300$ t/m^2 The value $(\alpha . E_p)$	For $E_s = 800$ t/m^2 The value $(1-\alpha)E_s$	Equivalent modulus of deformation 'E_{eq}', (9+10) (t/m^2)	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
CBRI Officer's Hostel site (Roorkee)	1	Single	0.49	0.0962	0.0962	0.1963	0.8037	451.49	642.96	1094.45	Plain Granular Piles (Test Nos. 1-7)
	2	Group of 2	1.19		0.1924	0.1618	0.8382	382.49	670.56	1053.05	
Molasses Tank site (Budaun)	3	Group of 2	1.175		0.1924	0.1637	0.8362	376.00	669.00	1045.00	
	4	Single	0.16	0.049	0.049	0.3062	0.6938	704.26	555.40	1259.66	
	5	Group of 2	0.48		0.098	0.2041	0.7959	469.43	636.72	1106.15	
	6	Group of 3	1.18		0.147	0.1245	0.8754	286.35	700.32	986.67	
	7	Group of 4	1.44		0.176	0.1361	0.8638	313.03	691.14	1004.14	
CBRI Officer's Hostel site (Roorkee)	8	Single	0.49	0.0962	0.0962	0.1963	0.8037	451.49	642.96	1094.45	Collectively skirted Granular Piles

Table 5.12 (Contd...)

1	2	3	4	5	6	7	8	9	10	11	12
	9	Group of 2	1.19		0.1924	0.1618	0.8382	382.49	667.36	1049.85	
	10	Group of 3	2.00		0.2886	0.1443	0.8557	331.89	684.56	1016.45	
	11	Group of 4	2.25		0.3848	0.1710	0.8290	393.30	663.20	1056.40	
	12	Group of 5	1.0		0.245	0.2450	0.7550	563.50	604.00	1167.50	
	13	Skirted Footing without granular piles	1.0		-	-	-	-	-	800. 00*	
	14	Group of 4	1.00	0.049	0.1960	0.1960	0.8040	450.80	643.20	1094.00	Collectively skirted granular pile using timber pile skirting
	15	Skirted Footing		-	-	-	-	-	-	800.00*	Prefab. pipe unit skirting without piles
	16	Group of 4	0.81	0.049	0.1960	0.2419	0.7581	556.37	606.48	1162.85	Collectively skirted granular piles using prefab. pipe unit skirting.
Molasses Tank site (Budaun)	17	Group of 4	3.44	0.0962	0.3848	0.1225	0.8775	281.75	698.00	979.75	Collectively skirted granular pile us- r.c.c. skirt
CBRI Officer's Hostel (Roorkee)	18	Single	0.16	0.049	0.049	0.3062	0.6938	704.26	555.04	1259.30	Individually skirted granular piles

Table 5.12 (Contd...)

1	2	3	4	5	6	7	8	9	10	11	12
	19	Group of 2	0.48		0.098	0.2041	0.7959	469.43	636.72	1106.15	
	20	Group of 3	1.13		0.147	0.1245	0.8754	286.35	700.32	988.87	
	21	Group of 4	1.44		0.196	0.1361	0.8638	313.06	691.11	1004.14	
Drilling rig site	22	Single pile	0.525	0.3	0.3	0.571	0.429	1713.0	85.8	1871.6	Plain pile
Silchar (Assam)	23	Group of 2	1.75	0.3	0.6	0.6	0.40	1800.0	180.0 (1)	1980.00 (3)	Collectively skirted pile
Oil Tank site Haldia (W. Bengal)	24	Group of 2	2.0	0.237	0.474	0.237	0.763	1185.0	152.6 (2)	1337.60 (4)	Plain pile

For pile diameter, depth, size of the pile cap and details of material of skirt refer Table 5.11 corresponding to the test numbers.

*Equivalent modulus of deformation for virgin soil without granular piles

(1)&(2) The value of E_p is taken as 3000 and 5000 t/m^2

(3)&(4) The value of E_s is taken $200 t/m^2$

col. 6 shows the values of predicted settlements for plain granular piles under the intensity of stress $20t/m^2$. The total settlements, S was found out in two parts viz., the settlement of the reinforced soil, L using equation 3.79 and the settlement under the granular pile toe, H by equation 3.80 Thus the total settlement was found out by equation 3.81

In the case of skirted granular piles, the total settlement under the footing, s was computed in three parts viz. the settlement in skirted reinforced soil plug, s, the settlement in un-skirted reinforced soil L and the settlement in the strata below the granular pile toe. The settlements L and H were computed in the same way as for the plain granular piles. Thus the total settlement(s) were calculated by equation 3.94. The calculated settlements for various skirted granular pile foundations are shown in col. 6 of the Table 5.13. The observed values of settlements from various stress-deformation curves were obtained for a stress of $20t/m^2$ and are shown in col. 7 (Table 5.13). The study of the ratio of predicted and observed settlements (col. 8, Table 5.13) show a reasonably good agreement. The variation in the ratio of settlement computed/observed in general found to be between 0.74 to 1.04 which within a narrow range indicates the validity of the proposed approach for settlement computation.

Table 5.13 - Observed & Predicted Settlements of Plain & Skirted Granular Piles

Site	Test No.	In situ Tests	Equivalent 'E_{eq}' (t/m^2)	Coeff. of volume decrease 'm_{veq}' $x10^{-4}$ (m^2/t)	Computed Settlement for $20t/m^2$ stress (mm)	Observed Settlement for $20t/m^2$ (mm)	Ratio of computed/observed Settlement	Remarks
1	2	3	4	5	6	7	8	9
CBRI Officers Hostel Site (Roorkee)	1	Single	1094.45	9.1370	17.0	16.25	1.04	Plain granular piles (Test nos 1 to 7)
	2	Group of 2	1053.05	9.4962	26.0	19.5	1.33	
Molasses Tank site (Budaun)	3	Group of 2	1045.00	9.5693	5.56*	5.75*	0.96	
CBRI Officers Hostel (Roorkee)	4	Single	1259.66	7.9386	4.4	4.8	0.91	Plain granular piles
	5	Group of 2	1106.15	9.0403	7.4	8.0	0.92	
	6	Group of 3	986.67	10.1351	15.3	16.0	0.96	
	7	Group of 4	1004.14	9.95	15.85	17.5	0.90	
	8	Single	1094.45	9.1370	7.0	7.75	0.90	Collectively skirted granular piles (Test nos. 8 to 11) 35cm Ø, 8m deep piles
	9	Group of 2	1049.85	9.5251	9.3	10.8	0.86	
	10	Group of 3	1016.45	9.8381	20.0	19.25	1.03	
	11	Group of 4	1056.40	9.4661	15.5	15.0	1.03	
	12	Group of 5	1167.50	8.5653	9.0	6.7	1.34	Collective skirting using Brick pannels

Table 5.13 (Contd..)

1	2	3	4	5	6	7	8	9
	13	Skirted footing without granular piles	800.00	12.5**	9.0*	11.8*	0.762	Using Timber pile skirting
	14	Group of 4	1094.00	9.1407	10.6	8.45	1.25	Collectively skirted granular piles using timber pile skirting
	15	Skirted footing without granular	800.00	12.5**	10.0*	11.8*	0.847	Using prefabricated pipe unit skirting.
	16	Group of 4	1162.85	8.6	9.3	9.0	1.03	Collectively skirted granular piles using prefab. pipe unit skirting.
Molasses Tank site (Budaun)	17	Group of 4	979.75	10.2061	16.6	15.0	1.10	Collectively skirted granular pile using r.c.c. skirt
CBRI Officers Hostel site (Roorkee)	18	Single	1259.30	7.9409	2.72	3.0	0.9	Individually skirted granular piles using 30cm dia. m.s. pipe (Test nos 18 to 21)
	19	Group of 2	1106.15	9.0403	4.25	5.3	0.8	
	20	Group of 3	988.87	10.1391	12.0	11.5	1.04	
	21	Group of 4	1004.14	9.95	10.78	10	1.07	
Drilling rig site Silchar	22	Single pile	1871.6	5.34	5.4	6.0	0.9	Single plain granular pile
	23	Group of 2	1159.82	8.62	13.4	11.25	1.19	Collectively skirted
Oil Tank site Haldia (W.B.)	24	Group of 2	1337.6	7.47	13.75	10.0	1.37	Plain Granular Piles

Note: For Pile dia., depth, size of the pile caps, material of skirt, refer table 5.11 corresponding to the test numbers
The computed settlement for test no. 1&2 in col. 6 is doubled due to submergence
*Settlement for an intensity of stress equal to $10t/m^2$. **Coefficient of volume decrease & settlement for virgin soil without piles.

CHAPTER VI

APPLICATIONS AND ADVANTAGES

6.1 General

Skirts around shallow foundations have been used in the past with some constructions often without realization of basic characteristics that make their use advantageous. For example failure of the clutches of 4.6m deep sheet piles provided under the edge of a steel tank to prevent the lateral flow of 3.7m soft clay overlying sand and firm clay, (Penman and Waston, 1963). The reasons of failure were attributed to excessive lateral forces developed due to presence of lenses of peat resulting into tearing of sheet piles. Skirts made of sheet piles, reinforced concrete or gravel around storage tanks have also been used (Roberts 1961, 1976), Fig. 6.1. Skirts have also been used in offshore structures (Fig. 6.2) primarily intended for driving and stability considerations (Hoeg 1976 & DiBiagio et.al 1976).

In describing the use of deep rings or skirts of crushed rocks Robert (1976) stated that 'The dimensions of the (crushed metal/gravel) ring (or skirt) must be such that the resistance to lateral movement of the skirt will be greater than the outward thrust of the over stressed soil. Thus through a combination of passive resistance and the base friction of the gravel ring, the outward movement of the localized weak layer can be prevented'. The foregoing statement brings out the importance of the dimensions, relative stiffness of the components and also the kinematic behaviour during the total loading range. The present investigation offers an improved understanding of the behaviour of skirted granular pile foundations imparting rationality and confidence in their use.

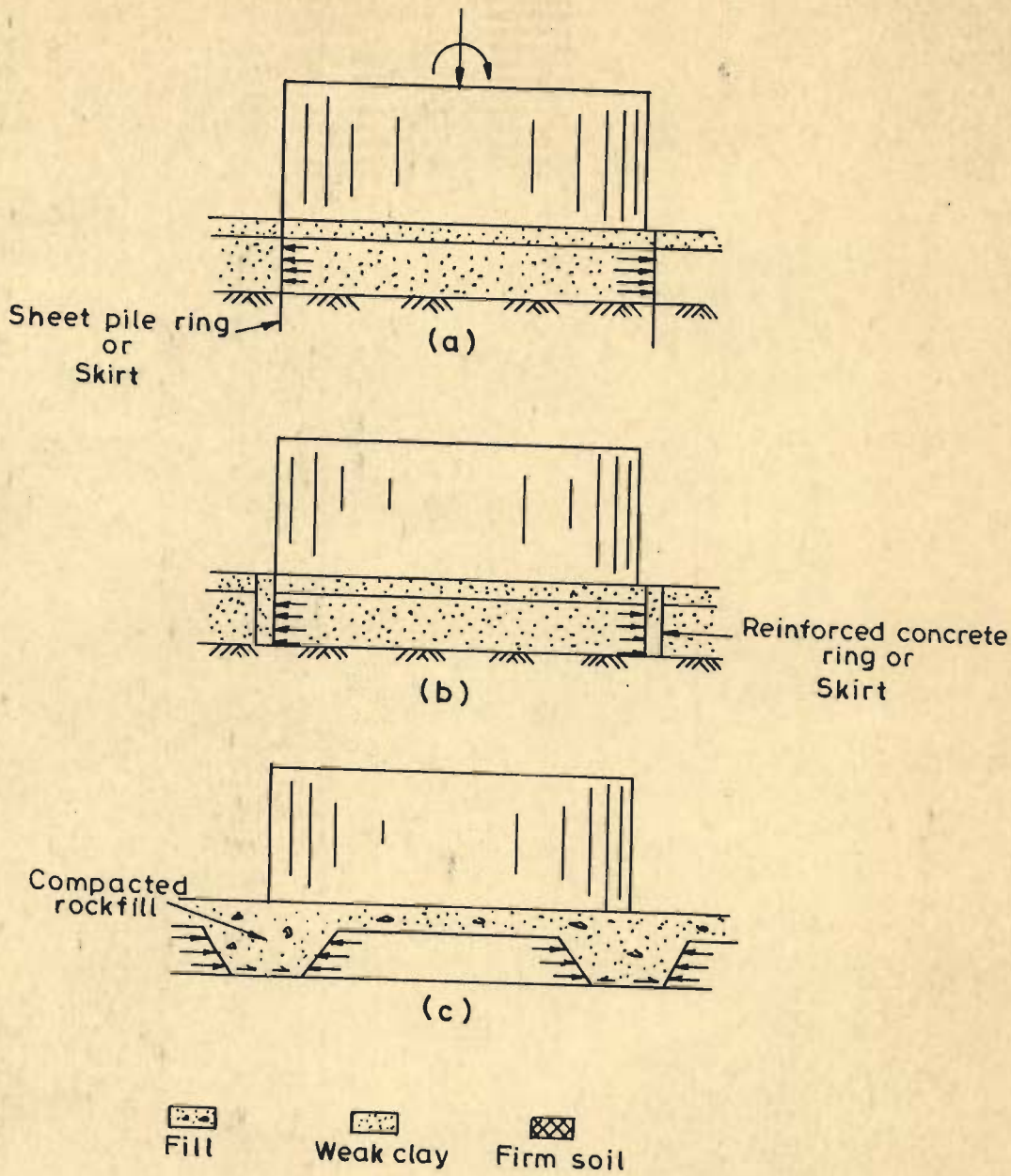


FIG.6.1 – RING OR SKIRTS FOR (LARGE) STORAGE TANKS
(AFTER ROBERTS 1961 AND 1976)

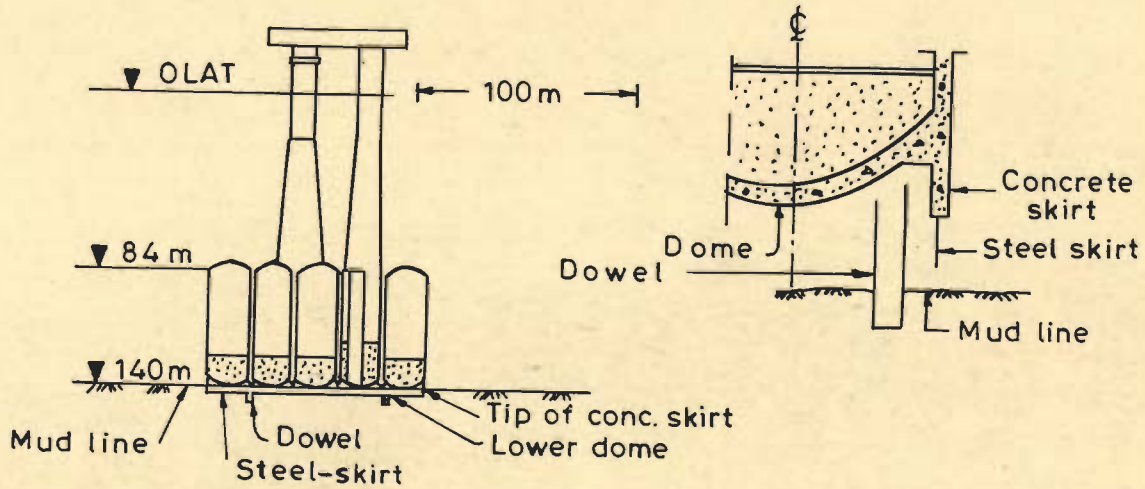


FIG.6.2-TYPICAL OFF SHORE STRUCTURE WITH SKIRTS (AFTER HOEG 1976 AND DIBIAGIO et.al 1976)

6.2 Potential Application Possibilities

In view of the potential advantages of the skirted granular piles, their use may lead to considerable economy without sacrificing safety for design of foundations in general and for complex construction in particular. They can be used advantageously as foundations for the following multitude of civil engineering constructions:

(a) To sustain high compressive loads with restricted settlements.

(b) To sustain high pull out loads with limited pull-movement

(c) Off-shore and marine structures.

(d) Erosion protection.

(e) Foundations for temporary structures.

(f) To resist blasts and earthquake loads besides densification of loose to medium dense saturated cohesionless sub-soil deposits to reduce the liquefaction potential.

(g) As strengthening measures for foundations under distress.

6.2.1 Foundations for Civil Engineering Structures

Skirted granular pile foundation may find application as a suitable economical alternative to existing foundation methods for all civil, military and industrial structures especially when the sub-soil conditions are not favourable. Skirt of limited penetration have been used in gravity type of off-shore structures from driving and stability considerations.

Situations where by necessity one has to use high loads with

restricted settlements, such as column footings in a framed structures, foundations of radars, transmission towers, Fig. 6.3, particle accelerators and machine foundations. Collectively skirted reinforced soil plug is likely to provide higher stiffness affecting the natural frequency of soil foundation system, Fig. 6.4.

6.2.2 Foundations subjected to High Pull Loads with Limited Pull Movement.

Skirted granular pile foundations may provide a better foundations for structures transmitting high uplift loads to the sub structure such as - suspension bridges, sport stadia, transmission line towers, radars and antennae etc. (Fig. 6.3).

6.2.3 Foundations in Flood Prone Zones

There are areas which are subjected to heavy floods during monsoon every year. The skirted granular pile foundations may help in checking the scour due to flood waves. Under the shallow footings, as it may find application as efficient foundations for bridge piers. Though granular piles have been used in the past under bridges (Engelhandt and Krisch 1977), the provision of skirting consisting of prefabricated pipe units or precast/prestressed concrete pile or even timber piles driven around the granular pile group and made rigid at the foundation bed level by r.c.c. beam may go a long way in checking erosion/scour near the bridge piers.

6.2.4 Foundations for Temporary Structures

The present day economy of any country all over the world is directly dependent on reserves of oil it possess. This calls for extensive efforts towards search for oils both on shore and off-shore. Oil

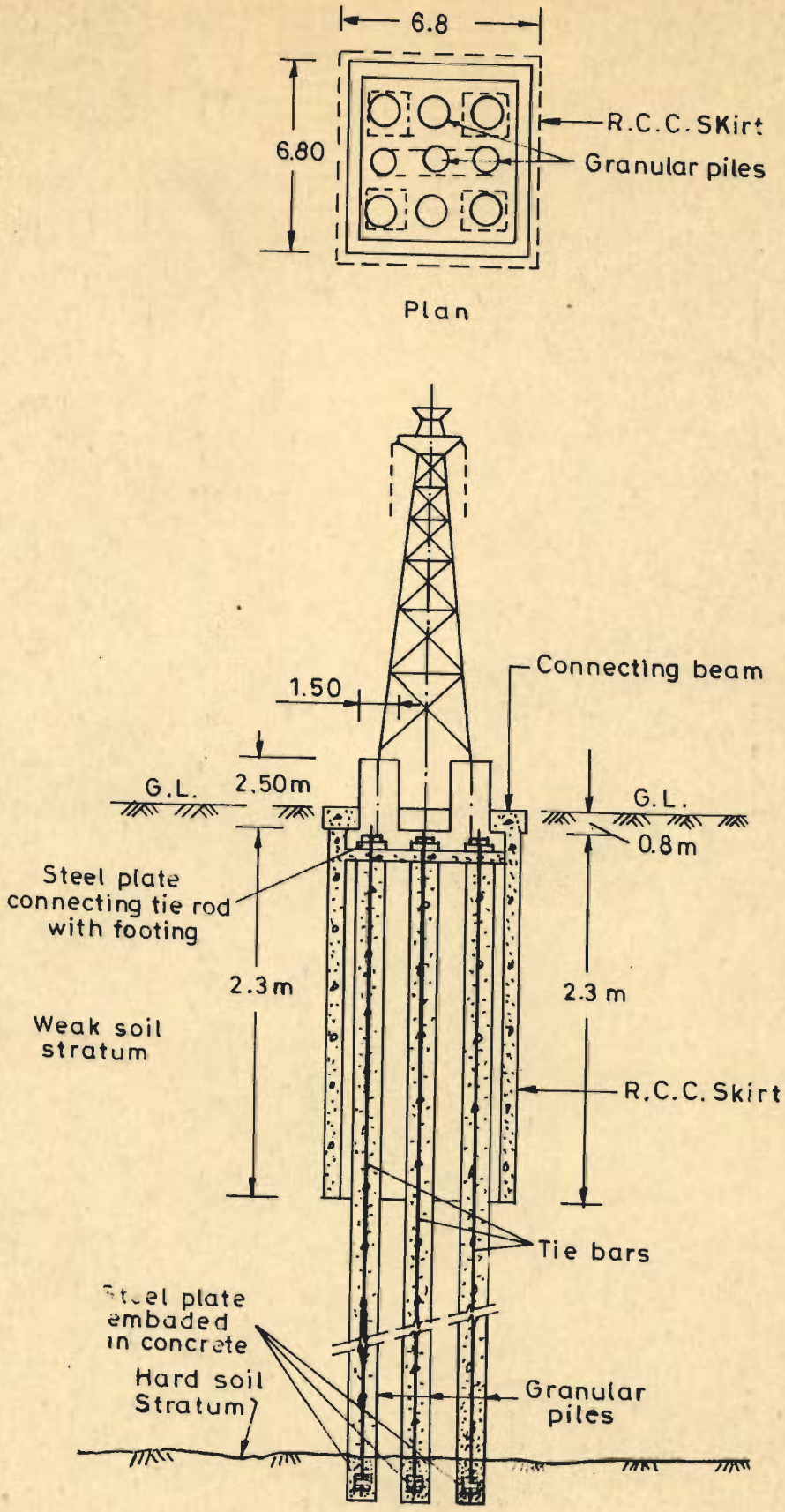


FIG.6.3-SKIRTED GRANULAR PILE FOUNDATION FOR TRANSMISSION LINE/ANTENNAE TOWERS

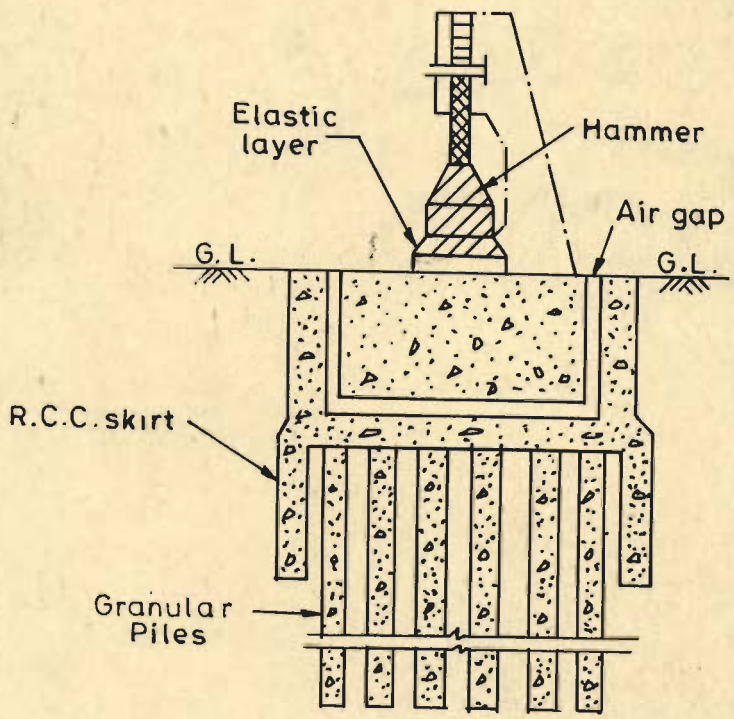


FIG.6.4-HAMMER FOUNDATIONS ON SKIRTED GRANULAR PILES

drilling on-shore calls for a suitable foundation to enable the driller to errect the drilling rig at a particular location. These rigs in general transmits a load to the foundation soil system which ranges between 1 kg/cm^2 to 2 kg/cm^2. The functional requirement demands for almost negligible total and differential settlements between different footings to support the rig load. In the case of sub-soil stratum consisting of loose to medium dense cohesionless soils or clay deposits having soft to medium consistancy upto a considerable depth, the designer is left with no choice than to adopt cast in situ driven or bored piles. This too is considered as one of the best engineering solution, however the method calls for transporting the heavy pile rig to the site situated in remote difficult unapproachable areas. Thus the high mobilisation cost leads to uneconomy. In such a situation skirted granular pile foundations may provide an economical alternative to pile foundations which does not require heavy rigs and the method of construction is based on indigenous knowhow.

In the case of oil drilling rigs, the foundation, is required for a specific period, the skirted granular pile foundation,(Fig. 6.5) may be designed for retrievable type of skirting using prefabricated pipe units, precast/ prestressed piles, pipe piles or timber piles held rigidily at the foundation level. Thus the cost of the skirt may be reduced considerably due to re-use value on different sites. Also, the concrete footing or raft placed on the group of granular piles, may be replaced by wooden slippers or grillage foundation which could also be transported and shifted to the next site alongwith the skirting piles(or units) and re-used.

6.2.5 Foundations in Weak Sub-Soil Deposits

As discussed in art. (6.2.4) when the foundation is to be designed in

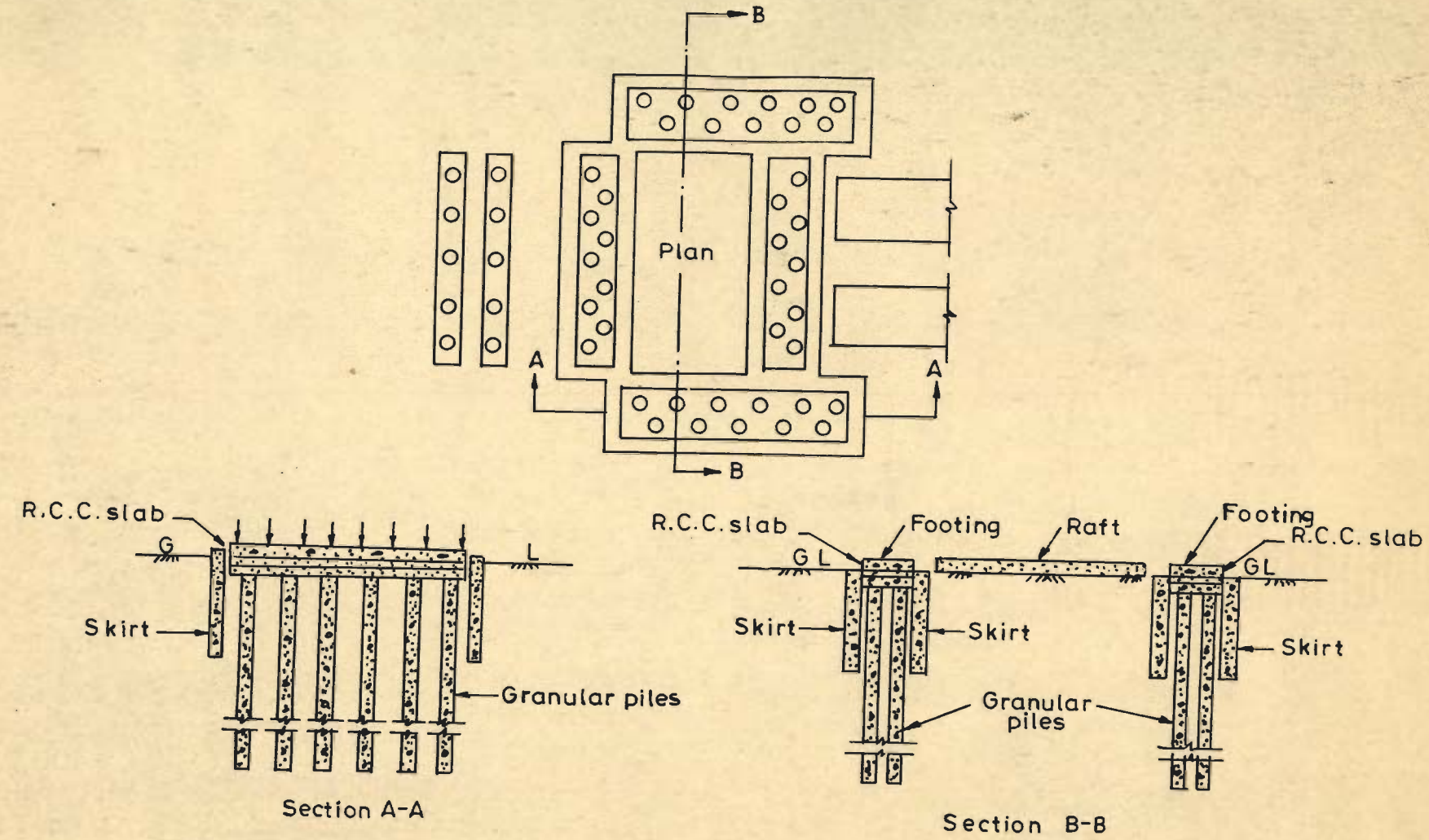


FIG.6.5 – SKIRTED GRANULAR PILE FOUNDATION FOR ON SHORE OIL DRILLING RIGS

sub-soil stratum having low bearing capacity and design loads are high, the designer has to think of special techniques of ground improvement methods which needs time and money both. In such situation, plain granular piles, in all types of weak sub-soil provide an ideal and economical solution with least time. Because of the fact, that during the construction of piles, the strength of the soil improves and hence the bearing capacity increases and settlement reduces.

6.2.6 Foundations in Soil Deposits Prone to Liquefaction

Loose-medium dense saturated sands and silts deposits when subjected to earthquake loads are likely to liquefy. In such situations installation of granular piles in such deposits is likely to help to a great extent in reducing the liquefaction potential of the soil deposit due to densification of deposit and rapid dissipation of excess hydro-static pressure built up during earthquake (Greenwood 1970, Seed & Brooker 1976, Engelhardt and Krisch, 1977). Further if the granular piles are skirted, the factor of safety for base shear due to horizontal movement of the footing in the area of high seismic susceptibility increases significantly (Engelhardt and Golding 1975). Thus the foundation may find potential application in atomic power plants where rigid criteria are required to be adhered.

6.2.7 Foundations Under Distress

In situations where foundations are under-going excessive settlements total or defferential or where more loads are required to be added on existing foundations, provision of skirt or skirted granular piles may provide an economical solution, Fig. 6.6.

If high loads are to be sustained due to addition of more storeys, a system of precast pegs, Fig. 6.7 and skirted granular piles may

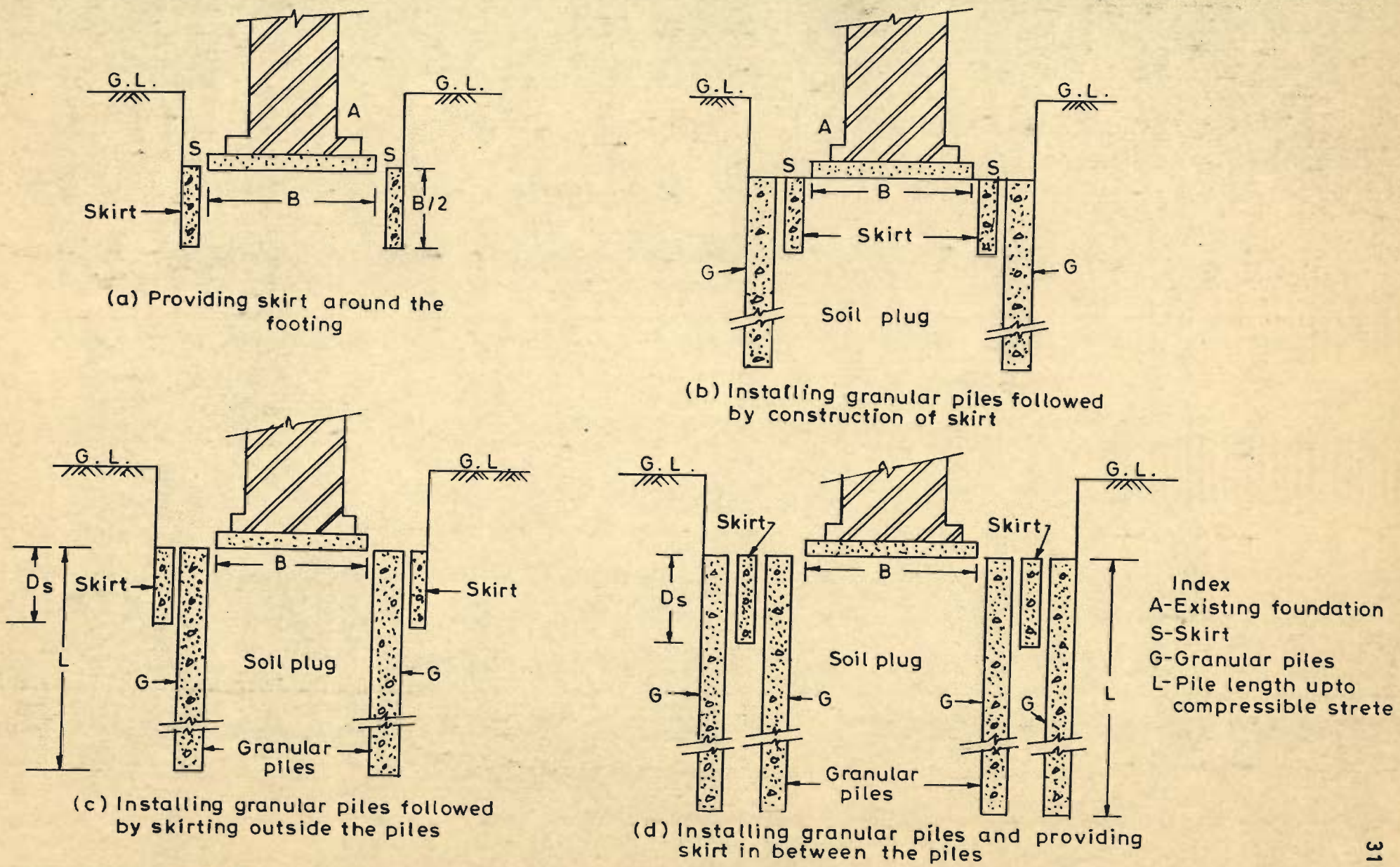


FIG.6.6- STRENGTHENING OF EXISTING FOUNDATION UNDER DISTRESS

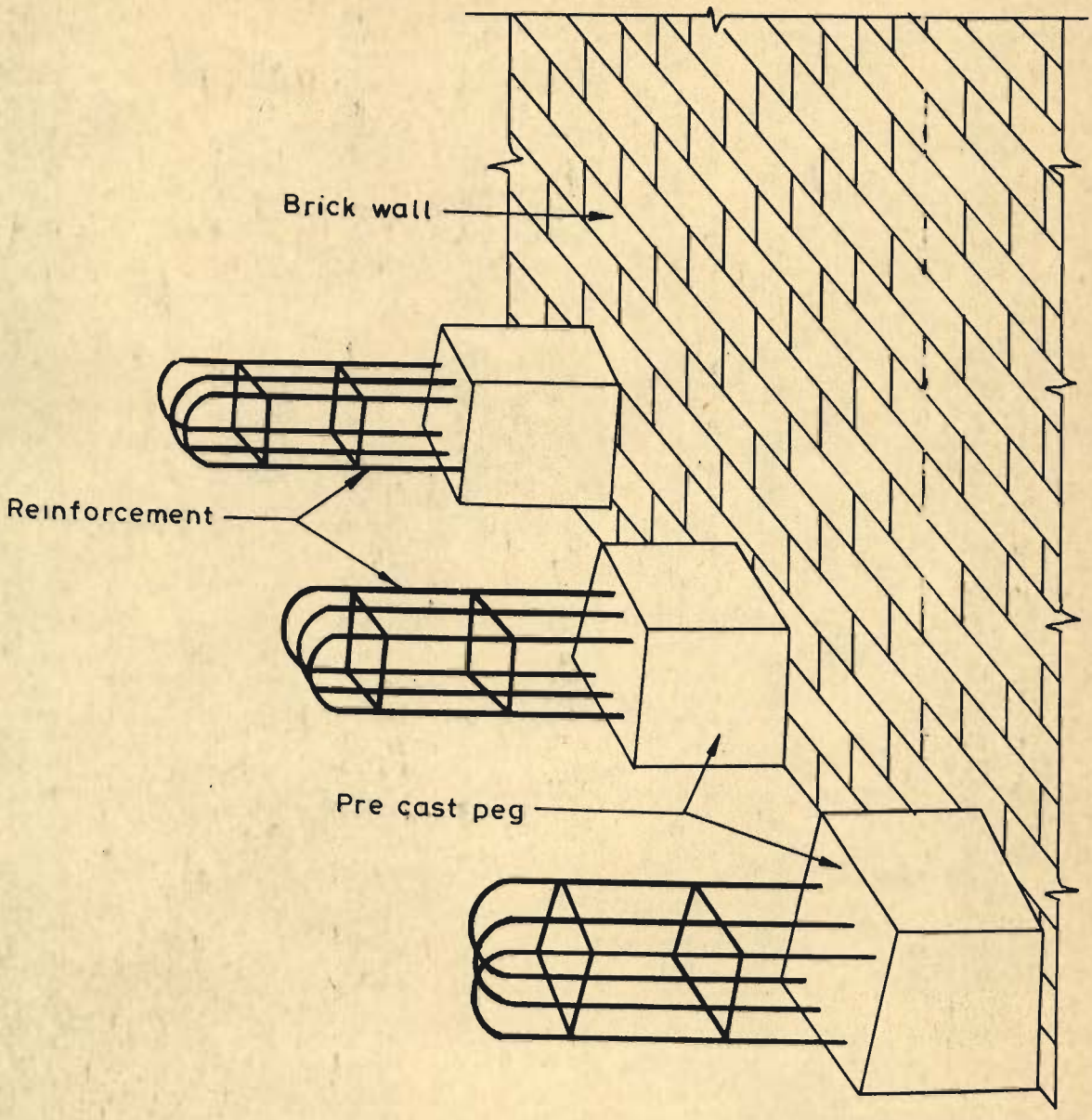


FIG.6.7. PRECAST PEG-TECHNIQUE

be used. The connecting beam may be cast insitu, resting on skirted granular piles (Fig. 6.8).

6.2.8 Heavy Storage Tank Foundations

Pile foundations are seldom used for heavy storage tank foundations due to large size of the tank, requiring thick pile caps (Robert 1976). Generally pad foundations are used if the bearing capacity of the sub-soil stratum is in accordance with the design load and anticipated settlements (total and differential) are within allowable limits. However, if these criterias are not satisfied, the designer is compelled to change his decisions and look for special foundation methods. In weak sub-soils tank foundations are known for incipient failures due to excessive differential settlements causing tearing of the shell plate, effecting the working of the floating roofs due to excessive edge distorsion (Bjerum and Overland, 1957, Mohan et.al 1977, Green and Height 1975, and Penmann and Watson 1963). In such situations skirted granular piles may be used, Fig. 6.9 with success.

6.3 Application to Live Structures

Since the developments during last five years of the skirted granular pile foundation, various structures have been designed and constructed/ under construction on this foundation. The details have been provided in Table 6.1.

6.4 Advantages of Skirted Granular Pile Foundations

Skirted granular pile foundation has several advantages over the existing foundation practices being being followed presently

These advantages are:

(a) Construction method is based on indigenous equipment and know how and does not require imported machines such as

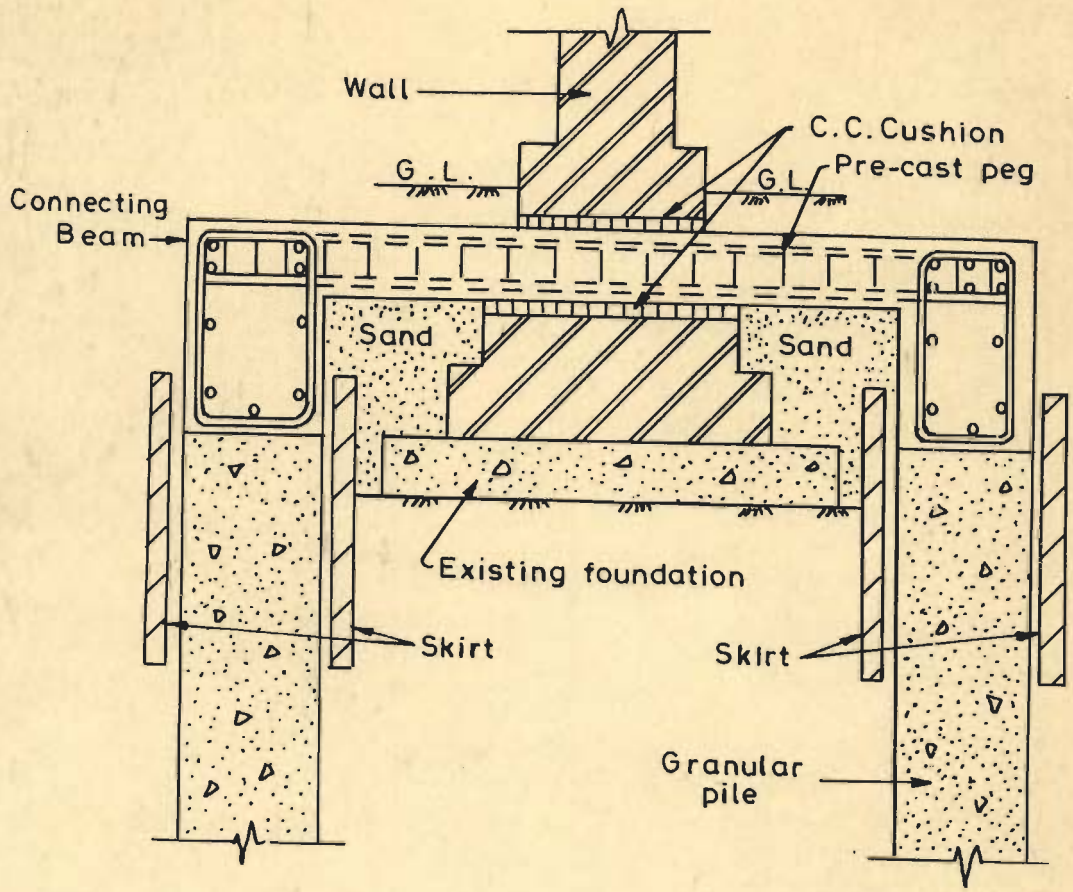


FIG.6.8-IMPROVING THE CAPACITY OF EXISTING FOUNDATION BY PRECAST PEG AND SKIRTED GRANULAR PILES

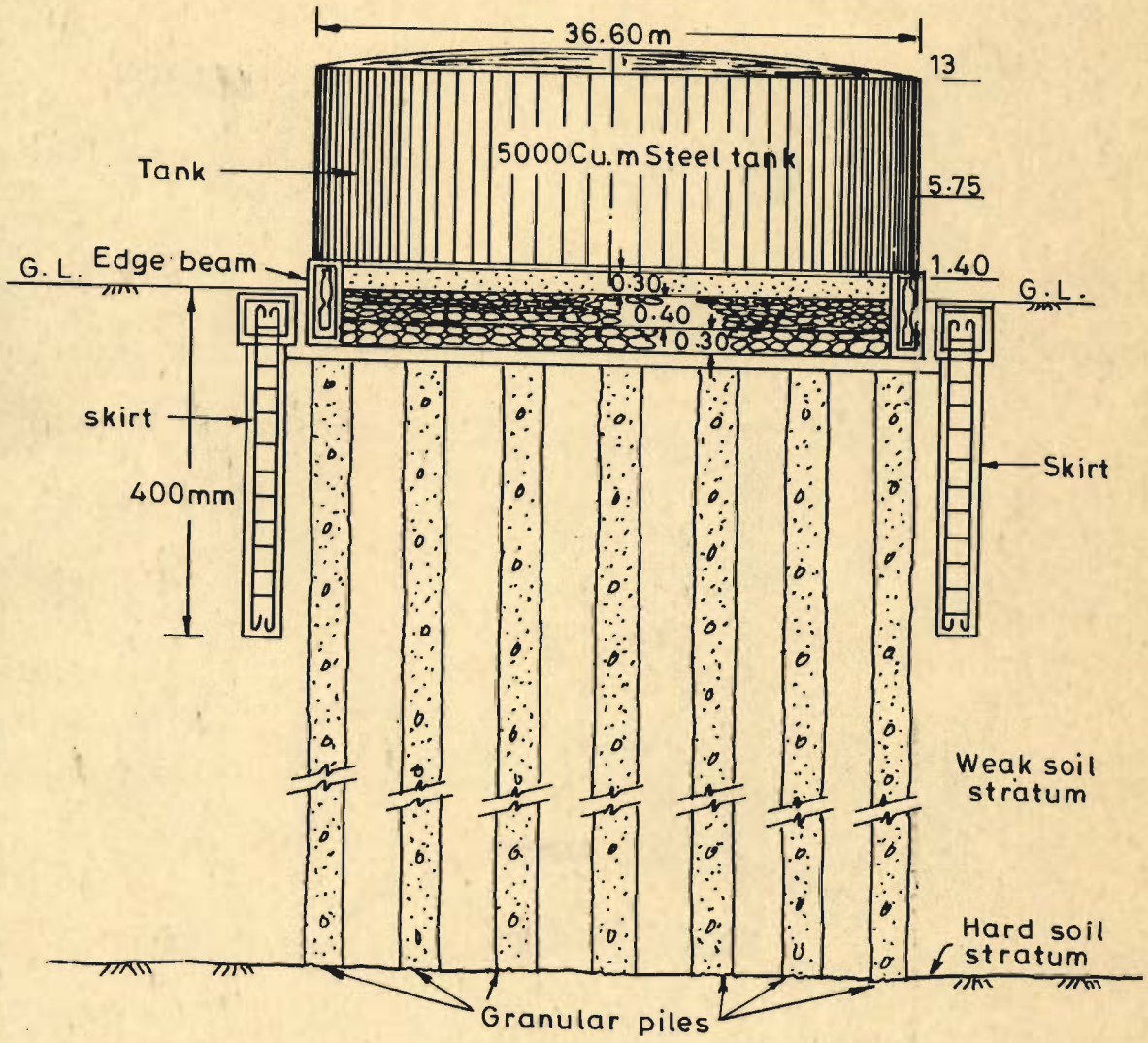


FIG.6.9. HEAVY STORAGE TANK ON SKIRTED GRANULAR PILE FOUNDATION

Table 6.1 - Application to Live Structures

Type of Structure	Details of Foundation Provided					Soil Type and depth (m)	Location	Sponsorer
	Granular Pile		Skirting Details					
	Dia (cm)	Depth (m)	Type	Thick-ness (cm)	Depth (m)			
1	2	3	4	5	6	7	8	9
1. Molasses Tank Foundation								
(a) 24m dia., 6.75m ht., 3000 cu.m capacity*	35	3.0	RCC sand witched in brick wall	45	1.5	loose sand deposit (3m)	Budaun (UP)	Adviser Civil, UP Coop. Sugar Federation, Lucknow (UP)
(b) 24m. dia., 6.75m ht, 3000 cu.m. capacity	35	3.5	-do-	45	2.0	Saturated sandy silt (3m)	-do-	-do-
(c) 20m dia., 9.5m ht, 5000 cu.m. capacity	35	4.0	RCC	45	2.0	Clayey silt (3.5m)	Bazpur (UP)	G.M. Coop. Sugar Factory Bazpur Nainital (UP)
2. Strengthening of a Ring foundation for a over-head water storage tank	55	7.0	RCC	25	3.0	Saturated silt (6.5m)	Gangoh (UP)	E.E. UP Jal Nigam, Saharanpur (UP)
3. Oil Drilling Rumanian Rig (F-400-4 DH)	60	9.0	Timber piles with rcc beam at top	20cm dia. pile with 60x40cm rcc beam	4.0	Clayey silt (saturated) (16m)	Silchar Masimpur site (Assam)	Chief Engineer (Civil) ONGC, Dehradun (UP)

Table 6.1 (Contd...)

4. Furnace Oil Tanks 3 nos, 13500 cu.m. capacity (42m dia., 9.5m ht)	55	11.0	RCC	40.0	3.5	Soft clay (10m)	Haldia (W. Bengal)	Chief Project Manager, IOCL, 26 Kasturba Gandhi Marg, Kailash Building, III floor New Delhi
5. Mandatory Crude Oil Tank 65000 cu.m. capacity (79m dia., 14.5m ht)	55	15.0	RCC	40.0	4.0	Soft clay (15m)	-do-	-do-

*Tank has since been completed. Full water load maintained for 15 days during water load test. No settlement or distortion was observed. The tank is in operation without any trouble.

NB: Full scale pile load tests on all the five sites have indicated good agreement with design predictions. These foundations are under construction.

vibroflot, pile driving rigs etc.

(b) The foundations uses the local semi-skilled labour and material.

(c) Does not require preload and thus saves the cost of preloading material and time for preloading.

(d) Provides economical foundation in weak sub-soil and method of construction is fast.

(e) Retrievable type of skirting for temporary structures reduces the cost significantly.

(f) The foundation may be used with success in areas of high seismic susceptibility.

CHAPTER VII

CONCLUSIONS

7.1 Introduction

Full scale field tests and analytical study related to plain and skirted granular piles, single and in groups in different soil conditions, under vertical compressive, uplift and lateral loads have been carried out. Based on the study, the following conclusions are drawn:

7.2 Experimental

7.2.1 Construction Technique

A simple and speedy construction technique of granular pile based on indigenous know-how, man power oriented technology avoiding the necessity of sophisticated machines has been developed. The technique is useful for construction of granular piles with different dia. and depth. Adopting this technique, plain granular piles with maximum pile diameter of 60cm and lengths upto 14m in different soil deposits have been successfully constructed.

7.2.2 Vertical Compressive Loads

(a) Plain Granular Piles

In-situ load tests on full scale plain granular piles single and in groups of 2, 3 and 4 installed in loose-medium dense cohesionless soil deposits and groups of 2 piles installed in deep layers of soft clayey silt and soft clay deposits respectively indicate a significant increase in load carrying capacity and considerable reduction in settlement over the virgin soil.

(b) Settlement of Plain/Skirted Granular Piles

Full scale in situ load tests on groups of 3 and 4 plain granular piles indicated a settlement reduction of 76 per cent over the settlement of a plain footing cast on the virgin soil under a load intensity equal to the ultimate bearing capacity of virgin soil. When the pile groups are skirted (individually or collectively) the reduction is found to be 86 per cent under the same intensity of stress. In case if the skirted pile group is provided with a central unit pile, the reduction in settlement is noted to be as 94 per cent.

(c) Effect of Skirting

Skirt (individually or collectively) around granular piles further increases the load carrying capacity of granular piles with further reduction in settlements.

(d) Material for Skirting

In-situ tests were carried out on groups of granular piles skirted collectively/individually with skirts of different materials. RCC skirt may be considered to be the best. However, if skirts using brick pannels, closely driven pipes, precast/pre-stressed concrete piles, timber piles are to be used it is necessary to suitably connect all the pile heads at the foundation level. Prefabricated pipe unit skirting has the advantage that these are retrievable and hence can be reused particularly for structures requiring short term use.

7.2.3 Vertical Pull out Loads

Pull-out load tests on full scale plain granular piles single and in groups indicated high uplift capacity with negligible pull-movement in loose cohesionless soil deposit. Test results also indicate that the pull-out

capacity of a unit pile cast in the virgin soil may be increased significantly by providing a granular pile row around it, and collectively skirting them. Further if the unit pile and the granular pile group is subjected to pull-out load together the pull out capacity of the skirted granular pile system maybe be increased about four fold over the unit pile cast in the virgin ground.

7.2.4 Stress-Deformation Characteristics

In-situ tests data on weak sub-soil stratum, when subjected to compressive or uplift loads, indicated a non linear behaviour till the ultimate bearing capacity is reached. Beyond this stress,it indicates a fully plastic behaviour.When the deposit is reinforced with plain/skirted granular piles (collectively or individually) the soil-pile system changes from non linear to an elastic behaviour changing to an elasto-plastic behaviour till plastic yield point (ultimate capacity) is reached. Beyond the plastic yield point a complete plastic behaviour is noted. The elastic and plastic yield points are found to be reached at an average deformation/pull movement equal to 5 and 10 per cent of the installed pile diameter respectively.

7.2.5 Group Efficiency

Based on in-situ test results on full scale plain and skirted (collectively or individually) granular piles single and in groups cast in loose-medium dense cohesionless soil deposits subjected to vertical compressive or pull-out loads, the group efficiency factor at a spacing of 3 times the pile diameter is unity. Also, the group efficiency factor is noted to be independent of the number of piles in the group.

7.2.6 Horizontal Loads

Full scale lateral load tests on skirted granular piles with a central

unit pile cast in loose-medium dense cohesionless soil deposit indicate that the safe lateral load capacity of skirted granular pile group with central unit pile increases by about four times over the unit pile lateral capacity.

7.2.7 Behaviour of Skirted Granular Piles with a Central Unit Pile Under Compressive & Up-lift Load

The ultimate load carrying capacity of 4-granular pile group with a unit pile cast in the centre and collectively skirted was found to increase 4 times over the ultimate capacity of the virgin soil under the compressive loads. It was also noted that the stress deformation behaviour of the 4 granular pile group with a unit pile, collectively skirted and subjected to compressive loads, compared well with collectively skirted 5 granular pile group with similar conditions suggesting that the unit pile may be replaced with that of granular pile. Also the skirted granular pile system with unit pile in the centre, when subjected to uplift load (applied on granular piles and unit pile) indicated a behaviour similar to that under compression tests.

7.3 Analytical

7.3.1 Ultimate Loads in Compression & Pull-out

Proposed analytical approach is found to give reasonably good values of ultimate load capacity of single and group of plain and skirted granular piles under vertical compressive and also pull out loads in different soil conditions. The predicted ultimate loads compare well with full scale insitu load test results.

7.3.2 Settlement

The proposed approach for settlement computations for plain and skirted granular pile groups predicts values which are noted to be in good agreement with the observed settlements.

7.4 Utility for Structures

Based on the proposed construction and design methodology, skirted granular pile foundation for a 3000 cu.m. capacity molasses steel tank (24m diameter, 6.75m height) were constructed. 230 granular piles, 35cm diameter, 3m long at a spacing of 3.5 pile diameter collectively skirted by a rigid skirt were constructed in a loose-medium dense cohesionless soil deposit. The skirt consisted of a 20cm thick, 2.20m deep r.c.c. wall. Subsequent to the completion of the foundation and the tank, the tank foundation was water load tested to its full capacity. The water load was maintained for a period of 15 days during which negiligible settlement was noted. The tank is under operation, since February, 1982.

CHAPTER VIII

SUGGESTIONS FOR FUTURE RESEARCH

On the basis of the present study the following suggestions may be helpful for further research:

8.1 Actual load transfer mechanism in a plain granular pile and pile group system along the pile length need be studied.

8.2 There is a need to study the sharing of the applied load on the soil-pile system as it is not well understood. Thus measurements of pore pressures by suitable instrumentation in granular pile/soil system need be studied to have a feed back data.

8.3 Transfer of stress from footing/soil plug reinforced with granular piles is another area which calls for fully instrumented measurement for better understanding of the load transfer mechanism and the pressure distribution below the skirt.

8.4 Influence of pile spacing in different soil conditions on the load/settlement characteristics of plain granular pile group should be investigated with a view to have a optimum spacing.

8.5 Effect of variation in pile length on the load settlement behaviour of single and groups of piles need be studied with an objective to optimize the pile length with respect to bearing and bulging failure modes.

8.6 Skirted Granular Pile Foundation should be studied under uplift loads in soft cohesive sub-soil deposits with greater emphasis on failure pattern mechanism under pull loads.

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Computation of ultimate load carrying capacity of a plain granular pile in Loose to Medium dense Cohesionless soil deposit.

(a) Input Data

Effective angle of shearing resistance, $\emptyset'$	$= 31^{o}$
Effective cohesion, c'	= 0
Bulk density soil, γ_{bulk}	$= 1.9 t/m^3$
Bulk density of pile material, γ_{bulk}	$= 2.2 t/m^3$
Pile length	= 8m
Pile diameter (assumed)	= 35cm
Static Cone Resistance, q_c	$= 32\ kg/cm^2$
Standard Penetration Value N_{SPT}	= 8

(b) Coefficient of Lateral Earth Pressure, K_o

$K_o = 1-(\sin 1.2\ \emptyset')$ (3.29)

Substituting proper values

$K_o = 1-\sin(37.2)$ $= 0.3954$

(c) Poisson's Ratio, μ

$$\mu = \left(\frac{K_o}{1+K_o}\right) \quad (3.33)$$

Substituting proper value =

$$\mu = \frac{0.3954}{1.3954} = 0.2$$

(d) Effective Normal Stress, σ_v

σ_v is found at the critical depth, L_c taken equal to 5 times the pile diameter

$$= 5x0.35x1.9 = 3.325 t/m^2$$

(e) Effective Mean Normal Stress, σ_m

$$\sigma_m = 1/3\ (1+2K_o)\ \sigma_v \quad (3.31)$$

Substituting suitable values

$\sigma_m = 1/3\ (1+2x0.3954)\ 3.325$
$= 1.9848 t/m^2$

(f) Soil Modulus, E_s

$$E_s = 2(1+D_r^2)\ q_c \qquad (3.66)$$

Taking $D_r = 0.5$ (Table 3.3) and substituting values in equation 3.49

$E_s = 2(1+0.5)\ 320$

$= 800 t/m^2$

(g) Corrected Soil Modulus, E'_s

$$E'_s = E_s\left[\frac{\sigma_m}{10}\right]^{0.5} = 356.4 t/m^2 \qquad (3.49)$$

(h) Rigidity Index, I_r.

$$I_r = \frac{E'_s}{2(1+\mu)(\sigma_m \tan \emptyset')} \qquad (3.57)$$

Substituting proper values

$$I_r = \frac{356.4}{2(1+.2)\ (1.9848x0.6)}$$

$= 125$

(i) Cavity Expansion Factor, F'_q

Referring to Fig. 3.4, for c=0, $\emptyset=31^o$ and $I_r = 125$, $F'_q = 8$

(j) Limit Lateral Stress, σ_{rL}

Referring eqn. (3.53)

$$\sigma_{rL} = \sigma_m F'_q \qquad (3.52)$$

Substituting suitable values $\sigma_{rL} = 1.9848x8=15.878 t/m^2$

(k) Ultimate Load Capacity when Pile Top is Loaded, q_{ult1}

$q_{ult} = 6 \times \sigma_{rL}$

Substituting $_{rL}$ values

$q_{ult\ 1} = 6x15.878 = 95.27t/m^2$

(l) Corrected Pile Modulus, E'_p

$$E'_p = E_p\left[\frac{\sigma_m}{10}\right]^{0.5} \qquad (3.54)$$

Where

$E_p = 2(1+D_r^2)\ q_{cp}$

$q_{cp} = 70\ kg/cm^2$ (from test)

Substituting values in equation 3.49

$E_p = 2(1+0.8^2)\ 700 = 2300t/m^2$

It may be noted that the value of E_p for compacted granular pile have been found as high as 600 K_p/cm^2 (Engelhardt & Krisch 1977) for the piles made by vibroflots. However for small hammers (125-500kg) value of E_p between 2400 to $5000t/m^2$ appears reasonable

$$E'_p = 2300\ \left[\frac{1.9848}{10}\right]^{0.5}$$

$$= 1024.67\ t/m^2$$

(m) Ratio $E'_p/E'_s = \frac{1024.67}{356.4} = 2.87$

Therefore, if the design load is $11t/m^2$, the load shared by the surrounding soil is given by (q_s) load shared by the soil $\frac{11}{3.87} = 2.84t/m^2$

(q_p) load shared by the pile $= \frac{11}{3.87} \times 2.87 = 8.16t/m^2$

(n) Check for Bearing Capacity

Since the load shared by the soil ($2.84t/m^2$) is less than the ultimate bearing capacity of soil ($8t/m^2$) hence safe.

(o) Additional Pile Capacity due to Load Shared by Soil

Increase in Effective Mean Normal Stress, σ'_m

$$\sigma'_m = 1/3\ (1+2K_o)\ q_s \qquad (3.56)$$

Substituting suitable values

$\sigma'_m = 1/3\ (1+2x0.3954)2.84$

$= 1.695 t/m^2$

(p) Corrected Soil Modulus, E'_s

$$E'_s = E_s[\frac{\sigma'_m}{10}]^{0.5} \qquad (3.55)$$

Substituting suitable values

$$E'_s = 800[\frac{1.695}{10}]^{0.5}$$

$$= 329.36 t/m^2$$

(q) Rigidity Index (I_r)

$$I_r = \frac{E'_s}{2(1+\mu)\ (\sigma'_m \tan\emptyset')} \qquad (3.57)$$

$$I_r = \frac{329.36}{2(1+0.2)(1.695x0.6)} = 135$$

Hence Cavity Expansion Factor, F'_q

Referring from Fig. 3.4, for $\emptyset=31$

$c = 0,\ I_r = 135,\ F'_q = 8.5$

(r) Additional Pile Capacity q_{ult}

Referring eqn (3.60)

$$(q_{ult})' = 6\sigma'_m\ F'_q \qquad (3.58\ \&\ 3.59)$$

Where

$(\sigma'_m F'_q) = \sigma_{rL}$ which is the limit lateral stress of the soil

$(q_{ult})' = 6x1.695 \quad 8.5$

$= 86.44\ t/m^2$

(s) Total ultimate Pile Capacity (q_{ult})

$$(q_{ult}) = (q_{ult})_1 + (q_{ult})' \qquad (3.60)$$

$$= 95.27+86.44 = 181.71 t/m^2$$

(t) Ultimate Load of Single Pile Q_p

$$Q_p = A_p\ (q_{ult}) \qquad (3.61)$$

$$= 0.096 \times 181.71$$

$$= 17.44 \text{ tonnes.}$$

Computation of Ultimate Load Carrying Capacity of a single plain Granular Pile in Loose to Medium Dense non Cohesive Soil Deposit

(a) Input Data

Effective angle of shearing resistance $\emptyset'$ = 30^0

Effective cohesion, c' = 0

Bulk density, γ_{bulk} = $1.7t/m^3$

Pile length, L upto concrete base = 3.5m

Pile dia., d = 25cm

(b) (i) Coefficient of Lateral Earth Pressure K_0 = 0.4122

(ii) Poisson's Ratio μ = 0.2914

(iii) Soil Modulus E_s = $800t/m^2$

The above values are computed in similar manner as in (APPENDIX "A")

(c) Effective Normal Stress, σ_v

Assuming height of bulge from the base of the pile as 5 pile dia. which is equal to 1.25m from the base of the pile. Hence the depth of top of bulge from the pile top is (2.25m)

Thus $\sigma_v = 1.7x2.25 = 3.825t/m^2$

(d) Effective Mean Normal Stress σ_m

$\sigma_m = 1/3\ (1+2K_0)$ (3.31)

$= 1/3\ (1+2x0.4122)\ 3.825$

$= 2.3261t/m^2$

(e) Corrected Soil Modulus E'_s

$$E'_s = E_s \quad 0.5 \quad 3.50 \qquad (3.96)$$

$$= 800 \left[\frac{2.3261}{10} \right]^{0.5}$$

$$= 385.83 t/m^2$$

(f) Rigidity Index I_r

$$I_r = \frac{E'_s}{2(1+\mu)(\sigma_m \tan\emptyset)} \quad 3.51 \qquad (3.97)$$

$$= \frac{385.83}{2(1.2944)(2.3261 x 0.7265)}$$

(g) Cavity Expansion Factor, F'_q

Referring Fig. 3.4, $F'_q = 7$ for $\emptyset = 30^o$, $I_r = 85$

(h) Limit Lateral Stress σ_{rL}

$$\sigma_{rL} = \sigma_m F'_q \quad \text{for cohesionless soil} \qquad (3.95)$$

$$= 2.3261 x 7$$

$$= 16.2827 t/m^2$$

(i) Ultimate Uplift Load Q_o

$$Q_o = (K_p \sigma_m F'_q) A_p \qquad (3.101)$$

$$6 x 16.2827$$

$$= 8.8962 x 0.049$$

$$= 4.786 \text{tonnes}$$

(j) Weight of the Pile W_g

$$W_g = 0.049 x 3.5 x 2.2 = 0.377 \text{ tonnes}$$

(k) Ultimate Uplift Pile Capacity, Q_u

$$Q_u = Q_u = W_g + Q_o \qquad (3.102)$$

$$= 0.377 + 4.786 = 5.163 \text{ tonnes}$$

Computation of Lateral Load Capacity of a Skirted Granular Pile Foundation with a Central Unit Pile

(a) Input Data

Dia. of the granular piles	= 25cm
Depth of the granular piles	= 3.5m
Outer dimensions of the skirt	= 130x130cm
Inner dimensions of the skirt	= 100x100cm
Dimension of the Unit Pile Consisting of two nos. 15cmx45cm cross units at right angles	= 15cmx45cm
Depth of the Unit pile	= 3.0m
Unit modulus of subgrade reaction (n_h) based on Mc Corcle (1969)	= 0.245 kg/cm^3
Unit modulus of subgrade reaction (n_h)	= 0.25 kg/cm^2 (used)
Lateral displacement (assumed)	= 12mm
Elastic modulus of concrete	= 2x10^5 kg/cm^2

(b) Moment of Inertia (I)

(i) M.I. for the skirt $= \frac{BD^3}{12} - \frac{bd^3}{12}$ ..(I)

Out side dimensions are B = 130cm, D = 130cm

and inside b = 100cm, d = 100cm

$$I_{(skirt)} = \frac{130^4}{12} - \frac{100^4}{12}$$

$$= 1.546\text{x}10^7 \text{ cm}^4$$

(ii) M.I. for the Unit Pile

$$I_{(unit\ pile)} = \frac{45(45)^3}{12} - 4\ \frac{15(15)^3}{12}$$

$$= 3.248 \text{ x } 10^5 \text{ cm}^4$$

(c) EI . values of Skirt and Unit pile

(i) $EI_{(Skirt)} = 2.0\text{x}10^5\ (1.546\text{x}10^7)$

$= 3.092\text{x}10^{11}$ kg.cm

(ii) $EI_{(unit\ pile)} = 2.0 \times 10^5\ (3.275 \times 10^5)$

$= 6.48 \times 10^{10}$ kg.cm

(d) The relative stiffness factor (T)

The relative stiffness factor (T) is given by the following equation according to Reese & Matlock (1956)

$$T = \left[\frac{EI}{n_h}\right]^{1/5} \qquad \ldots(II)$$

(i) The value of (T) for the skirt is given by the equation II.

$$(T)_{Skirt} = \left[\frac{30.92 \times 10^{10}}{0.25}\right]^{1/5} = 262\text{cm}$$

(ii) The value of (T) for the unit pile is given by

$$(T)_{unit\ pile} = \left[\frac{6.48 \times 10^{10}}{0.25}\right]^{1/5} = 191.8\text{cm}$$

(e) The Lateral load Q_{hg} for 12mm deflection

If the assumptions of elastic behaviour is introduced for the pile and lateral deflection "y" remains small then according to Reese & Matlock (1956)

$$y = y_A + y_B \qquad \ldots(III)$$

where

y_A is deflection due to Q_{hg} and

y_B is deflection due to moment (M_g)

In the present case since the Q_{hg} is taken at foundation level, hence y_B becomes zero. Thus

$$y = y_A \qquad \ldots(IV)$$

$$\text{and } y = \left(\frac{Q_{hg}\ T^3}{EI}\right).\ A_y. \qquad \ldots(V)$$

where

A_y is the deflection coefficient which is taken as 2.435 for Q_{hg} acting at foundation level, (Reese & Matlock 1956).

(i) Therefore the lateral load (Q_{hg}) for the skirt is given by

$$(Q_{hg})_{skirt} = \frac{y.EI}{A_{y(T)}^3} = \frac{1.2x(30.92x10^{10})}{2.435x(262)^3}$$

$$= 8.76 \text{ tonnes}$$

(ii) and Q_{hg} for the unit pile is given by

$$(Q_{hg})_{unit\ pile} = \frac{1.2(6.49x10^{10})}{2.435x(191.8)^3}$$

$$= 4.7 \text{ tonnes}$$

Hence the total Lateral load (Q_{hg}) is found by adding (i) and (ii)

Thus $(Q_{hg})_{Total} = 8.472+4.7$

$$(Q_{hg})_{Total} = 13.1 \text{ tonnes}$$

VITAE

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