BEARING CAPACITY OF UNDER-REAM PILES IN WEAK SOILS

A DISSERTATION
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By

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CERTIFICATE

Certified that the dissertation 'Bearing Capacity of Under-Reamed Piles in Weak Soils' being submitted by Shri K.G.S. Jain, in partial fulfilment for the award of the Degree of Master of Engineering in Soil Mechanics and Foundation Engineering of the University of Roorkee, Roorkee is a record of the student's own work carried out by him under my guidance. The matter embodied in this dissertation has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a period of 6 months from May 1970 till November 1970 for preparing this dissertation.

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ABSTRACT

The present study was carried out to study the bearing capacity of single and multiple under-reamed piles in the weak soils covering a large portion of the Shrinagar Valley. The study included full scale tests on 8 nos. $4\frac{1}{2}$ meter deep piles alongwith other field test and an attempt has been made to be able to compute the bearing capacity of these piles by simple field tests like the Dynamic cone penetration tests. The study has revealed that the bearing capacities computed from shear characteristics of undisturbed samples, comparable values as found from actual load tests are obtained. But, the existing relations, based on SPT values, estimate very high values. For this, suitable factors, that could be applied over the estimated values, have been worked out.

While computing the bearing capacity of under-reamed piles, it has been generally observed that the bearing part is much over-estimated. This could be due to the release of stresses in the zone around the bulbs due to cutting away of the soil. This may be particularly true when under-reaming is done in silty soils under water.

Particular reference has been made to the present practice where a tighter settlement criteria in most codes for routine interpretation of load test on piles is prescribed. On the basis of the present tests, it is found that the safe load tables, given in the Code (IS:2911-Part 1, 1964) which is under revision, should be based on 25 mm. and not 12 mm. settlement criteria.

The study has also revealed that by reducing the spacing of under-reamed piles from 2 times to $1\frac{1}{2}$ times the under-reamed diameter, there is a reduction of about 7 percent in the bearing capacity. However, the contribution in bearing by the pile cap was found to be fairly large, almost equal to its individual bearing.

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NOTATIONS

α	Rates of shearing strength of soil/frictional resistance
β	Adhesion coefficient
γ	Average unit weight of soil
a	Reduction factor on N_{cp}
A_p	Sectional area of pile toe
A_s	Lateral surface
B	Width of base
C	Cohesion
$\bar{C}$	Average cohesion
C_p	Average shearing strength
CRP	Constant rate of penetration method
D, H	Depth of pile
F_1	Factor of safety for the frictional resistance
F_2	Factor of safety for the point resistance
f_s	Skin friction
K	Earth pressure coefficient
ML	Maintained load test
N	Number of blows per 30 cm penetration in a Standard penetration test
N_1	Average of SPT values over pile length
N_2	SPT at bearing level
N_c	Number of blows per 30 cm penetration of cone
N_b, N_{cp}, N_q, N_{qp}	Pile bearing capacity factors
N_Φ	$\tan^2 (45 + \Phi/2)$

Φ	Angle of internal friction of soil
q_c(av.)	Average penetration resistance over the length of the pile
q_c	Average penetration resistance around the pile toe or individual bulbs
q_u	Unconfined compressive strength
Q_s	Safe load
Q_u	Ultimate bearing capacity
SPT	Standard penetration test
τ_f	Undrained strength of soil
σ_f	Bearing capacity
Z	Depth below ground level

CHAPTER - I

1. INTRODUCTION

Under-reamed piles were first used in early 40's in Texas where expansive soils of poor engineering properties were found in abundance. It was Jennings et.al (1949) who used under-reamed pile on a fairly large scale in South Africa. Collins (1953) was the first to suggest a reasonable theory behind the use of under-reamed pile. The buildings founded on normal foundations cracked due to differential ground movements caused by alternate swelling and shrinkage of the soil due to changes in its moisture content. For safeguarding foundations against these movements effectively, the best remedy was to anchor the structure at the depth where volumetric change of the soil due to change in seasonal and other variations was negligible. Under-reamed piles provided the remedy.

In India, under-reamed piles were first introduced in 1955 (Dinesh Mohan and Jain) for expansive soil areas. Use of these piles was subsequently suggested for filled up grounds and other poor soils overlying firm strata (IS:2911 Pt.1,1964). However, no systematic study has so far been carried out for assessing the bearing capacity of under-reamed piles in weak soils.

Foundations generally adopted in such soils are raft or piles. In raft foundations, settlements can be very large. The problem becomes all the more acute when water table is close to ground level and it becomes difficult to provide a basement.

Pile foundations generally suit well as there is wide choice in their type and very often better strata is available at some depth. For conventional foundations deep excavation may be required but by adopting piles, no such excavation is needed. Also sinking of floors near the walls can be avoided as no back filling is involved in this case.

In loose soil deposits, driven piles are generally to be preferred as they compact the soil fairly well and thus increase the bearing. These can be either precast concrete piles or driven cast-in-situ concrete piles. Timber piles can be used in places where good timber is locally available and water table remains close to ground level.

The problem is more acute in soft clay and silt deposits. For the present study,therefore, Srinagar valley was selected. The strata in the valley is weak for considerable depth and water table is close to ground level. Full scale tests were planned and carried out on piles of both uniform diameter and with one, two and three bulbs. Soil exploration included field tests (Standard penetration tests, Dynamic cone penetration tests, Static cone penetration tests and Plate load tests), and laboratory investigations on undisturbed samples collected from boreholes with piston samplers.

CHAPTER - 2

2. REVIEW OF LITERATURE

Theory, in so far as it is tied to the facts from which it evolved, belongs to the past: facts under observation constitute the present, which is never any better than our means of observing it. Finally, the abstractions of analysis and synthesis, and retheorising, are our hopeful projection into the future; and so the cycle proceeds.

2.1 Present Theory of Under-reamed Piles in Expansive Soils

2.1.1 Uplift forces

Collins (1953) forwarded a preliminary theory for the design of under-reamed piles. It was based on a study carried out on piles embedded to a depth of 30 ft. (9.25 m) which had failed in tension. According to this theory, full friction on the pile stem is considered acting upwards during heaving, and this resulted in providing very heavy reinforcement in the piles. Subsequent work carried out by the Central Building Research Institute indicated that in black cotton soils of India sub-continent, only nominal reinforcement (3/4 percent of pile x-section) was generally sufficient. This was particularly because of short length of piles ($3\frac{1}{2}$ meter) needed here and that full friction never mobilises during heaving (C.B.R.I. Building Digest 37 now revised to 56). The work of Collins has also been questioned by Donaldson (1963), pointing out that overseas research workers had found that in practice there was marked reduction in the frictional forces developed on piles. Baiseff and Burke (1965) also state that provision of tensile reinforcement according to Collins makes the use of under-reamed piles not only expensive but impracticable especially where deep piles are required. Donaldson (1967) has carried out some further experiments for predicting the uplift forces on anchor piles. He found that if enough moisture was available, then full adhesion was utilised in the development of uplift forces.

2.1.2 Provision of Bulbs

Model load tests by Neumann and Peleg (1955) have indicated that the load carrying capacity increased if two bulbs were provided. Mintskoviskii (1964) also states that all the bulbs are effective. Dinesh Mohan, Jain and Sharma (1967) have reported on the basis of both laboratory and field investigations that by providing one bulb, the bearing is increased upto 100 percent and by providing two bulbs, there is a further increase of 50%. Poulos has shown that settlement of piles is reduced considerably with increase in number of bulbs

Number of bulbs	Percentage decrease in settlement
1	24
2	38
3	42
4	49
5	56

2.2 Bearing Capacity of Foundations

The first step in the analysis of the adequacy of a foundation is to ensure that the maximum foundation loading assumed will not cause a shear failure of the under-lying soil thereby causing punching of the foundation into the soil. This is characterised by very large and unpredictable settlements.

As loads are applied to a foundation, settlements occur. A typical load test result of footing settlement is observed. The observed settlement corresponds to a load or pressure and the settlement is generally continuous with a slope increasingly steepening until the ultimate is reached.(Terzaghi and Peck 1967). As an instance, on a saturated clay, a quick, undrained, loading condition most likely to cause failure is distinct from the slow long observation loading. Typical pressure settlement curves of load tests on dense or stiff and, loose or soft soils are shown in Fig. 1.

From practical considerations, though failure conditions are not so important but, every foundation must be investigated with regard to a minimum factor of safety against ultimate bearing capacity failure and such safe loading that any deformations and settlements are within permissible limits (Skempton 1951).

2.3 Pile Bearing Capacity

The bearing capacity of a single pile is taken as the sum of point resistance and skin friction, both of which depend on the properties of the soil and the method of installing the pile.

As regards the point resistance, the basic bearing capacity equation can be suitably adopted into the form:

$$\sigma_f = c\,N_{cp} + \gamma\,D\,N_{qp}$$

The pile bearing capacity factors N_{cp} and N_{qp} are semi-empirically established to account for the limiting values of depth factors; the influence of the shape of the base and the compressibility of the soil. (Meyerhof 1951). The values of N_{cp} and N_{qp} are shown in Fig. 2. These are said to apply only if the pile base is embedded in the load bearing stratum near the base atleast to a depth which is about

$$D = 4\sqrt{N_{\Phi}}\,B$$

where $N_{\Phi} = \tan^3(45 + \Phi/2)$

If the soil properties vary near the base, use of average values is suggested in accordance with the probable failure zone, of the order of one pile diameter above to four below the base.

It will be noticed that the value of $N_{cp} = 9$ for deep plates is generally accepted. Experiments on large bored piles in London clay have shown that a lower value than 9 is called for. However, Professor Skempton (1966) has concluded that the value of N_{cp} may remain 9 and a factor 'a' be applied where

$a = 0.8$ for $B \leqslant 3$ ft.

and $a = 0.75$ for $B \geqslant 3$ ft.

For sensitive clays, however, Ladanyi (1967) concludes that the N_{cp} value of 9, valid for non-sensitive clays, should decrease to about 6 to 7 for clays of moderate sensitivity, and to about 4 for highly sensitive clays.

As regards deep foundations in sands, study carried out by Vesic (1965) has forced revision of earlier theoretical concepts Fig. 3(c). This is particularly due to the insignificant dimension of the point in comparison with the depth. The value of N_{cp} are fairly well established but there remains wide scatter in values of N_{qp}. It emphasizes need for prototype observation of facts using improved technique, considering influence of compressibility and deformation particularly in case of C, Φ soils.

2.3.1 Point Bearing and Skin Friction for Piles

In piles there is joint contribution of skin friction and point bearing, and the former often constitutes the principal contribution.

(a) Point resistance

It has already been pointed out that the factor $N_{cp} = 9$ is not general. Meyerhof (1964) in his discussion has summarised that his own rigid plastic analysis had indicated $N_{cp} = 9.3$; simplified recommendations could be made and the N_{cp} values should be corrected by a reduction factor varying from about 0.5 to 1.0.

Sowers (1961) on basis of model tests has reported N_{cp} = ~~58~~ 5 to 8. Not many field measurements are available to check the deep N_{cp} for soft clays. Kerisel (1965) reported values even higher than 9. Dinesh Mohan et.al (1961) have reported values for expansive clays wherein comparing with vane shear strengths of

14 to 18 t/m^3, the resulting N_{cp} values are 8.2, 5.7; 7.8 and 7.2:

An important fact that is revealed from the more recent works (Vesic 1965) is that the point resistance increased quasi-proportionately with depth only at relatively shallow depths, while at greater depths, generally exceeding about 10 to 20 foundation diameters, this resistance reached asymptotically almost constant final values. Thus, the main problem in determining the point resistance shifted to one of establishing the stress field due to overburden stresses that can be considered effective. Vesic achieves the desired comparatibility between observed and theoretical bearing values by postulating that the stress field around a deep foundation resembles that observed in a silo or, generally in a mass of soil above a yielding horizontal support i.e. arching as shown in Fig. 4 (Vesic 1965). The critical depth beyond which no further increase in bearing capacity is obtained in a homogeneous soil, is shorter in a loose material than in a dense one.

Thus, it appears important to recognize that "because the load transfer is expected near the pile, the vertical pressure must not be equal to the overburden pressure as 'γz'" (Nishida 1963). Thus a fundamental fallacy lies in the fact while computing the bearing capacity of piers and piles that overburden stress is considered varying linearly with depth.

(b) Skin friction

In homogeneous soils, the skin friction (f_s) is often the principal component and is taken equal to α times the undrained strength of soil T_f. Values of α have been determined experimentally. The skin friction in sands also follows the same trend as in point resistance i.e. reaching asymptotically almost constant final values with increase of pile length after an early quasi-proportional increase with depth upto about 10-20 diameters. Moreover, displacements needed to reach the ultimate friction are independent of the diameter, whereas those required to develop the ultimate base resistance are roughly proportional to size. Also, during the progress of a typical load test not only do the proportions of overall skin resistance to point resistance gradually decrease because of the factor mentioned above, but also the distribution of skin friction along the pile length changes appreciably.

Considering saturated clays in which the adhesion coefficient β has been traditionally defined by the overall ratio of the ultimate lateral resistance (under rapid loading) to the product of the lateral surface 'A_s' with the undrained strength 'T_f'. For purposes of design T_f may be taken as in-situ undrained strength which would be available to the designer. Value of β will also be affected by (1) the type of contact surface, (2) the construction effects and (3) variations of strains along the pile.

Considering the nature of construction material and consequent skin friction, a table of values is furnished by Potyondy (1961). Values range from about 0.5 for smooth steel on clay, to about 1.0 for rough concrete. The study also reveals that the general (C, Φ) soils may yield significantly different results. Meyerhof (1963) based on model tests has reported values 0.6 to 0.8 for steel, 0.7 to 0.9 for wood, and 0.8 to 1.0 for concrete. However, it should not be expected that peak adhesion value may develop simultaneously over the entire length of pile (Cooke and Whitaker 1961).

Meyerhof (1951) suggested $\beta = 0.5$ in sensitive clays, shortly after driving of concrete, timber or steel piles, and $\beta = 1$ for insensitive clays. The 'T_f' is calculated for earth pressure coefficient $K = 1$ with an upper limit of 1 kg/cm^2. Mohan and Chandra (1961) reported for cast-in-situ bored piles in expansive clays that the average $\beta = 0.5$. Kerisel (1965) recommended curve for the long term β value is given in Fig. 5.

We find that there are four factors which have a major effect on the accuracy of the computed results. These are (1) the material properties of the soil, (2) the elastic-plastic tip movement, (3) the tip punching force, and (4) the normal stress on the pile soil interface. The major problem lies in the evaluation of the fourth factor. Nishida (1961) has furnished derivations of stresses around a compaction pile. Nair (1967) treats an essentially similar problem of stress transmission and consequent elastic settlement caused by a friction pile. For the case of piles, in saturated clays, the stresses transmitted to the soil by the static loading in question will create pore pressures that result in additional time dependant consolidation settlement. Poulos and Davis (1968) investigated the settlement behaviour of a single axially loaded incompressible pile in an ideal elastic soil mass. They, however, concluded that the major portion of the total final settlement of a single pile in an ideal soil occurs as immediate settlement and that only a small portion occurs as time dependant consolidation settlement.

2.3.2 Load Tests on Piles

Selection of a method and interpretation of load test data is important when there is need of predicting settlements on the basis of load tests particularly in clays where pore pressure development is likely. However, none of the test and interpretation procedure published have specifically distinguished between different types of soils or piles. For saturated clays which are subject to compression in shear, to obtain a conservative estimate of the failure load a fairly rapid, undrained loading is preferred.

In the traditional method (Maintained load test) (ML) each load increment is maintained constant until the rate of settlement is so small that the pile soil interaction almost reaches equilibrium. Such a test often runs for a few days.

Building Research Station in U.K. introduced the so called constant rate of penetration (C.R.P.) method. Results are furnished (Whitaker and Cooke 1961) to prove that the ultimate loads defined by successive loading-unloading cycles of C.R.P. tests establish an envelope that truely defines the load-penetration diagram of the pile into the soil. Dinesh Mohan and Jain (1967) confirm the satisfactory correlation between ML and C.R.P. ultimate loads in expansive clays. However, it is better to run a combined ML-CRP test for a clear indication of settlements; upto design load ML is preferable and for a rapid failure CRP test is followed.

For developing a practical method of running an adequate test quickly, Dinesh Mohan and Jain (1967) proposed the so called 'Equilibrium method' in which both the settlements and ultimate of the ML test are closely reproduced. It consists in applying a given load increment, maintaining it for about five minutes, and then allowing it to reduce itself due to the yielding of the ground. The next higher load is then applied and the process is repeated. For higher loads it is required to maintain the initial load for a period of 10-15 minutes before it is allowed to diminish. The load at which equilibrium is attained is always lower than the maximum and it provides a better average than that obtained in a maintained load test.

It is, however, generally agreed that most of the settlement of individual piles is very rapid. In any long duration prediction, the major effects derive from execution effects and many other unknown pile-soil interactions and it becomes extremely difficult to explain the phenomena. from consolidation theory or from rheological behaviour of soil after driving. Sometimes loading and unloading after each cycle helps in better interpretation of ultimate and allowable loads (Szechy 1961) and as indirect means of distinguishing between friction and point bearing contributions (Van Weele 1957, Jain and Kumar 1963, Dinesh Mohan and Jain 1967).

2.3.3 Other In-situ Tests

For the design of deep foundations, in-situ testing has always been preferred. The methods which have gained popularity are Deep sounding or Static cone penetration tests, Standard penetration test and Dynamic cone penetration tests.

(a) Static cone penetration tests

A portable sounding apparatus with which reliable information could be acquired within a short time, about the consistency of the different layers of the sub-soil has been described as early as 1936 by Barentsen. The idea can be said to have gained phenomena when Skempton and Bishop (1950) used cone tests for the measurement of undrained strength of clays in the laboratory. It was later on that there has been considerable development of the equipment and it was possible to measure cone resistance and skin friction separately by introducing a sleeve around the rods pushing the cone-Fig. 6. Begemann (1953) used a friction jacket following the cone for the measurement of unit skin friction alongwith cone resistance.

Static cone penetration tests have since been used widely for estimating bearing capacity of piles. The ultimate pile point resistance is often same as the penetrometer point resistance (Kerisel 1961, de Beer 1963, Vesic 1965). However, same depth/diameter ratio within the stratum should be maintained. A certain minimum thickness (10 to 20 diameters) of uniform soil is required in order to obtain a penetration resistance which is truely representative of a particular stratum. Thus, the penetrometer point resistance is much more zig-zag than that would be obtained for a pile point. Vander Veen (1957) suggests that the average may be obtained over a distance from one diameter below the actual point to 3.75 diameters above it. However, this would not be applicable to bored piles particularly in weak soils where there may be more chances of pile movement due to settlement of the strata below the pile tip as indicated by Vesic (1965) in Fig. 4(c). In such situations it is more likely that the strata which is affected most lies one diameter above and 2 diameters below the point. Based on a large number of field tests, Dinesh Mohan, Jain and Kumar (1963) recommended the use of static penetration tests in evaluating the bearing capacity of piles and suggested that the safe load on a pile may be worked out from the relation

$$Q_s = \frac{q_c(av.)A_s}{50} \times \frac{1}{F_1} + q_c\,A_p \times \frac{1}{F_2}$$

Where $q_c(av.)$ = average penetration resistance over the length of the pile

q_c = average penetration resistance around the pile toe

A_s = surface area of the pile shaft

A_p = sectional area of the pile toe.

F_1 = factor of safety for the frictional resistance

F_2 = factor of safety for the point resistance.

The values of factors of safety F_1 and F_2 are suggested to be 2 and 2.5 respectively.

(b) Standard penetration tests

Use of Standard penetration tests (SPT) in evaluating the allowable bearing pressure of footings occupies an important place in foundation engineering (Terzaghi and Peck 1967). Terzaghi and Peck (1948, 1967) have given an approximate means of establishing the unconfined compressive strength of clays from the SPT values-Table 1.

TABLE 1

Consistency	Very soft	Soft	Medium	Stiff	Very stiff	Hard
SPT	2	1-4	4-8	8-15	15-30	30
qu (t/m^2)	less than 2.5	2.5-5	5-10	10-20	20-40	over 40

This corresponds to a practically linear relation of unconfined compressive strength (kg/cm^2) = $\frac{SPT}{8}$. Peck and Reed (1954) have suggested on the basis of an extensive study of Chicago clays a conservative boundary qu = SPT/6 although an average curve approximates SPT/4. Mellow et.al (1959) obtained for an unsaturated silty clay of Sao Paulo a statistical correlation $q_u = 0.061$ SPT + 1.3 (kg/cm^2). Golder (1961) refers to a good correlation with a clay of liquid limit = 33%, the corresponding data may be represented by the relation, $q_u = \frac{SPT - 2}{7}$ $\pm$ 0.4 kg/cm^2. Ireland (1953) refers to correlations in clay ranging between SPT = 6 q_u and SPT = 5 q_u. Recently (Mello 1967) has shown that with increase in sensitivity of clay from 1 to 10, the SPT values corresponding to a given undisturbed q_u value would drop to as little as 15 percent.

(c) Dynamic cone penetration test

This test is carried out similar to a standard penetration test by driving a 60° cone having a base diameter of 6.5 cms. The size of cone has been standardised by conducting a large number of tests in the field with varying sizes of cones, and a cone of 2.5 in. diameter has been recommended (Dinesh Mohan, Agarwala and Tolia, 1970). The number of blows per 30 cm of penetration of the cone (N_c) provides the penetration resistance. When the cone is driven to some depth, there can be friction on the driving rods effecting the number of blows. This friction can be eliminated by circulating bentonite slurry. Driving of the cone and pushing in of the slurry is simultaneously carried out. In cohesive soils, however, friction does not come into play for considerable depth as the diameter of driving rods is only $1\frac{5}{8}$ inch against $2\frac{1}{2}$ inches of the cone.

The test provides a continuous record of soil resistance which generally tallies with N values in most soils as shown in Fig.7 and Fig. 8 (Dinesh Mohan, Sengupta and Jain, 1967). Figure 7 shows the relation between Dynamic cone resistance and N-value in sandy soils and Fig. 8 in silty clays. It will be seen that there is an excellent straight line relation in sandy soil whereas in silty clays there is some scatter. Details of the method are given in Appendix - A.

2.4 Design of Pile Foundations

The design of a pile foundation, and the estimation of its probable behaviour in supporting building loads involves quite a few problems. There are many rule of thumb efficiency formulae. These formulae do not take into account the nature of soil strata, types of piles used etc. The most important factor that concerns piles in silt or soft clay is the examination of ultimate bearing capacity of the entire group (Terzaghi and Peck 1967) considering the piles and the confined mass of soil capable of sinking as overall 'caisson' or pier. Efficiency factor is higher for (a) smaller piles, (b) greater spacings, and (c) smaller number of piles in the group. Settlement ratio defined as the ratio between settlement of pile group and settlement of single pile at failure is found to increase with length and number of piles in the group. The settlement ratio value is generally found to vary between 3 to 5.

2.4.1 Stress Change Around a Pile

Whereas considerable data is available regarding stress changes around a driven pile, not enough work appears to have been done on bored piles specially in weak soils under water. Chadeison (1961) compared for bored piles the use of drilling mud as against a driven casing, and concluded that the bearing capacity of the pile in the first case was somewhat better than in the second. However, in weak soils under water, this would considerably depend on the nature of strata, cased or uncased boring and the fluid filling the bore during boring operations. Since the introduction of multi-under-reamed piles (Dinesh Mohan, Jain and Sharma, 1967), for using such piles in soft deposits, a detailed study is needed as there is likelihood of considerable stress relief around the bulbs and normal assumptions (Jain, 1966) may not apply in all cases.

Some of the advantages of multi-under-reaming suggested are -

(a) Saving in operationa cost due to reduced lengths.

(b) Use of lighter equipment and less concrete.

(c) Groups of piles could generally be replaced by single piles.

(d) It ensured a more uniform settlement of the structure since the influence zone increased considerably because the dispersion of the load began from a higher level.

(e) Due to a substantial increase in the friction component, the actual settlement under a structural load was less than that of piles with only one large bulb or group of piles and this increase in friction component also decreased the elastic compression of the pile stem.

In Fig. 9, is shown an idealised case of an incompressible pile in elastic-plastic soil. Curves A and A' show the generalised behaviour of single and multiple under-reamed piles respectively, and B, B' and C, C' represent the contribution of skin friction and end bearing. Due to substantial increase in the skin friction component in the multiple under-reamed pile, the settlement at the assigned working load is less than that of a single under-reamed pile. The criteria of increased friction is explained by Fig. 10 where friction in a double and triple under-reamed piles is shown along a cylinder circumcribing the bulbs. For working out the frictional resistance, full shearing strength of the soil is accounted for and not half of of it as in case of a pile of uniform diameter.

2.4.2 Allowable Settlements

Very often tighter settlement criteria in most Codes for routine interpretation of load test on piles is prescribed. Sometimes, for the same structure a settlement many times more would be permitted if a different foundation was adopted. However, the recent Building Code for India (1970) prescribes a reasonable criteria for assessing safe load on piles by saying that; "if a settlement other than 12 mm is permissible in a given case, depending on the nature and type of the structure, the actual total settlement permissible shall be used for assessing the safe load instead of 12 mm". In this regard the present practice (Bjerrum 1963) to ascertain allowable settlement is shown in Fig. 11. This may, however, be taken as a guide only as there are a number of important factors to be considered jointly before any reasonable assessment of allowable settlement can be made.

CHAPTER - 3

DESCRIPTION OF TESTING SITE AND FIELD INVESTIGATIONS

3.1 Geological Formation of Kashmir and Srinagar Valley

The physical geography of the Kashmir area (Wadia 1957) indicates violent changes during its formation and what was once a region of quiet marine sedimentation was converted into a great theatre of volcanicity which slowly subsided. From Jhelum to Ravi, it now constitutes a remarkably consistant and characteristic belt of altered limestone overlain by a thick shale with coal seams at the base. Its width varies from a few yards to about 4 miles. A diagrammatic section across the Kashmir Himalaya is shown in Fig. 12.

There are not many large lakes in Kashmir region except the Wular lake at river Jhelum. The Srinagar Valley has many small lakes besides the river Jhelum which flows in the midst of the city. The valley is fairly flat and the ground generally consists of soft silty soil to a depth of 4 to 10 meters which progressively increases in strength beyond about 6 meters.

3.2 Location of the Test Site

The site is situated in Badami Bagh area in Srinagar. It is a fairly flat ground. A plot of land about 30 m x 30 m was selected for the study. The area is known to have weak soils to considerable depth and has been a problem to engineers since long.

3.3 Selection of the Pile Test Site

For the selection of site, a series of dynamic cone penetration tests were carried out to a depth of 9 meters in the area. The exact location of the site was made when the penetration resistance was found to be fairly uniform over the required area to a depth of about 6 meters. The location of the three dynamic cone penetration tests which indicate fairly uniform strata are shown in Fig. 13 and the penetration resistance of the ground is shown in Fig. 14.

These tests were followed by a 30 cm diameter bore hole made with a spiral earth auger. When the bore hole was about a meter deep ground water started seeping into the bore hole. When augering reached 2 meter level, it was decided to pump out water and see if boring could be continued in dry. Immediately after pumping, large amount of water started seeping into the bore hole. The sides of the bore hole also started caving in as the water was pumped out. The bore was therefore allowed to get filled up with water. The boring was again continued to $3\frac{1}{2}$ m depth where under-reaming was done. The amount of soil taken out and widening of blades indicated that a correct shape of bulb was formed. The use of bentonite slurry was therefore considered unnecessary. The site was finally selected for the pile tests.

3.4 Field Investigations

The preliminary investigations were followed by an exhaustive field study. This included undisturbed soil sampling and laboratory investigation, Standard penetration tests through bore holes, Static cone penetration tests, Dynamic cone penetration tests and Plate load tests.

3.4.1 Undisturbed Soil Sampling and Laboratory Investigations

Four nos. 25 cms diameter bore holes were made upto a depth a little over $5\frac{1}{2}$ meter. The location of the bore holes is shown in Fig. 13. Samples of both $7\frac{1}{2}$ cm (3") and 3.75 cm ($1\frac{1}{2}$") diameter were obtained by using piston sampler in the former case and thin tubes for the latter. All the samples were extracted out of the tubes at the site. They were properly sealed by waxing and sent to the laboratory in boxes fitted with resilient material on the inside. The samples when received at the laboratory in Roorkee were in excellent condition. Routine laboratory tests were conducted on soil samples and the engineering properties were found. These are given in Table 2. It will be observed that the strata consists of mostly medium plastic clay (CI group) with fairly uniform moisture content and unit weight. The plasticity index (PI) varies from 7 to 16, the lower value being for the fine clayey silt (ML group). The values of C and Φ vary considerably though all the samples almost passed 200 sieve and this variation could be due to the actual clay content of the soil.

3.4.2 Standard Penetration Tests

SPT tests were carried out at two locations as shown in Fig. 13. The penetration resistance is shown in Fig. 15. Tests

TABLE 2. Soil Test Results

Depth in meters	Soil type	Unit weight gm/cc	Unit day weight gm/cc	Moisture content	Liquid limit	Plastic limit	Plasti-city index	C kg/cm²	Φ Oo	Unconfined strength kg/cm³	200 Passing %	C_c
0.95	CI	1.98	1.50	32.0	43.0	30.0	13.0	0.42	6.5	-	-	-
1.78	CL	1.87	1.44	30.0	33.0	24.0	9.0	0.320	5.5	-	99.0	0.16
2.4	ML	1.87	1.48	26.0	33.0	25.0	7.0	0.0	29.0	-	-	-
2.94	CI	1.92	1.50	28.0	38.0	24.0	14.0	-	-	0.49	97.0	0.17
3.56	CI	1.97	1.52	28.0	38.0	24.0	14.0	0.70	2.0	-	-	-
4.1	CI	1.92	1.48	29.0	40.0	28.0	12.0	0.46	7.0	0.28	99.5	0.24
5.66	CI	1.92	1.45	32.0	42.0	26.0	16.0	0.14	17.0	-	98.5	0.37

were carried out in bore hole made with 10 cm diameter spiral auger. No casing or stabilisation with bentonite slurry was required as the sides of the bore holes did not cave in. The maximum depth of the bore holes was 10 meters. Normally a bore hole of this depth below water table does not stand unsupported. However, in this case, by the repeated operations of lowering and taking out the auger a thick slurry was formed and this helped in retaining the sides of the bore hole.

The results of the tests show that the top strata upto about 5 m was a loose deposit giving the number of blows of 2 to 4 only.

3.4.3 Dynamic Cone Penetration Tests

The tests were carried out as per IS:4968 (Part II) - 1968, Method of Subsurface Sounding for soils, at five locations as shown in Fig. 13. The penetration data is shown in Figl 14 and 15. These tests present a more continuous record of strength variations in the strata.

From the penetration records it is evident that the top one meter at the site shows the effect of dessication thereby improving penetration resistance. Beyond this level below water table, poor strata is met upto about 3 meters depth. After this depth, slightly increasing trend in penetration resistance is observed. This is due to the increasing strength of the strata. It could also be partly due to the increasing friction with depth on the driving rods.

It may be noted that in upper strata, where friction on the driving rods is negligible, SPT and dynamic cone values are in good agreement.

Details for conducting Dynamic cone penetration tests are given in Appendix A.

3.4.4 Static Cone Penetration Tests

The tests were carried out at 2 locations as shown in Fig. 13. The penetration test data is shown in Fig. 16. The cone penetration resistance indicates a deposit of almost uniform strength to a depth of about 7 meters showing only a slight increase at 3 meters. Four static penetration tests were also conducted at the site about 100 meters away from the piles. The penetration resistance alongwith skin friction on the mantel is shown in Fig. 17. It will be seen that the cone resistance is almost uniform indicating low value upto about 10 meters showing only a slight increase beyond 7 meters. The skin friction values show a continuous increase which becomes more prominent beyond 7 meter depth. The test results corroborate the dynamic cone penetration test results to certain extent. However, the static penetration tests should be considered more reliable than SPT tests as regards the cone penetration resistance. This is because there is little chance of side friction coming into play because of the sleeve following the cone. There is no effect of bore hole size, method of boring etc.

3.4.5 Plate Load Test

Plate load test was carried out at one location as shown in Fig. 13. The load settlement curve is shown in Fig. 18(a). This curve is plotted on log-log scale in Fig. 18(b). The test was carried out at one meter depth (at water table), without any surcharge, on a plate of 30 cm x 30 cm with the help of a hydraulic jack. The results indicate a stratum of very poor bearing which is indicated by penetration tests as well. A local shear failure is indicated, ultimate bearing capacity being about 1.3 kg/cm^2 only.

3.5 Pile Load Tests

For evaluating the bearing capacity of under-reamed piles in weak soils and establishing its correlation with the results of field tests on the soils, seven piles were cast and tested with respect to load. The details of the test piles are shown in Table 3. An angle iron (75 x 75 x 6 mm) reinforcement was used in all the piles.

In addition to above, 2 piles were also made for obtaining the reaction for the triple under-reamed pile. The layout of the various piles is shown in Fig. 13. A typical set up for the test is shown in Fig. 19. For uniform dia., single under-reamed and double under-reamed piles, hold fast type anchors were used and a set up is shown in Fig. 20. Set-up for pull-out test is shown in Fig. 21.

A sketch of the under-reaming tool used is shown in Fig. 22 and constructional details of the piles are given in appendix B.

TABLE 3. Details of Test Piles

Serial No.	Pile type	No. of piles	Depth of pile m	Dia of piles (cm)	Under reamed dia (cms)	Number of bulbs	Spacing of bulbs in a pile (cms)	Spacing of piles (cms)	Remarks
1	Uniform dia	Two	4.5	30.0	-	-	-	-	One pile for compre-ssion test and one for pull out tests
2	Single under-reamed	One	4.5	30.0	75.0	One	-	-	-
3	Double under-reamed	Three	4.5	30.0	75.0	Two	113.0	-	One pile for compression test and two as anch-or piles
4	Triple under-reamed	One	4.5	30.0	75.0	Three	113.0	-	-
5	Group of two single under-reamed piles	One	4.5	30.0	75.0	One in each pile	-	113.0	The pile cap was 60 x 172.5 x 100 cm deep and was resting on the ground

3.5.1 Load Test Procedure

Reaction was obtained through R.S. joists bolted down to either anchor piles or hold fast type soil anchors. In some cases additional dead loading with bags filled with sand was also resorted to.

Load was applied by a hydraulic jack having a pressure gauge assembly and a remote control pump. For smaller loads, the jack with attached pump was used.

For loading, the method of equilibrium was adopted. In this method, about 1/10 of the estimated load is first applied by a hydraulic jack in a period of 3 to 5 minutes. It is maintained for about 5 minutes and then allowed to reduce itself due to the yielding of the ground. Within a few minutes, a state of equilibrium is achieved. The next higher load is then applied and the process is repeated. For higher loads, the initial load is maintained for a period of 10-15 minutes before it is allowed to diminish. The state of equilibrium reaches fairly quickly. At each stage of loading, a cycle of loading and unloading was adopted to find out the elastic rebound of the pile top.

Details of the load test procedure are given in Appendix C.

3.5.2 Load Test Data

The diameter of stem and the depth of various piles was kept the same. To evaluate the contribution of each bulb on the pile stem, a uniform diameter pile was also made alongwith piles with one, two and three bulbs Fig. 10. In addition to this, a group of two single under-reamed piles with the cap resting on the ground was also tested. Later on, on the same group a test was repeated after excavating below the pile caps so that the cap did not contribute towards the bearing.

Load settlement curves for net and gross settlements for uniform diameter, single, double and triple under-reamed piles are presented in Fig. 23 to Fig. 26 respectively. The pull-out resistance of uniform diameter pile is shown in Fig.27. The resistance provided by the group of two single under-reamed piles is shown in Fig. 28.

Tests were carried out upto a settlement of about one inch or more. In none of the tests, a clear break in the curve was obtained. Moreover, even at considerable settlements the curves show an increasing trend, therefore for deciding the ultimate, a settlement criteria is called for. As per IS:2911 (Part I), load bearing concrete piles, the safe bearing capacity of piles is taken as least of the following:

1) Two-third of the final load at which the total settlement attains a value of 12 mm., unless it is established that a total settlement different from 12 mm is permissible in a given case on the basis of nature and type of the structure. In the latter case the actual total settlement permissible shall be used for assessing the safe load instead of 12 mm.

2) Two-third of the final load at which the net settlement

attains a value of 6 mm.

3) 50 percent of the final load at which the total settlement equals one-tenth of the pile diameter.

In the above criteria, total settlement is the settlement under the full applied load, and the net settlement is the settlement that remains after the applied load is removed.

For large diameter piles (24" dia. and above) the safe bearing capacity is worked out on the basis of 1" instead of 1/2" as stated in clause above.

In uniform diameter piles, all the above criteria may be applied. But, for an under-reamed pile, criteria No. 3 cannot be applied straight away as it is not clear which diameter should be considered. If the stem diameter is considered, it will provide comparatively much lower value than if its bulb diameter were considered. Moreover, criteria should be different for single, double and triple under-reamed piles. It seems logical that as compared to a uniform dia. pile, ultimate bearing capacity of under-reamed piles should be worked out on an increasing percentage over stem diameter depending upon the number of bulbs. There being no fixed criteria available for deciding the ultimate capacity of piles, the values have been worked out by taking 10 percent of stem diameter for uniform diameter piles and 10 percent of the bulb diameter for the under-reamed piles. From the load settlement curves it would be seen that all piles could not be tested to 10 percent of base diameter due to difficulties of loading. From the uniform diameter pile test, it is indicated that the load settlement curves should flatten out. Based on this, the load settlement curves were extended and loads against the required settlement (10% of base diameter) were extrapolated. The ultimate load bearing capacity obtained are shown in Table 4.

TABLE 4

Ultimate Bearing Capacity of Piles

Sl. No.	Type of pile	Type of test	Total settlement criteria	Ultimate capacity Tons
1	Uniform dia	Compression	10% of base dia (30 mm)	11.2
2	Uniform dia	Pull out	10% of base dia (30 mm) (curve flatters earlier)	7.0
3	Single under-reamed	Compression	10% of base dia i.e. under-reamed dia (75 mm)	17.0
4	Double under-reamed	Compression		22.3
5	Triple under-reamed	Compression		26.6
6	Group of two S.U. pile	Compression (cap resting in ground)	Load test carried upto settlement of 14 mm only	33.5
7	Group of two S.U. pile	Compression (Soil from below the pile cap removed)	10% of pile base dia.i.e., under-reamed dia(75 mm)	27.5

The group of two single under-reamed piles was tested in the first instance with its cap resting in the ground. In this case, comparatively high bearing capacity is indicated. However, without separation of the effect of pile cap, the previously shown criteria cannot be applied for the bearing capacity of the group. After carrying out this test, upto about 33.5 tons load, the test was discontinued and the contact between the pile cap and the ground was eliminated by removing the soil from below the pile cap. Of course, the results will not be exactly applicable in the initial stages of loading, however, behaviour towards the ultimate will not be materially affected and the curve shown in Fig. 28 may be considered fairly indicative of the group behaviour. Bearing capacity extrapolated from the settlement criteria (10% of the bulb dia.) works out to about 27.5 tons.

CHAPTER 4

RESULTS OF BEARING CAPACITY TESTS ON PILES

4.1 General

Bearing capacity of the piles obtained from actual load tests as shown in Table 4 have been compared against those obtained from soil properties from laboratory tests and field tests-viz. Static cone penetration and Standard penetration tests. Besides this an empirical approach is also made towards calculating bearing capacity of these piles.

4.2 Bearing Capacity from Soil Properties

4.2.1 Criteria I - Shear Consideration

Calculations are made on the criteria that the bearing is provided by the toe in the case of uniform diameter piles or the bearing provided by the projected area of an under-ream bulb, while the friction is considered on a soil column enclosed between the two extreme bulbs and the pile shaft beyond the uppermost bulb. While calculating the bearing at the toe, the overburden at that level is also taken into consideration. The ultimate bearing capacity Q_u is given by the formula (Jain and Sharma) -

$$Q_u = (A_p N_b C_p + \gamma H N_q A_p) + \alpha \bar{C} A_s$$

where A_p = Area of the pile base (projected area of the bulb in case of an under-reamed pile)

N_b = Bearing capacity factor taken as 5. In cohesive soils, often N_b = 9 had been recommended. Later Skempton (1966) gave a reduction factor of 0.8. However, Terzaghi has suggested a reduction in shearing strength by two third in weak soils where local shear failure is likely. Thus, the value of the factor may be taken as $9 \times 0.8 \times 2/3 = 4.8$ say 5. This value of N_b is further confirmed by the work carried out by Vesic (1965).

C_p = Average shearing strength as obtained by averaging out the cohesion obtained by triaxial tests and shearing strength obtained by the unconfined compression strength tests.

γ = Average soil density over the pile length (submerged density for pile under water)

H = Total depth of pile from ground level to toe.

N_q = Bearing capacity factor. This will depend on the angle of internal friction Φ of the soil as given in Table 5 (after Vesic 1965).

α = Reduction factor which may be taken as 0.5. In case of double and multi-under-reamed piles, shearing takes place along the surface of a cylinder of diameter equal to the under-reamed diameter, and for this portion (between the centre lines of the first and last under-reamed) the value of α is taken as 1.

$\bar{C}$ = Average cohesion as obtained by triaxial tests along the pile length.

A_s = Surface area of pile shaft. In case of under-reamed piles, the portion upto the top of bulb is considered. In case of double and triple under-reamed pile, the diameter of the portion between the first and the last under-ream should be taken equal to the under-ream diameter.

The ultimate bearing capacity values calculated from the above expression are given in column 2 of table 6. Example for double under-reamed pile is given below:

<u>Double Under-Reamed Pile</u>

$$Q_u = (A_p\, N_b\, C_p + \gamma\, H\, N_q\, A_p) + \alpha\, \bar{C}\, A_s$$

For the above pile

$$A_p = \frac{\pi \times 75^2}{4} \text{ sq cms.,} \qquad N_b = 5$$

TABLE 5

Bearing Capacity Factor N_q in the Case of Local or Punching Shear Failure

Angle of internal friction ϕ Degrees	Bearing capacity factor N_q
0	1.0
5	1.2
10	1.6
15	2.2
20	3.3
25	5.3
30	9.5
35	18.7
40	42.5
45	115.0
50	422.0

TABLE 6. Ultimate Capacities From Soil Properties

Type of pile	Calculate ultimate capacities		Observed ultimate capacities (10% of base dia criteria)	Difference (%) from observed	
	Shearing consideration	Bearing consideration		Shearing consideration	Bearing consideration
(1)	(2)	(3)	(4)	(5)	(6)
Uniform diameter	10.62 (1.6+0.62)+8.4)	10.62 (1.6+0.62)+8.4	11.2	-5.2	-5.2
Single under-reamed	16.55 (6.6+3.40)+6.5	16.55 (6.6+3.40)+6.5	17.0	-2.65	-2.65
Double under-reamed	24.50 (6.6+3.4)+14.5	24.93 (15.45+5.00)+4.48	22.8	+7.45	+11.2
Triple under-reamed	33.25 (16.6+3.40)+23.2	30.45 (21.9+6.3)+2.3	26.6	+25.0	+14.5

$C_p = 0.3$ kg/cm², $\gamma = 1.92$ gms/cc (upto 1 M depth)

and= 0.92 gm/cc (below water table)

$H = (1 + 2.90) = 3.90$ M. $N_q = 1.6$ ($\phi = 10^o$)

$\alpha = 0.5$ (For shaft friction)

and = 1.0 (For cylindrical portion circumscribing the under-ream)

$\bar{C} = 0.38$ kg/cm², $A_s = \pi \times 30 \times 250$ sq cm. (for shaft friction)

and = $\pi \times 75 \times 112.5$ (for cylindrical portion circumscribing the under-reams)

$Q_u = (6600 + 3400) + (4480 + 10030) = 24510$ kg i.e. 24.5 T.

4.2.2 Criteria II: Bearing Consideration

If every bulb is considered towards bearing individually, the skin friction is considered over the shaft portion extending above the top most bulb only. For point bearing, the projected area of the lowermost bulb is taken. In double and triple under-reamed piles, in addition to above, the projected area in the form of annular rings for the remaining bulbs is also considered. Overburden is taken individually at the centre of each bulb.

The ultimate bearing capacity values obtained on the above criteria are shown in Column 3, Table-6. An example for double under-reamed pile is given below -

The same formula as in clause 4.2.1 shall be used here.

$A_p = \frac{\pi \times 75^2}{4}$ sq.cm (for bottom bulb)

and = $\frac{\pi}{4} \times (75^2 - 30^2)$ sq cm (for upper bulb i.e. annular ring)

$N_b = 5$ for both the bulbs , $C_p = 0.30$ kg/cm² (for bottom bulb)

and = 0.48 kg/cm^2 (for upper bulb)

γ = 1.92 gms/cc (above water table)

and = 0.92 gms/cc (below water table)

H = (1 + 2.9) = 3.9 M (for bottom bulb)

and = (1 + 1.74) = 2.74 M (for upper bulb)

N_q = 1.6 ($\phi = 10^o$), 1.2 ($\phi = 5^o$) (for upper bulb)
(for bottom bulb).

$\alpha = 0.5$, $\bar{C} = 0.38$, $A_s = \pi \times 30 \times 250$ sq cm.

Q_u = (6600 + 8850) + 3400 + 1600 + 4480

= 15.45 + 5.00 + 4.48 = 24.93 tons.

4.3 Bearing Capacity from Static Cone Penetration Test

Similar to the previous considerations ultimate bearing capacities are calculated from Static cone penetration values when bearing is considered at the toe for the lower most bulb and in the other case when individual bearing of every bulb is considered. The following expression (Dinesh Mohan, Jain and Kumar, 1963) has been used for working out the ultimate bearing capacity values Q_u

$$Q_u = \frac{q_c\ (av.) \times A_s}{50} + q_c \times A_p$$

where $q_c(av.)$ = Average static cone penetration resistance over the length of the pile.

A_s = Surface area of the pile shaft. In case of under-reamed piles, the portion upto top of bulb is considered. In double and triple under-reamed piles, the diameter of the portion between the first and the last under-ream should be taken equal to the under-ream diameter.

q_c = Average static cone penetration resistance around the pile toe (considered between 1 x dia above and 2 x dia. below the pile base.

For under-reamed piles, the base is to be taken at the centre of the bulb under consideration).

A_p = Area of the pile base (projected area of the bulb in case of under-reamed pile).

4.3.1 Criteria I - Shear Consideration

Calculations are made on the criteria that the bearing is provided by the toe in case of uniform diameter piles or, the bearing provided by the projected area of an under-ream bulb, while friction is considered on a soil column enclosed between the two extreme bulbs and the pile shaft beyond the upper most bulb. No overburden is separately considered in this case as the cone penetration resistance result is inclusive of the same. Rest of the considerations for calculation are the same as in clause.4.2.1. The bearing capacities calculated are shown in column 2 of table 7. An illustration for double under-reamed pile is given below:

$$Q_u = \frac{q_c(av.) \times A_s}{50} + q_c \cdot A_p$$

For the pile,

$q_c(av.) = 7$ kg/cm^2 (For both shaft and cylindrical portion circumscribing the under-reams)

$A_s = \pi \times 30 \times 250$ sq cms (shaft portion)

and $= \pi \times 75 \times 112.5$ sq cm (for cylinder circumscribing the under-reams).

$q_c = 6$ kg/cm^3, $A_p = \frac{\pi \times 75^2}{4}$ sq cms.

$Q_u = (3.3 + 3.72) + 26.50$ i.e. 33.52 Tons.

4.3.2 Criteria II - Bearing Consideration

If every bulb is considered towards bearing then same

TABLE 7. Ultimate Capacities from Static Cone Penetration Test

Type of pile	Calculated ultimate capacities		Observed ultimate capacities	Difference (%) from observed	
	Shearing consideration	Bearing consideration		Shearing consideration	Bearing consideration
(1)	(2)	(3)	(4)	(5)	(6)
Uniform diameter	10.20 (5.95+4.25)	10.20 (5.95+4.25)	11.2	-8.9	-8.9
Single under-reamed	31.20 (4.7+26.5)	31.20 (4.7 + 26.5)	17.0	+83.5	+83.5
Double under-reamed	33.52 (7.05 + 26.5)	66.60 (3.3 + 63.3)	22.8	+47	+192
Triple under-reamed	35.66 (9.16 + 26.50)	85.32 (1.72 + 83.90)	26.6	+34	+220

criteria as given in clause 4.2.2 would be applicable except that no overburden is separately taken into consideration and that the value of q_c is taken as an average of 1 x width of the ring above the centre of bulb and 2 x width of the ring below where the width of ring is equal to $\frac{1}{2}$ x (Diameter of bulb minus diameter of shaft).

The bearing capacities worked out based on the above assumption are given in column 3, table 7. An example for the double under-reamed pile is given below:

Formula will be used same as above in clause 4.3.1.

q_c (av.) = 7 kg/cm², $A_s = \pi \times 30 \times 250$ sq cms.

q_c = 6 kg/cm² (for bottom bulb)

and = 10 kg/cm² (for upper bulb)

$A_p = \frac{\pi \times 75^2}{4}$ sq. cm (for bottom bulb)

and = $\frac{\pi \times (75^2 - 30^2)}{4}$ sq. cm. (for upper bulb annular ring)

$Q_u = 3.3 + (26.50 + 36.80)$ i.e. 66.6 Tons.

4.4 Bearing Capacity Based on Standard Penetration Test Values

Meyerhof (1956) has given the following relation correlating the SPT value N and Static penetration resistance q_c.

$q_c = 2\ N$ for clay

$q_c = 4\ N$ for sand

Sengupta (1964) has confirmed that in silty clay the former relation holds good. It will also be noticed that for the depth of piles considered, the number of blows for dynamic cone penetration test are almost equal to the SPT values. Accordingly, after putting the q_c values as 2 N in formulae discussed in article 4.3.1 and 4.3.2, the respective bearing capacities obtained are shown in columns 2 and 3 of table 8. Illustration is given below for a double under-reamed pile using the above relation from two criteria V and VI.

(V). From existing relation formula is

$$Q_u = \frac{N_1 \times A_s}{25} + 2\, N_2 \times A_p$$

Here $N_1 = 3$ (for shaft friction)

and $= 4$ (for circumscribing cylinder)

$A_s = \pi \times 30 \times 250$ sq.cm. (for shaft friction)

and $= \pi \times 75 \times 112.5$ sq.cm. (for circumscribing cylinder)

$N_2 = 4$, $A_p = \frac{\pi \times 75^2}{4}$ sq.cms.

$Q_u = (2.82 + 3.18) + 35.3$ i.e. 41.3 Tons

(VI). Relation is

$$Q_u = \frac{N_1 \times A_s}{25} + 2\, N_2\, A_p$$

Here $N_1 = 3$, $A_s = \pi \times 30 \times 250$ sq.cms.

$N_2 = 4$ (for both the bulbs)

$A_p = \frac{\pi \times 75^2}{4}$ sq.cms (for bottom bulb)

and $= \frac{\pi \times (75^2 - 30^2)}{4}$ sq.cm (for upper bulb)

$Q_u = 2.82 + (35.3 + 29.5)$ i.e. 67.62 Tons

4.5 Arbitrary Factors for Skin Friction and Point Bearing

Bearing capacity of a uniform diameter pile in downward

TABLE 8. Ultimate Capacities From 'N' Values and Emperical Relations

Type of pile	Calculated ultimate capacities (Tons)					
	Existing relation		Calculated factors from actual load test		Empirical relation	
	Shearing	Bearing	Shearing	Bearing	Shearing	Bearing
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Uniform diameter	10.75 (5.1 + 5.65)	10.75 (5.1 + 5.65)	11.2 (6.23 + 4.97)	11.2 (6.23 + 3.4)	11.2 (7.8 + 3.4)	11.2 (7.8 + 3.4)
Single under-reamed	39.35 (4.05 + 35.3)	39.35 4.05 + 35.3)	35.93 (4.93 + 31.0)	35.93 (4.93 + 31.0)	15.23 (5.73 + 9.5)	15.23 (5.73 + 9.5)
Double under-reamed	41.30 (6.0 + 35.3)	67.62 (2.82 + 64.8)	38.32 (7.32 + 31.0)	60.34 (3.44 + 56.9)	17.9 (8.4 + 9.5)	21.3 (3.9 + 17.4)
Triple under-reamed	43.13 (7.83 + 35.3)	81.02 (1.47 + 79.55)	40.55 (9.55 + 31.0)	71.64 (1.79 + 69.85)	20.53 (11.03 + 9.5)	23.38 (2.03 + 21.35)

TABLE 8 (Continued)

Observed ultimate capacities (Tons	Difference (%) from observed					
	Exiting relation		Calculated factors		Empirical relations	
	Shearing	Bearing	Shearing	Bearing	Shearing	Bearing
(8)	(9)	(10)	(11)	(12)	(13)	(14)
11.2	-4.0	-4.0	-	-	-	-
17.0	+131	+131	+111	+111	-10.4	-10.4
22.8	+81	+196	+68.2	+164	-21.4	-6.6
26.6	+62	+204	+52.5	+169	-22.8	-12.1

loading and uplift are shown in Table 4 as 11.2 and 7 tons respectively. From these values point bearing and skin friction can be worked out (assuming that skin friction in downward load and pull-out is the same after correcting for the weight of the pile). On the basis of these values and the standard penetration test data, factors can be worked out for computing the bearing capacity from SPT values.

The factor for skin friction is found from the expression

$$\frac{N_1 \times A_s}{\text{skin friction}}$$

where N_1 = average of SPT values over pile length i.e. 3

A_s = surface area of pile shaft i.e. $\pi \times 30 \times 450$ sq.cm.

and skin friction =(pull out resistance of pile-weight of pile)

$= (7-0.77) = 6.23$ Tons i.e. 6230 kg.

Thus factor $= \frac{3 \times (\pi \times 30 \times 450)}{6230} = 20.5$ (approximately)

The factor for point bearing has been worked out after deducting the skin friction as obtained from the pull-out test from the ultimate capacity of the pile as

$$\frac{N_2 \times A_p}{\text{point bearing}}$$

where N_2 = value of SPT at bearing level under consideration i.e. 4

A_p = cross-sectional area of pile toe i.e., $\frac{\pi \times 30^2}{4}$ sq.cm.

and point bearing =(total downward load-skin friction)

$= 11.2-6.23 = 4.97$ Tons i.e. 4970 kg.

Thus factor

$$= \frac{4 \times (\pi \times 30^2)/4}{4970} = 0.57 \text{ (approximately)}$$

Using these factors, the formula for ultimate bearing capacity Q_u can be witten as:

$$Q_u = \frac{N_1 \times A_s}{20.5} + \frac{N_2 \times A_p}{0.57}$$

Bearing capacities can be worked out from both the considerations (shearing and bearing) in which bearing at the toe only or/and the bearing for individual bulbs are considered as discussed in previous sections. The values are shown in column 4 and 5 of Table 8. The bearing capacity calculations for double under-reamed pile from the above relation for both the criteria are given below: (VII and VIII)

(VII). The formula is

$$Q_u = \frac{N_1 A_s}{20.5} + \frac{N_2 A_p}{0.57}$$

Here $N_1 = 3$ (for shaft friction)

and $= 4$ (for cylinder circumscribing the under-reams)

$A_s = \pi \times 30 \times 250$ sq.cms. (for shaft friction)

and $= \pi \times 75 \times 112.5$ sq.cms. (for circumscribing cylinder)

$N_2 = 4$, $A_p = \frac{\pi \times 75^2}{4}$ sq.cms.

$Q_u = (3.44 + 3.88) + 31.0$ i.e. 38.32 tons.

(VIII). Relation is the same as before -

$N_1 = 3$, $A_s = \pi \times 30 \times 250$ sq.cms

$N_2 = 4$ (For both the bulbs)

$A_p = \frac{\pi \times 75^2}{4}$ sq.cms (for bottom bulb)

and $= \frac{\pi \times (75^2 - 30^2)}{4}$ sq.cms, (for upper bulb)

$Q_u = 3.44 + (31.0 + 25.9)$ i.e. 60.34 Tons.

4.6 Bearing Capacity on the Basis of Stress Release due to Under-reaming

In article 4.5, the frictional resistance on pile stem in direct loading and uplift are assumed to be the same. However, it is a recognised fact that in uplift, comparatively less value of friction are observed. Therefore, it will be desirable if in calculating the skin friction part, computations are made by making an arbitrary increase over the skin friction, obtained by uplift tests. A 25% increase has been taken into consideration in working out the value. The value for point bearing for the uniform diameter pile has been worked out after deducting the skin friction as obtained by the above consideration. This value of point bearing has been reduced to 50 percent in the case of under-reamed piles because of release of stresses in the bulb portion and consequent decrease in bearing.

Pull-out resistance of uniform dia.pile = 7 Tons

Weight of pile = 0.77 Tons

Skin friction in pull-out = 6.23 Tons

Estimated skin friction in push = $\frac{1.23 \times 125}{100}$ = 7.8 Tons

i.e. 7800 kg.

$$= \frac{7800}{\pi \times 30 \times 450} = 0.17 \text{ kg/cm}^2 \text{ (approximately)}$$

Ultimate load in push (observed) = 11.2 Tons

Point resistance in push test = 11.2-7.8 = 3.4 Tons

i.e. 3400 kg.

$$= \frac{3400}{\pi/4 \times 30^2} = 4.3 \text{ kg/cm}^2 \text{ (approximately)}$$

For under-reamed piles, assumed point bearing = $\frac{4.3}{2}$ = 2.15 kg/cm^2.

Based on the above criteria, bearing capacity of piles have been worked out from both the considerations (shearing and bearing) and are shown in columns 6 and 7 of Table 8. An example of calculations for the bearing capacity of double under-reamed pile from the two criteria using the above values is illustrated below -

(IX). From the above criteria -

Q_u = Average unit skin friction X area of surface (A_s) + average point bearing x A_p.

Here average unit skin friction = 0.17 kg/cm^2

$A_s = \pi \times 30 \times 250$ sq. cms. (for shaft friction)

and $= \pi \times 75 \times \overset{112.5}{\cancel{250}}$ sq. cms. (for cylinder circumscribing the under-reams).

Average point bearing = 2.15 kg/cm^2

$$A_p = \frac{\pi \times 75^2}{4} \text{ sq. cms.}$$

Q_u = (3.9 + 4.5) + 9.5 = 17.9 Tons.

(X). Relation is same as before -

Here also average unit skin friction = 0.17 kg/cm^2

$A_s = \pi \times 30 \times 250$ sq. cms.

Average point bearing = 2.15 kg/cm^2 (for both the bulbs)

$$A_p = \frac{\pi \times 75^2}{4} \text{ sq. cms. (for bottom bulb)}$$

$$\text{and} = \frac{\pi(75^2 - 30^2)}{4} \text{ sq. cms. (for upper bulb annular ring)}$$

Q_u = 3.9 + (9.5 + 7.9) i.e. 21.3 Tons.

CHAPTER 5

DISCUSSION OF TEST RESULTS

Ultimate pile bearing capacities worked out from soil properties (table-6) show that the values are fairly comparable with the observed load test values, the triple under-reamed pile bearing capacity being somewhat on the high side. This may be probably due to a reduction in the actual strength of the strata at the level of the third (upper most) bulb, which was not fully revealed from the samples tested but was indicated by the field tests. The values are particularly increased in the frictional part (23.2 T) as in calculation of bearing capacity an average strength of soil has been considered.

Ultimate bearing capacities worked out from static cone penetration (table-7) shows good agreement in uniform diameter pile capacity. The skin friction part (5.95 Tons) also compares with the computed value from the actual pull-out resistance of the pile (6.2 Tons). The estimated values of under-reamed piles, however, are on the high side. This is due to an over estimation of bearing which is all the more conspicuous when bearing is considered for the individual bulbs. This is due to the fact that excavation of soil in the portion of under-ream bulb releases the natural stresses in the soil, specially when under-water, resulting in the loosening of the soil around. The estimated skin friction part, however, is under-estimated with increase in number of bulbs and this could be due to increase in frictional resistance when shearing is considered as shown in

Fig. 10.

Ultimate pile bearing capacities worked out from 'N' values, and the empirical relation discussed in clause 4.6, are shown in Table-8, columns 2 to 7. The values of uniform diameter pile (columns 2 and 3) are in agreement with the observed values whereas there is considerable increase in the estimated values for under-reamed piles. This is mostly due to an over-estimation of the bearing part, which is more conspicuous when the bearing is considered for individual bulb. The friction, as before is found to be under-estimated slightly with increase in number of bulbs.

The values in columns 4 and 5 are based on arbitrary factors from actual load tests of uniform diameter pile (clause 4.5). The estimated bearing capacities of under-reamed piles show over-estimation of bearing part as before but the skin friction (shearing) part appears to be more correct.

The values in columns 6 and 7 which are based on empirical relation (clause 4.6) indicate that the criteria based on bearing consideration provides more comparable values than the criteria for shearing consideration. This could be because in multiple under-reamed piles shearing is taken around the cylindrical surface and its value is likely to be more than the frictional resistance in a uniform diameter pile.

By comparing the values of skin friction and point bearing as obtained by various methods, it will be observed that skin friction can be directly computed from field test data with fair accuracy, the point bearing part particularly in under-reamed piles needs careful consideration in weak soils under water as it can be overestimated.

Table 9 provides factors worked out for the bearing part of pile capacities.

$$\text{Factor} = \frac{\text{Bearing part of pile capacity from the formula}}{\text{Actual load test value minus skin friction from formula}}$$

From table-9, columns 2 and 3 it would be apparent that the estimated values could be 3.5 times as high from bearing consideration whereas from shearing consideration, it would be about twice. In the latter case, least over-estimation is done. This may be due to the fact that an average value of penetration is taken over the length of pile whereas in case of bearing, there are chances of local disturbances in recording the strength of soil by this test. However, at the same time this method is still not common in India, as the equipment is costly. Itsuse is therefore limited. Therefore a simpler test like Dynamic cone penetration could be used which has been already correlated with N values. Considering N values based on existing relations (columns 4 and 5), the factor is found to vary from 1.88 to 3.3. This might be due to the fact that existing formulae might not be applicable in the case of short piles under consideration.

The factors in Table 9 (columns 6 and 7) indicate less variation in estimation of the bearing part in under-reamed piles. Hence, in estimating the bearing capacity of under-reamed piles, the bearing can be obtained by dividing the bearing part by a factor of 2.5 in the formula (clause 4.5). The 'N' values can be

TABLE 9. Factors on Bearing Part of Pile Capacities Calculated from Field Tests

Type of pile	Static cone penetration resistance		'N' Values (S.P.T.)			
			Existing relation		Calculated factors from actual load tests of uniform dia piles	
	Shearing	Bearing	Shearing	Bearing	Shearing	Bearing
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Uniform diameter	1.0	1.0	1.0	1.0	1.0	1.0
Single under-reamed	2.15	2.15	2.72	2.72	2.58	5.58
Double under-reamed	1.70	3.24	2.10	3.25	2.00	2.93
Triple under-reamed	1.52	3.37	1.88	3.30	1.83	2.81

obtained by carrying out a simple dynamic cone test.

Safe bearing capacities of piles as worked out from actual load tests are shown in Table-10. The ultimate loads based on total settlement criteria at 10 percent of base diameter are shown in column 3. After applying a factor of safety of 1.75 as recommended by Sengupta (1970) for weak soils, the safe loads are shown in column 6. Observed loads at 25 mm. settlement and safe load after applying a factor of safety of 1.5 are shown in columns 4 and 7. The observed values at 18 mm. net settlement are taken as safe loads after applying a factor of safety of 1.5 and are shown in columns 5 and 8. It will be observed that the safe loads are generally on the lower side for the net settlement criteria. The observed total settlements at the safe loads worked out are all within 10 mm. whereas net settlements are within 5 mm, which seemsto be quite within safe limits.

Loads worked out from the safe load table vide Building Digest 56 and IS:2911 Part I-1964, are shown in Table-11, alongwith those worked out earlier from other criteria, and it will be observed that they tally within practical limits.

As regards the group effect a test was carried out on a group of two single under-reamed piles kept at a spacing of 112.5 cms. (1.5 times the under-reamed diameter), cap resting in ground. The observed load at 25 mm. total settlement is about 37 tons Fig. 27. When the soil from below the pile cap is removed, observed load at 25 mm. settlement is 24 tons. If we

TAVLE 10. Safe Capacities of Piles from Actual Load Test

Sl. No.	Type of pile	Observed Load (Tons)			Safe Load (Tons)			Observed settlement (mm) at calculated safe load from the two criteria		
		Total settlement criteria		Net settlement	Total settlement criteria		Net settlement	Total settlement		Net settlement
		10% of base dia.	25 mm	18 mm	F.S. 1.75 on (3)	F.S. 1.5 on (4)	nt. F.S. 1.5 on (5)	on (6)	on (7)	on (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	Uniform dia.	11.2	10.85	10.8	6.4	7.24	7.2	3.5	5.0	3.5
2	Single under-reamed	17.0	13.5	12.75	9.72	9.0	8.5	9.2	7.2	4.8
3	Double under-reamed	22.8	18.75	18.25	13.0	12.5	12.5	6.4	6.0	4.0
4	Triple under-reamed	26.6	20.5	19.25	15.2	13.7	12.85	6.0	4.0	2.0

TABLE 11. Safe Loads for Under-Reamed Piles in Weak Soils

Sl. No.	Type of piles	Safe load as per load test — Total settlement criteria I	Safe load as per load test — Total settlement criteria II	Safe load as per load test — Net settlement criteria	Safe load as per building digest 56	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Single under-reamed	9.72	9.0	8.5	10.0	The safe load in column (6) are also as per IS: 2911 (Part-1)-1964 (under revision)
2	Double under-reamed	13.0	12.5	12.2	14.0	
3	Triple under-reamed	15.2	13.7	12.85	16.0	

work out the bearing for the pile cap (size 1.72 m x 0.60 m) on the basis of observed load, 1.2 kg/cm^2 at 25 mm. settlement, in the plate load test (Fig. 18), we get the value 12.4 tons. This when added to 24 gives 36.4 tons which is close to 37 tons. However, considering the curve showing double the loads taken by one single under-reamed pile, it will be observed that the value obtained from load test of group of 2 piles is less by about 18 percent than above. Out of this, deducting friction for 1 meter, this being the depth of pile cap, the reduction due to group effect is only 7 percent. The bearing contributed by the pile cap, on the other hand, is quite substantial.

In Calcutta, there is a practice that the bearing capacity of soil which is normally taken as $\frac{1}{2}$ kg/cm^2 is also taken into account when timber piles are driven through it for the pile cap. Thus the total bearing capacity is taken as bearing capacity of individual piles + $\frac{1}{2}$ kg/cm^2 for the soil under the pile cap. In short bored piles also, this criteria may probably work and if not full, atleast a part of the bearing capacity of the soil should be taken into account.

CHAPTER - 6

CONCLUDING REMARKS AND RECOMMENDATIONS

6.1 Conclusions

1. There is a good agreement in the observed bearing capacity of single and multiple under-reamed piles and those computed from soil properties.

2. By static cone penetration test the computed pile capacities are on higher side as compared to the observed ones.

3. If bearing capacities are computed by SPT values according to Meyerhof's recommendations, still higher values are obtained.

4. If point bearing and skin friction factors are worked out from load tests on uniform diameter piles for calculation of bearing capacity using SPT values, the pile capacities obtained are still higher but lower than those obtained from Meyerhof formulae. The bearing capacity of under-reamed piles can be worked out by dividing the bearing part with a factor of 2.5 in formula based on above factors.

5. Skin friction part of the bearing capacity of piles as determined from various formulae agrees well with that obtained in load tests. However, the higher values obtained by computations based on Static and Standard penetration values are mainly due to an over-estimating of point bearing part of bearing capacity to an extent of 2 to $3\frac{1}{2}$ times of the actual values.

6. If is felt that the skin friction obtained from pull-out test on uniform diameter piles should be increased by 25 percent

and the bearing for under-ream bulbs should be decreased by 50% in estimating the bearing capacity of under-reamed piles if a load test data is available for uniform diameter piles.

7. In computing bearing capacity of piles having more than one bulb, the value worked out considering bearing on all the bulbs are generally higher than those obtained by considering bearing at the lower most bulb and shearing along a cylindrical surface circumscribing the under-reams.

8. For working out safe loads from a load test on piles, a factor of safety of 1.5 on the observed load at 25 mm settlement appears to be reasonable.

9. Observed settlements at calculated safe loads from the two criteria i.e. total settlement and net settlement are within 10 mm and 5 mm respectively which are quite within safe limits.

10. Bearing capacity of under-reamed piles given in C.B.R.I. Digest 56 compres well with the observed values if the capacities are reduced to 50% in weak soils under water, and the values given are considered to be based on 25 mm settlement instead of 12 mm.

11. Bearing capacity of a group of two single under-reamed piles was found to reduce by 7 percent when the spacing is 1.5 times the under-reamed diameter. The arbitrary reduction sugges-ted in Indian Standard 2911 (Part 1), 1964 is 10 percent.

12. Contribution to bearing by the pile cap was almost equal to its individual bearing and therefore in computing the capacity of pile group, the bearing of pile cap should also be considered.

6.2 Recommendations

1. Bearing capacity of piles can be estimated from the shear properties of the soil determined from undisturbed samples.

2. In the absence of detailed investigations, a preliminary estimate of bearing capacity of piles in silty clays of poor bearing below water table ($N < 4$) can be made by dividing the bearing portion with a factor of 2.5.

3. The estimates should be based on the consideration that bearing is provided at the toe level only whereas for the portion between the bulbs, friction should be considered on a cylindrical surface circumscribing the under-ream bulbs.

4. As dynamic cone values are related to SPT values, it will be much more convenient to carryout dynamic cone penetration tests for estimating the bearing capacity of piles.

5. In weak soils, safe loads on piles should be worked out by applying a factor of safety 1.5 on the observed load at 25 mm total settlement.

6. Safe loads table given in CBRI Digest 56 should be considered at 25 mm settlement for weak soils.

7. A reduction of 10 percent in bearing capacity of a group of two piles should be made when the pile spacing is reduced to $1\frac{1}{2}$ times the under-reamed diameter against the normal spacing i.e. 2 times the under-reamed diameter.

8. In a group of short bored piles, a considerable portion of bearing may be contributed by the pile cap.

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APPENDIX - A

CBRI CONE PENETRATION TEST

Introduction

For an accurate assessment of local variation and erratic substrata condition, cone penetration tests provide an excellent method. The static penetrometer is a more useful test but it needs a more elaborate set up than a Dynamic cone test. The dynamic cone penetration test provides an almost continuous record of penetration resistance which is so useful in finding out any pockets in the substrata.

Technique of Dynamic Cone Penetration Test

This test is carried out somewhat similar to a SPT test. A 60° cone having a base diameter of 6.5 cm. is driven by a 64 kg hammer falling from a height of 30 cm. The number of blows are recorded every 15 cm. The total number of blows for 30 cm. penetration on the cone provides the penetration resistance (N_c).

The drill rods used for driving the cone are of 4.1 cms. dia. ('A' rods) against 6.5 cm. of the cone. A cavity is thus formed around the driving rods. At larger depths, (beyond 6 m) the sides of the hole start falling down thus providing friction on the rods. For this, use of bentonite slurry is made. A 5 percent by weight solution of good bentonite in water is made and it is circulated through the driving rods to stabilize the sides of the bore and thus eliminate friction.

The solution of bentonite should be well prepared and should be of uniform consistency. Presence of bentonite in form of lumps in the slurry may cause chocking up of the equipment.

Testing Procedure

A general assembly of the test set up is shown in Fig. 29. ~~[illegible]~~ A guide is fixed on ground. This helps in keeping the drill rods vertical and in position. The cone is connected to a driving rod and placed in position. A driving head with a guide rod is fitted to the drill rod. The guide rod is then connected through a flexible pipe to a water swivel and then through another larger tube to a reciprocating type piston pump. The pump is run by two persons. Slurry from the feeding tank is sucked into the pump, pushed upto the swivel, and then to the cone which has a few holes for slurry to come out. Driving of the cone is done simultaneously. For this, one or two persons rotate the handle of the winch. Another person pulls the rope holding the drop weight wound twice round the winch drum. This makes the drop weight to rise. The pull required is nominal. When the drop weight reaches the requisite height of 75 cms. a slight release in the pull keeps the weight in position. To drop the weight, the grip on the rope is released. This makes the drop weight fall under gravity. Efficient circulation of slurry is very necessary to get reliable results. This is effected by rotating the drill rods from the top after each foot of penetration.

The set up becomes much simpler when bentonite slurry is not to be used. There is no pump, flexible tubes, swivel etc. needed, and the process becomes very fast. However, the depth for such a test should be limited to 6 m unless it is confirmed by manually rotating the rods that skin friction developed on the rods is negligible even after 6 m depth.

Correlations

When bentonite slurry is used - $N_c = N$

When driving without slurry is used -

N_c = 1.5 N. for depth upto 3.m.

N_c = 1.75 N for depth upto 3.to 6.m.

Various Parts of the Equipment (for Dry Probing)

1. 60^{o} cones (push fit).
2. 'A' rods with nipples.
3. Tripod hoist with a manually operated winch fitted to it, and complete with a fixed pulley, built in ladder etc.
4. Boring guide.
5. 64 kg. drop weight complete with driving head and guide pipe.

APPENDIX - B

CONSTRUCTION OF UNDER-REAMED PILES IN GROUND WITH HIGH WATER TABLE

In clayey soils there is no difficulty because the amount of water seeping into the bore is small and it can be overcome by speeding up the process of boring and concreting. In sandy soils, however, the sides may cave in and this poses a more difficult problem. It is simple to construct a uniform diameter pile by using a cased bore hole but casings cannot be used for under-reamed piles. Bentonite slurry can be used for retaining the sides.

For concreting, the tremie pipe is provided with cap at the bottom and the reinforcement is placed inside the tremie so that it is not contaminated with the slurry. This method has been effectively used in constructing under-reamed piles.

EQUIPMENT

Boring Guide - The Auger boring guide developed earlier for use in construction of bored piles was provided with longer detachable collar. It is fixed on the lower side of the boring guide so as to effectively protect the upper end of the bore hole from widening due to the frequent insertion and extraction of the boring assembly.

Auger - A spiral earth auger was used with success. Sufficient care was taken not to rotate the handle too much otherwise the soil starts collecting over the spirals making it difficult to extract the auger.

Under-reamer - A modified portable under-reaming tool developed earlier for use in black cotton soils was used. The bucket is attached to the blades assembly through a bush bearing for smooth working. The bucket of the tool was provided with gravity valves which open down during extraction and break any vacuum at the bottom.

Boring and Under-reaming - Boring may be done partly in dry and partly in wet. For boring the flaps of the boring guide are opened, earth auger is introduced, pressed down and rotated. It is taken out when full. The soil is removed and the auger is again introduced. As it goes down, the upper flaps of the guide and thereafter the lower flaps also should be closed. The extensions are provided with sockets. The process is continued till required depth is reached.

Care should be taken to keep the top one metre of the bore hole truely vertical. If the bore hole gets inclined, the cutting tool may be used for scraping the sides and the bore made vertical.

For under reaming, the portable under-reaming tool is lowered into the bore hole. As pressure is applied from top, the blades try to widen out. When handle is rotated, the soil is cut from the sides and it falls into the bucket. The tool is taken out a few times to empty the bucket. The process is continued till under-reaming is complete.

Correct under-reaming is an important requirement. A pin inserted in the shaft of the under-reaming tool would control the maximum diameter required. This can be ascertained by (1) the vertical movement of the handle, (2) when no further soil is cut. The assembly is then pulled out and the bore which is full of slurry is ready for concreting.

CONCRETING

Casting of good quality concrete below water table calls for good planning and careful execution as difficulties are sometimes encountered. Pile should normally be concreted soon after the bore hole is ready. Concreting should be done under the supervision of a responsible person.

A tremie pipe is used for concreting under water. Normally the recommended size of the tremie pipe is 8 times the size of the aggregate. It should not be less than 15 cm. diameter. It depends on the size of reinforcement also.

For concreting, the tremie pipe, with its lower end closed by means of a detachable cap, is lowered into the bore hole and allowed to rest at the base (Fig. 30). It is held in position with a clamp hanging from the tripod. Most of the bentonite slurry over flows with the lowering of the tremie pipe. A funnel is now placed on the top of the tremie. The reinforcement+ is then lowered carefully through the tremie pipe and adjusted in position. It is advisable to pour two buckets of concrete in the tremie pipe before lowering of the reinforcement. It helps in providing bottom cover to the pile reinforcement. A wheel barrow may be used for concreting with advantage. The concrete should

+The reinforcement may preferably be in the form of a single angle iron piece since it provides little resistance to the flow of concrete and has good bond due to its large surface area.

be thoroughly mixed having a slump of at least 15 cm. The aggregate used should be well graded. After a few buckets of concrete is dropped in the pipe the cap provided at the bottom is detached by shaking the reinforcement and at the same time pulling up the pipe a little. With the progress of concreting the pipe is pulled up gradually. The bottom of the tremie pipe should be kept embedded in the fresh concrete throughout the concreting operation and concreting should be continuous. Rodding is normally sufficient to compact the concrete.

It is very desirable to keep sufficient concrete in the tremie when it is finally withdrawn. This helps in removing the laitance. It should be ensured that there is good concrete right to the top of the pile.

CONCLUSION

The technique is simple and can be adopted by local contractors. The equipments required are also easy to handle and are readily available.

APPENDIX 'C'

LOAD TEST ON PILES

C-1 Load Test Set-up

For uniform diameter and single under-reamed piles, reaction was obtained through rolled steel joints (RSJ) bolted down to holdfast type soil anchors as shown in Fig. 20.

For double under-reamed pile, reaction was obtained through two RSJ bolted down to two piles and also by putting some soil anchors across another set of RSJ put across the first set.

For the triple under-reamed piles and group of piles, reaction was obtained partly through two RSJ bolted down to two double under-reamed anchor piles and partly through soil anchors put across another set of RSJ.

A precise hydraulic jack (Simplex type) with detachable pump assembly was used for finding out the load transmitted to the piles. At lower loads, a proving ring of 10 tons capacity was also used.

Three dial gauges (.001") were used for each pile. These were fixed on datum bars which were placed on firm supports. One of the dial gauges had a longer travel (2"). The others were provided with precise spacers for extending their measuring range from 1/2" to over 2".

C-2 Method of Load Application

A load of about one tenth of the estimated ultimate load was first applied by a hydraulic jack in three to five minutes. It was maintained for about five minutes and then allowed to reduce itself due to the yielding of the ground. Within a few minutes, a state of equilibrium used to reach. The next higher load was then applied and the process repeated. For higher loads, the initial load was maintained for 10-15 minutes before it was allowed to diminish. The state of equilibrium was reached when rate of movement of pile top became less than .02 mm. per hour. The total time required by the method was considerably less than required in a maintain load test. At each stage of loading, a cycle of loading and unloading was also adopted in order to evaluate the net settlement of pile at each stage.

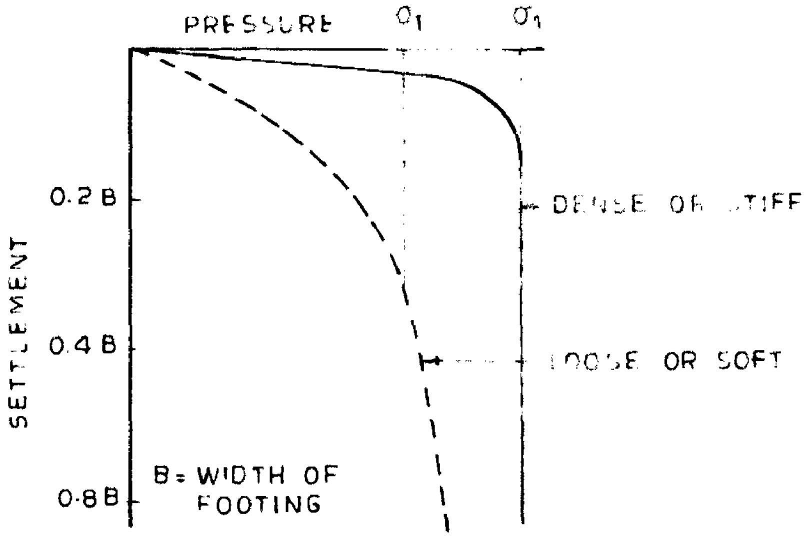


FIG. 1. PRESSURE - SETTLEMENT CURVES OF LOAD TESTS

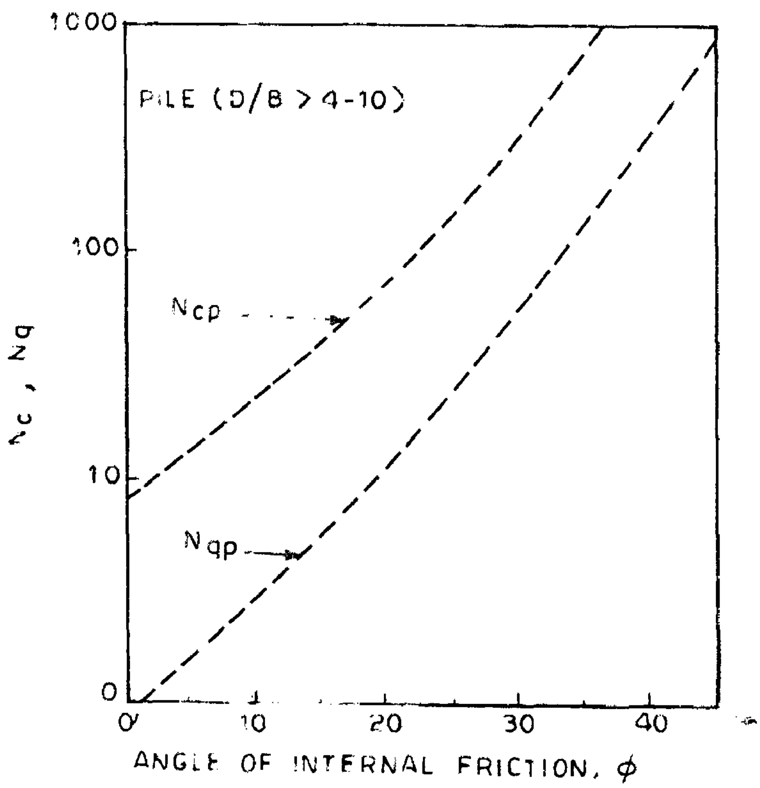
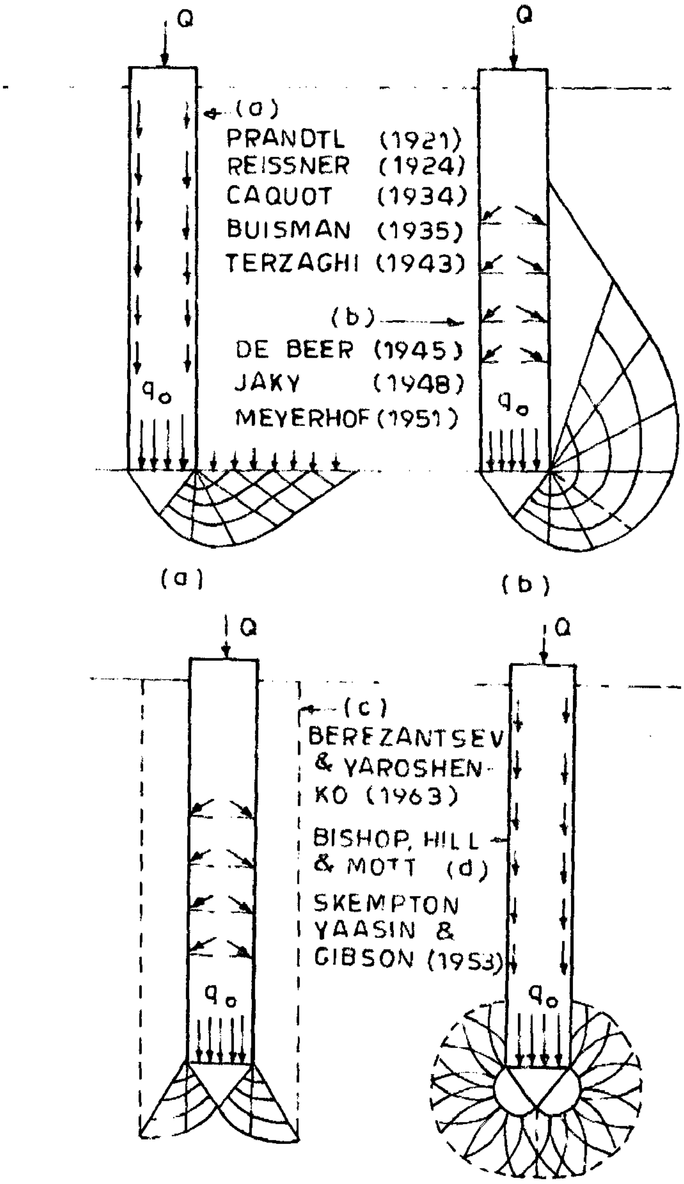
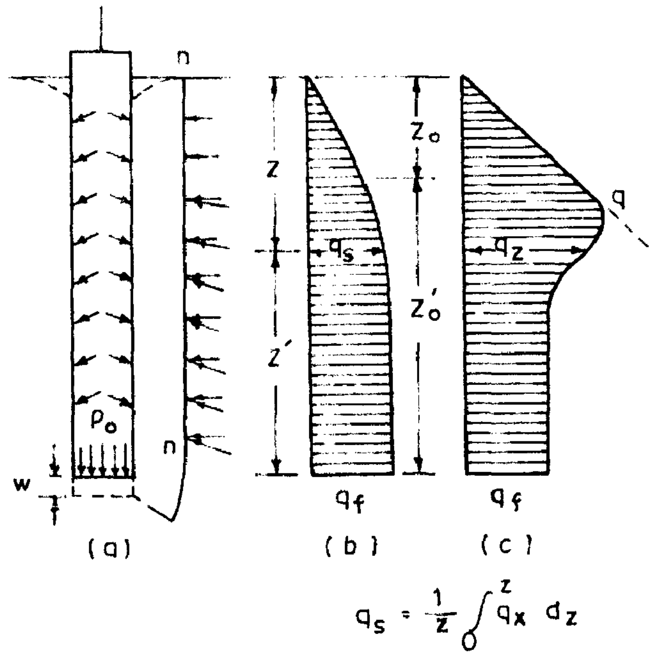


FIG. 2. MEYERHOF (1963) GENERAL BEARING CAPACITY FACTORS



(c) VESIC (1963)

FIG. 3_DIFFERENT FAILURE PATTERNS FOR CIRCULAR DEEP FOUNDATION (VESIC 1965)



$$q_s = \frac{1}{z}\int_0^z q_x \, dz$$

FIG. 4_STRESS DISTRIBUTION AROUND A DEEP FOUNDATION (AFTER VESIC 1965)

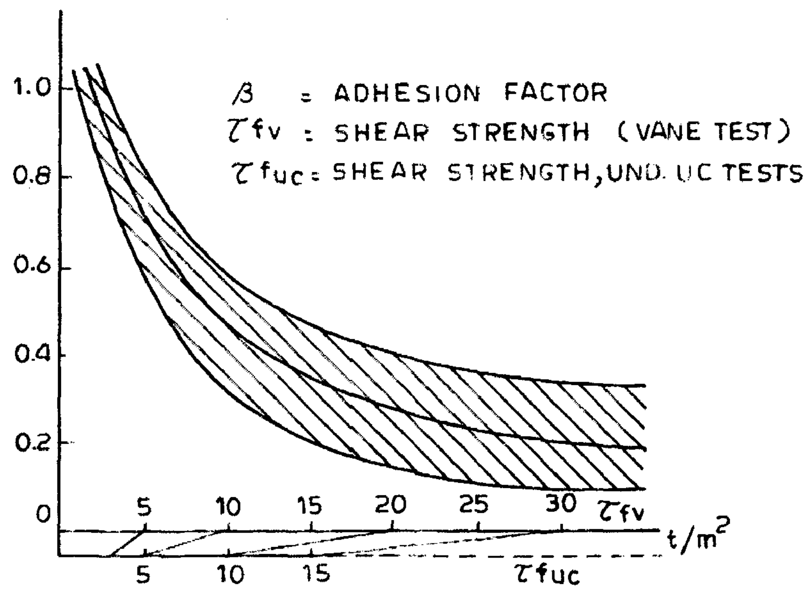
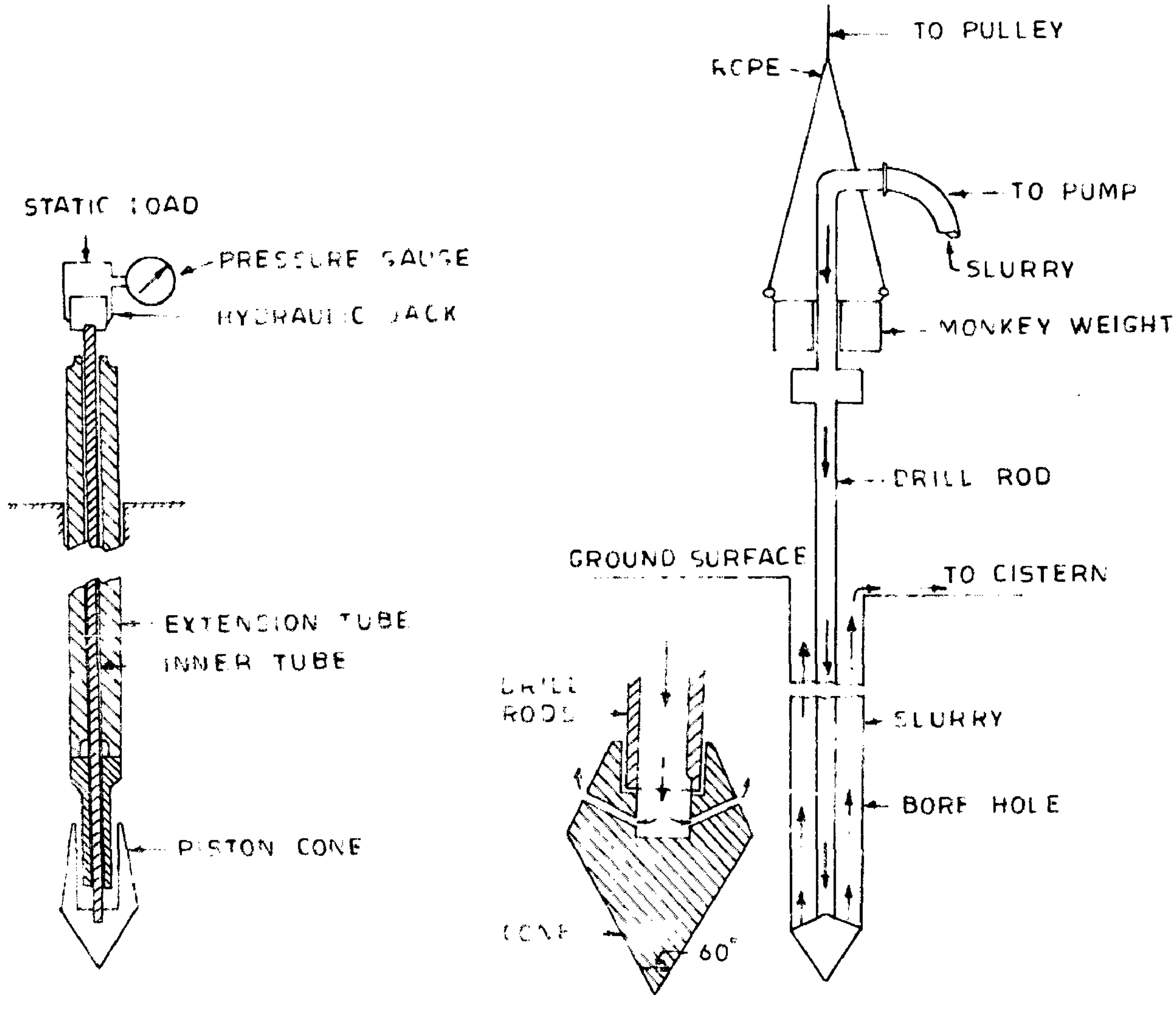


FIG. 5_RECOMMENDED ADHESION FACTORS FOR DRIVEN AND BORED PILES IN CLAYS (AFTER KERISEL 1965)



STATIC CONE PENETRATION TEST APPLIANCE

DYNAMIC CONE TEST ASSEMBLY

FIG. 6 CONE PENETRATION TEST ASSEMBLIES

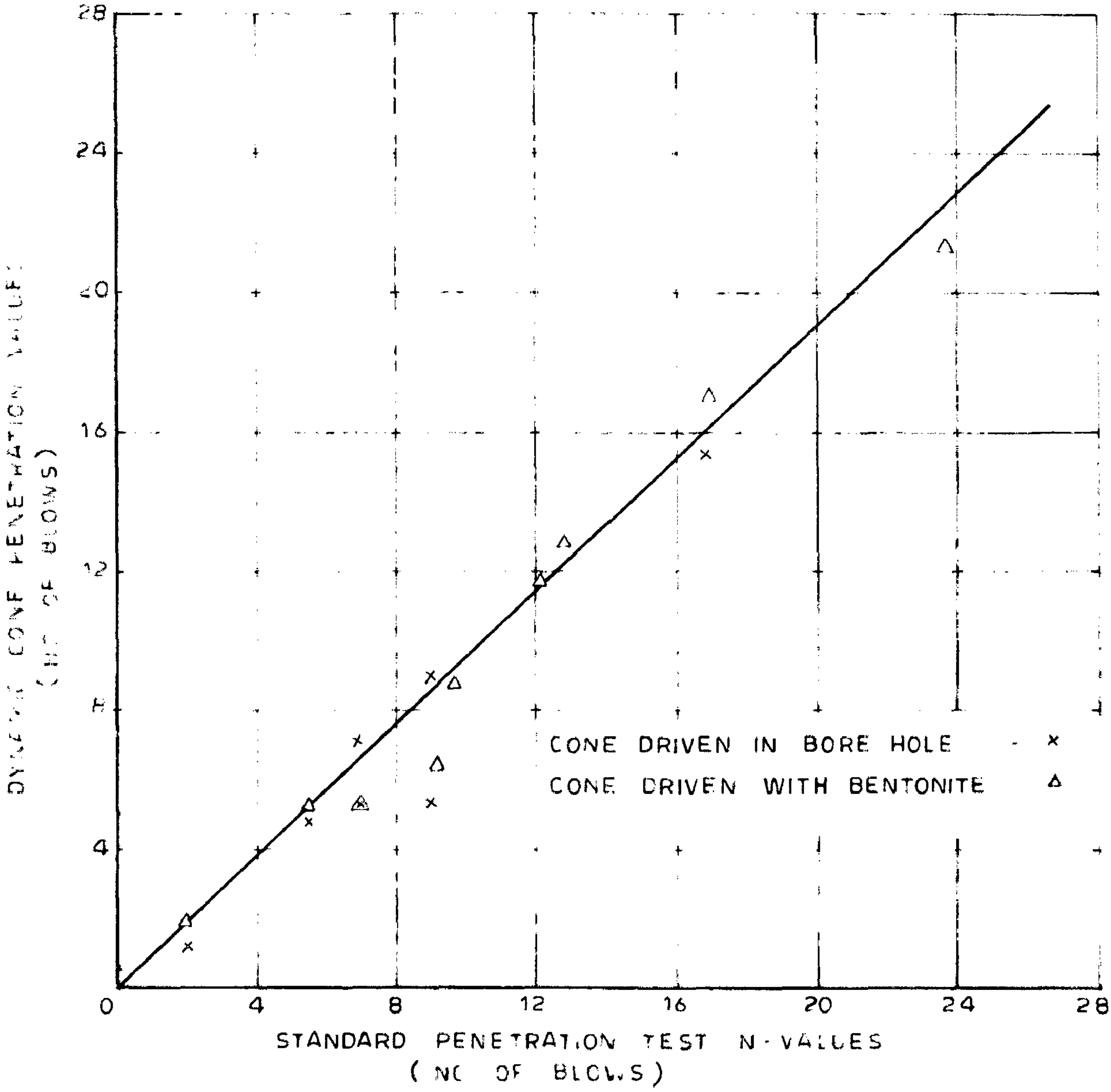


FIG. 7 - RELATION BETWEEN DYNAMIC CONE RESISTANCE AND N-VALUES (SANDY SOILS)

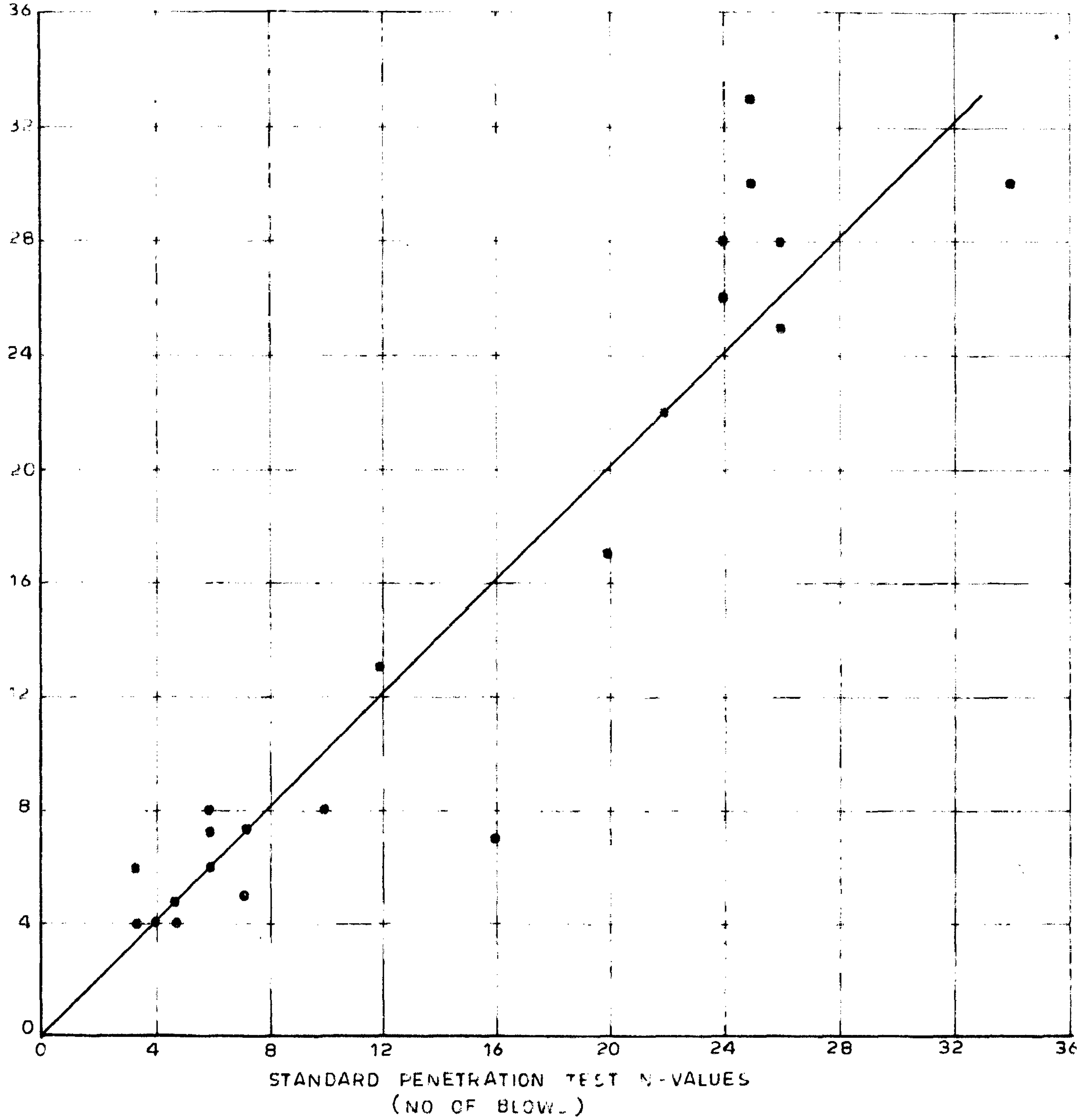


FIG. 8_RELATION BETWEEN DYNAMIC CONE RESISTANCE AND N VALUES (SILTY CLAY)

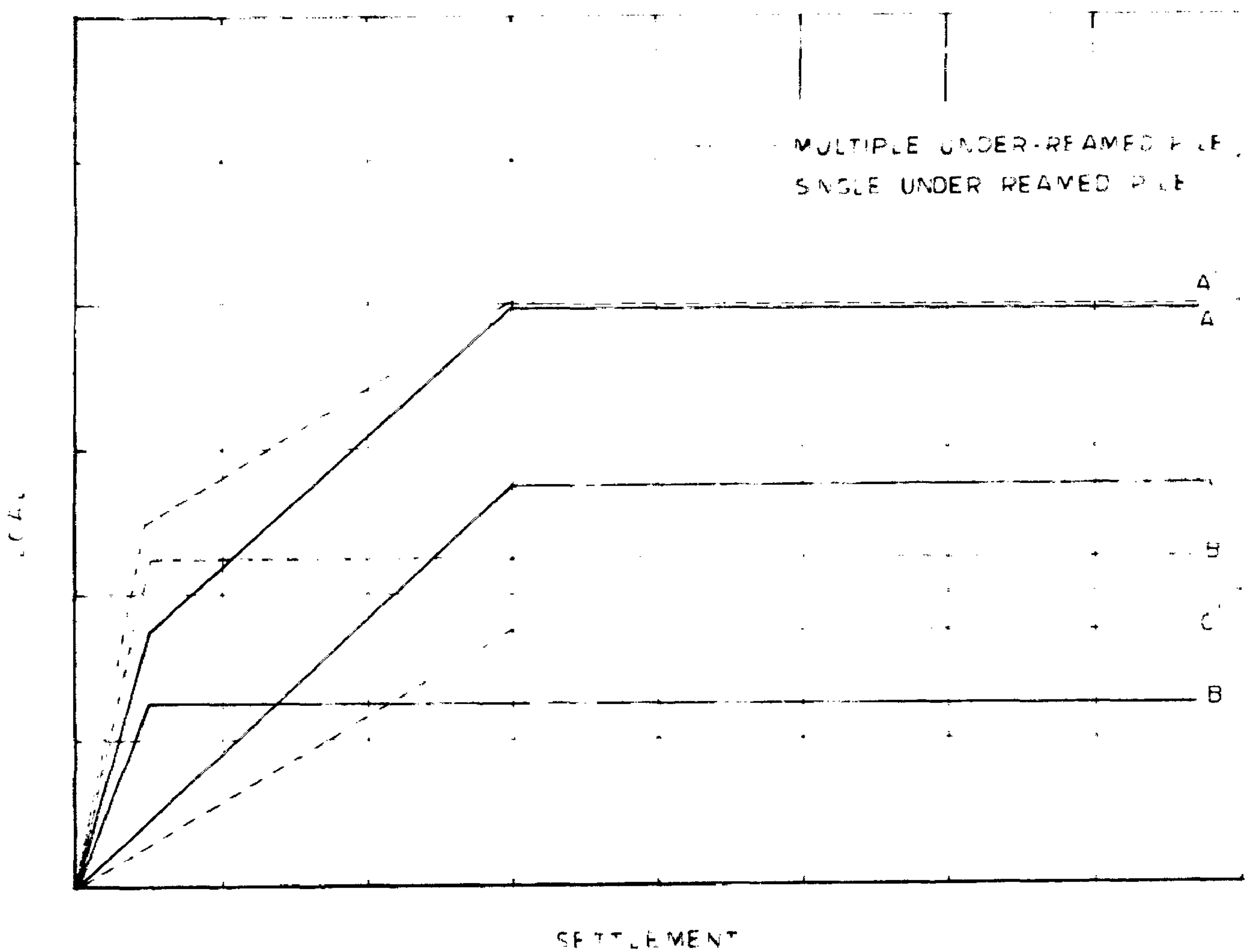
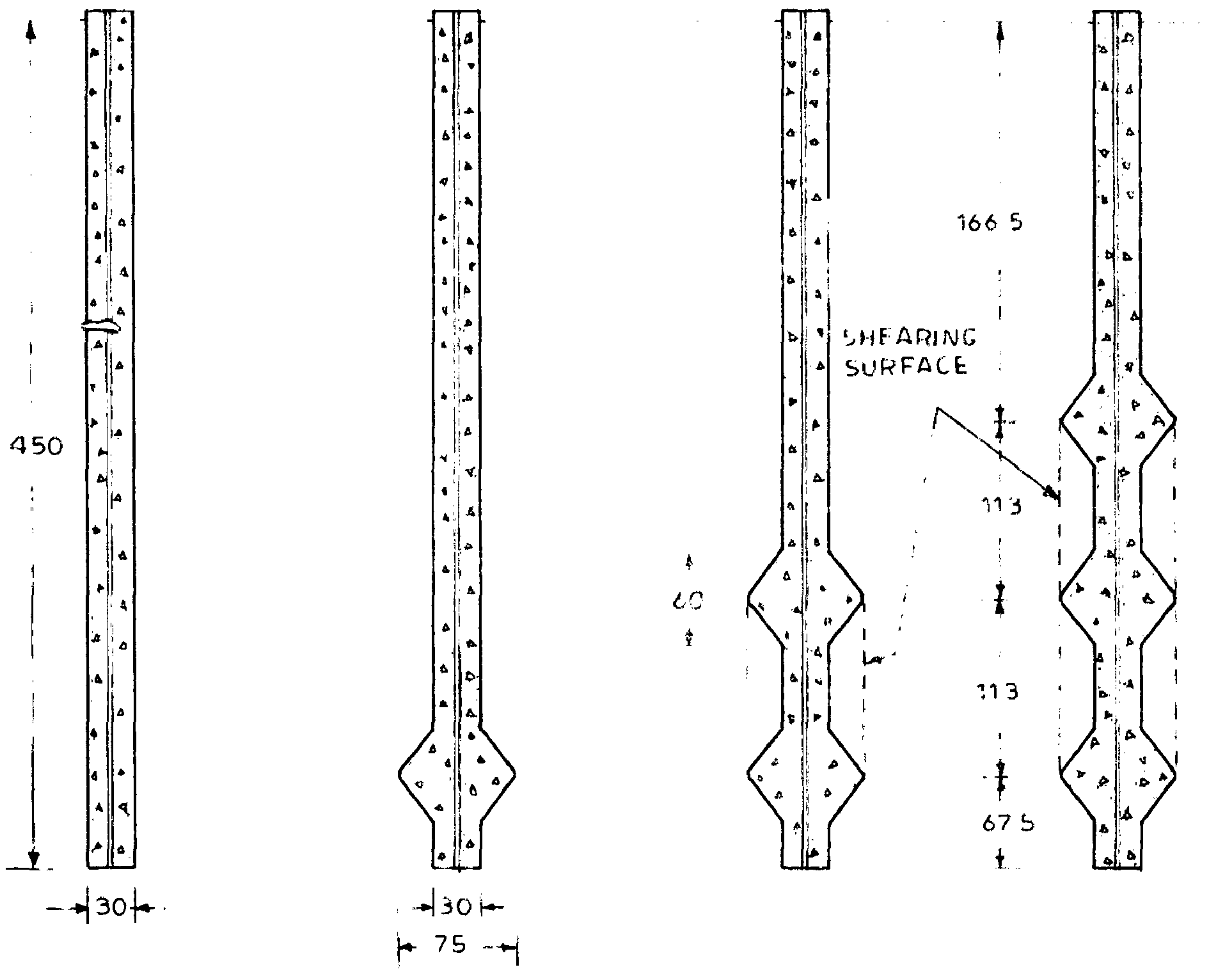


FIG 9. LOAD SETTLEMENT CURVES FOR AN IDEAL ELASTIC PLASTIC SOIL



ALL DIMENSIONS ARE IN CENTIMETERS

FIG 10-UNIFORM DIAMETER AND UNDER REAMED TEST PILES

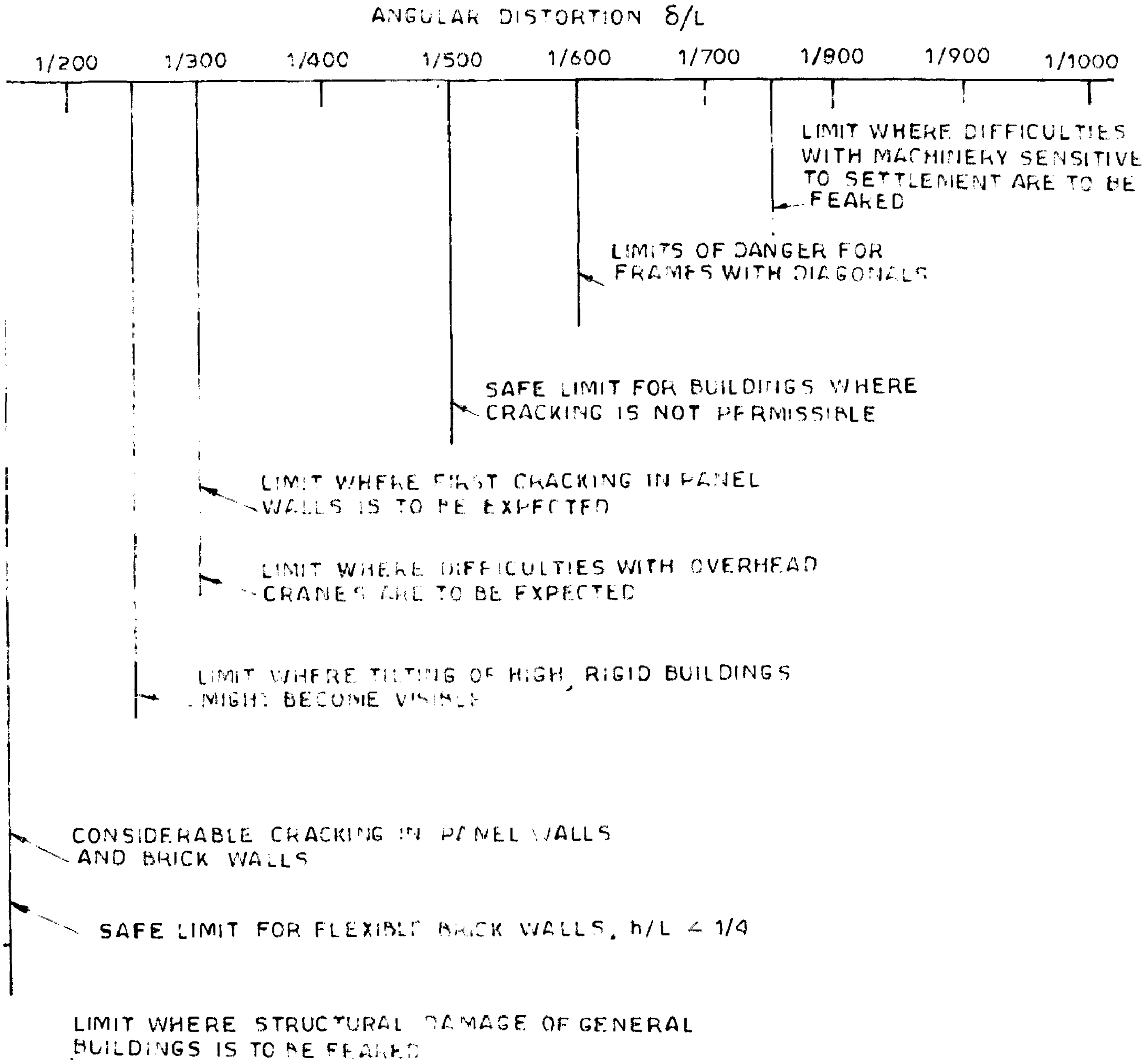


FIG. 1' _DAMAGE CRITERIA (AFTER BJERRUM 1963)

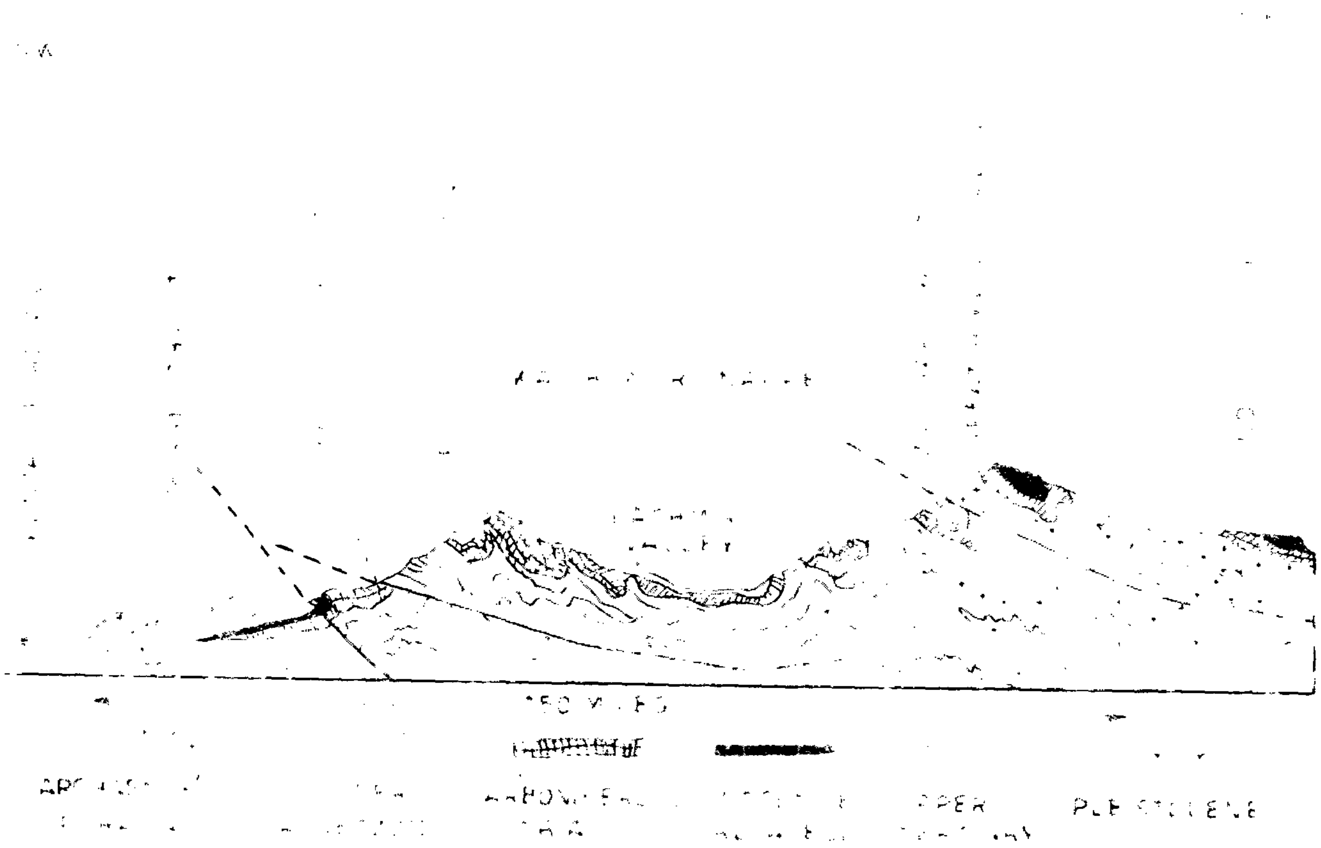


FIG. 12. DIAGRAMMATIC SECTION ACROSS THE KASHMIR HIMALAYA

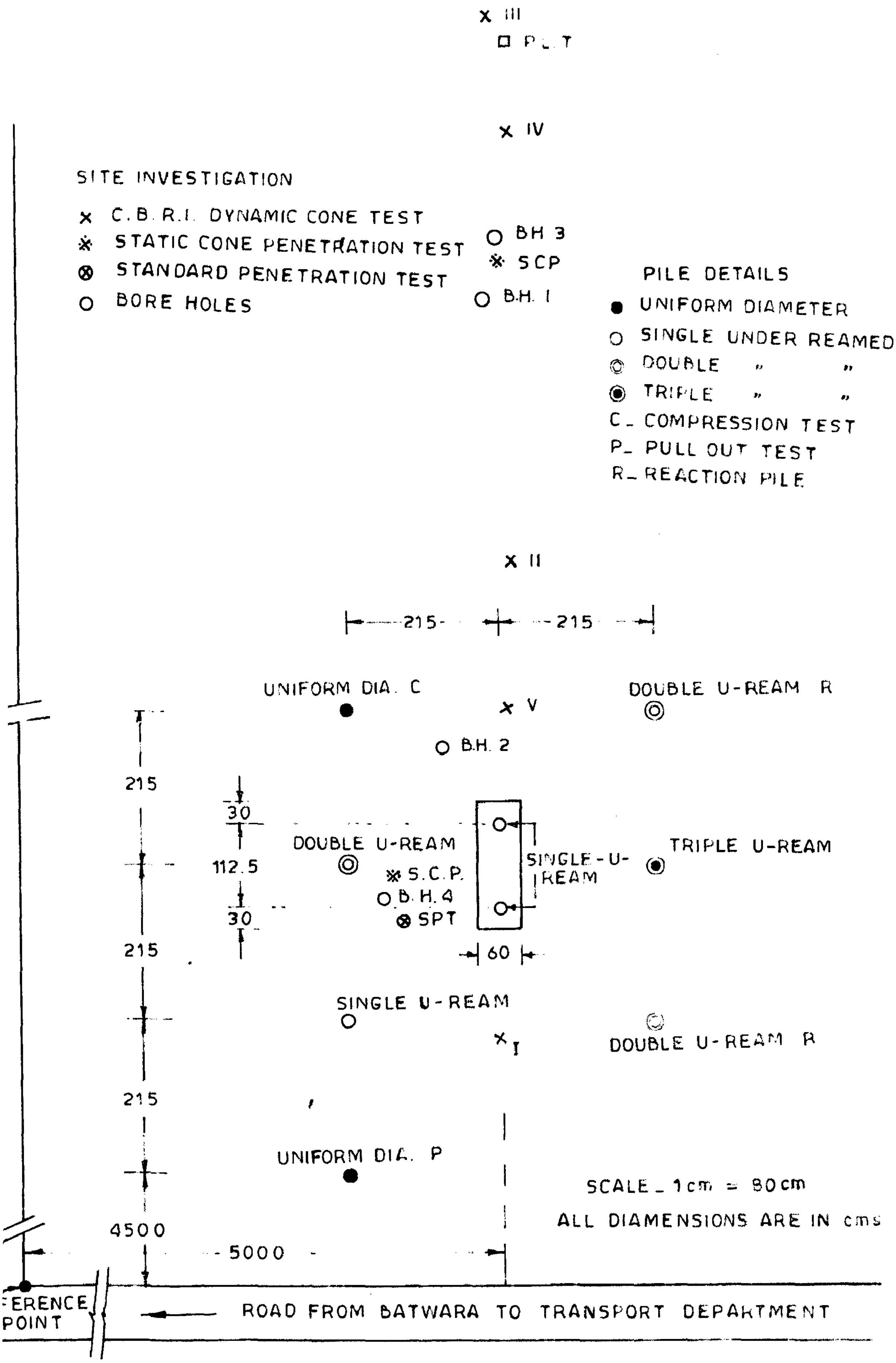


FIG. 13 – SITE PLAN SHOWING LOCATION OF TESTS

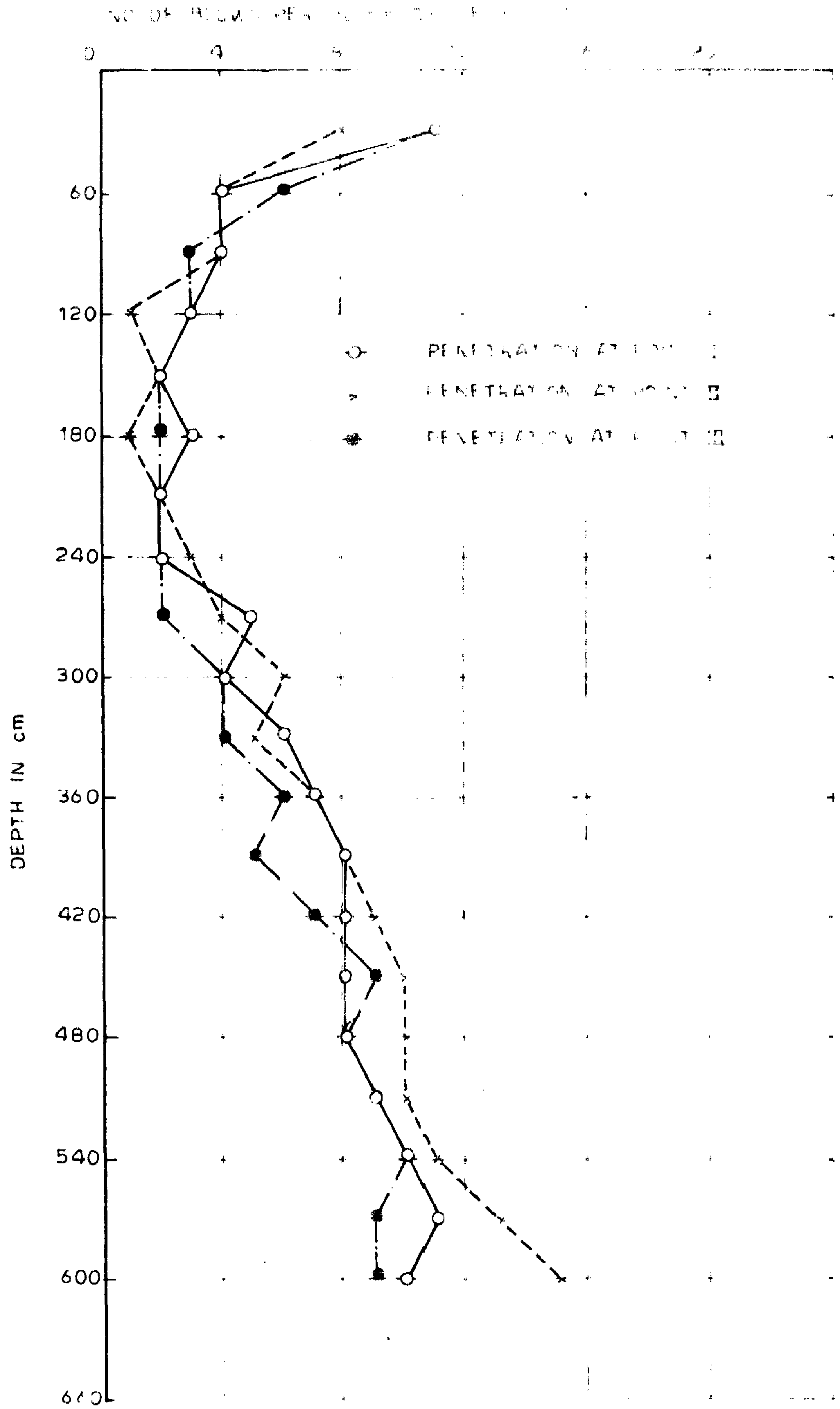


FIG.14- DYNAMIC CONE PENETRATION RESISTANCE ACROSS THE SITE

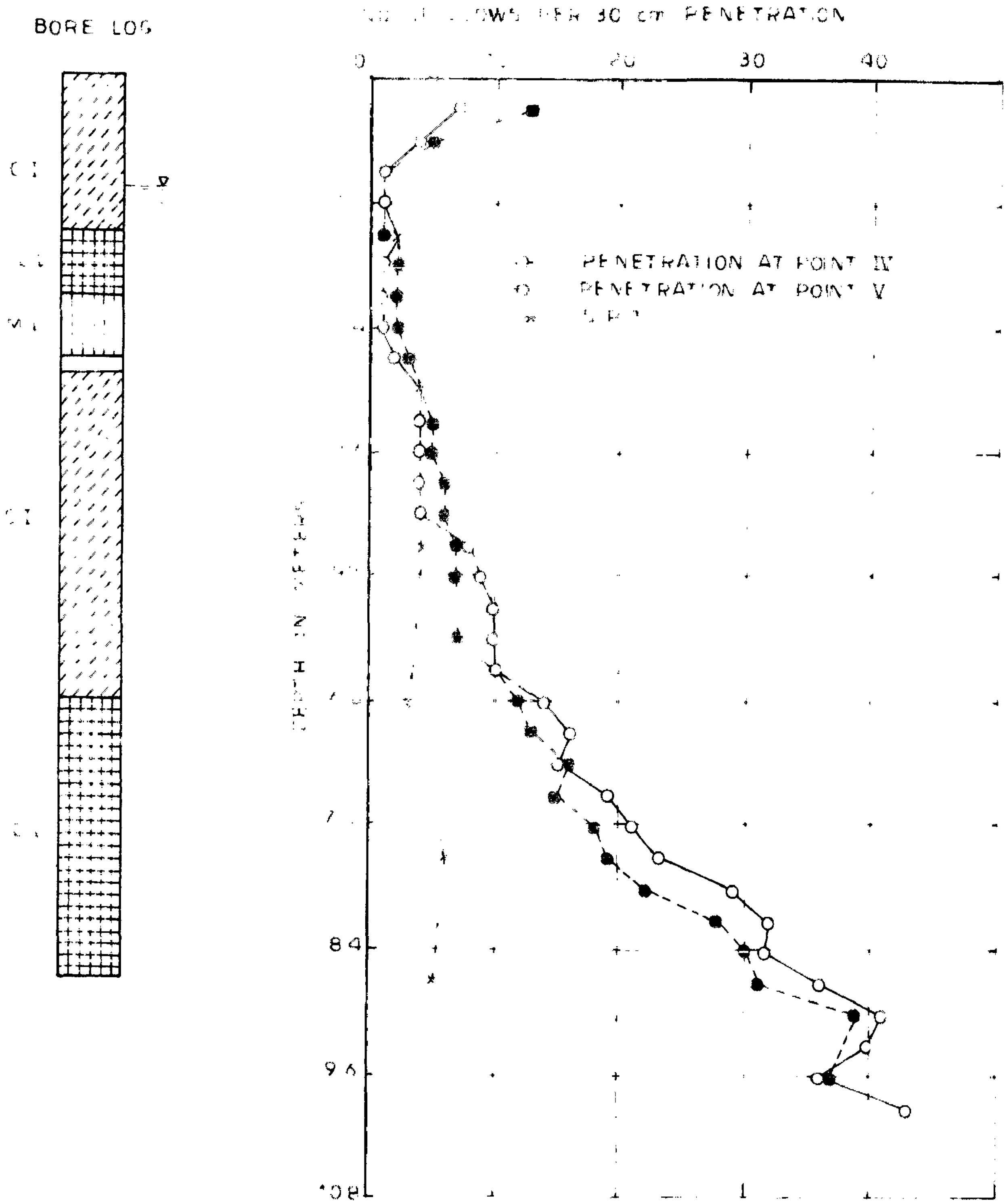


FIG 15. BORE LOG AND DYNAMIC PENETRATION RESISTANCE

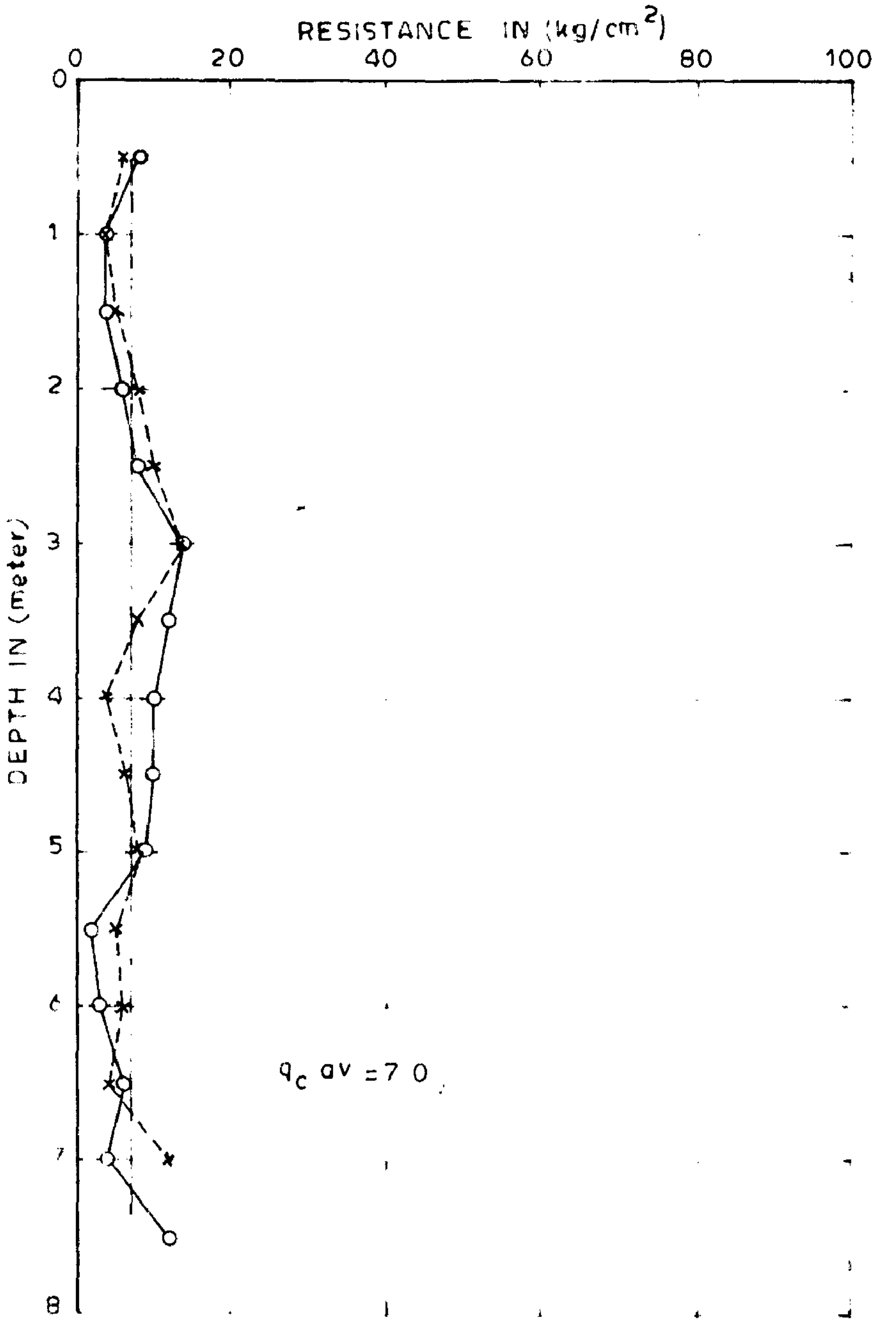
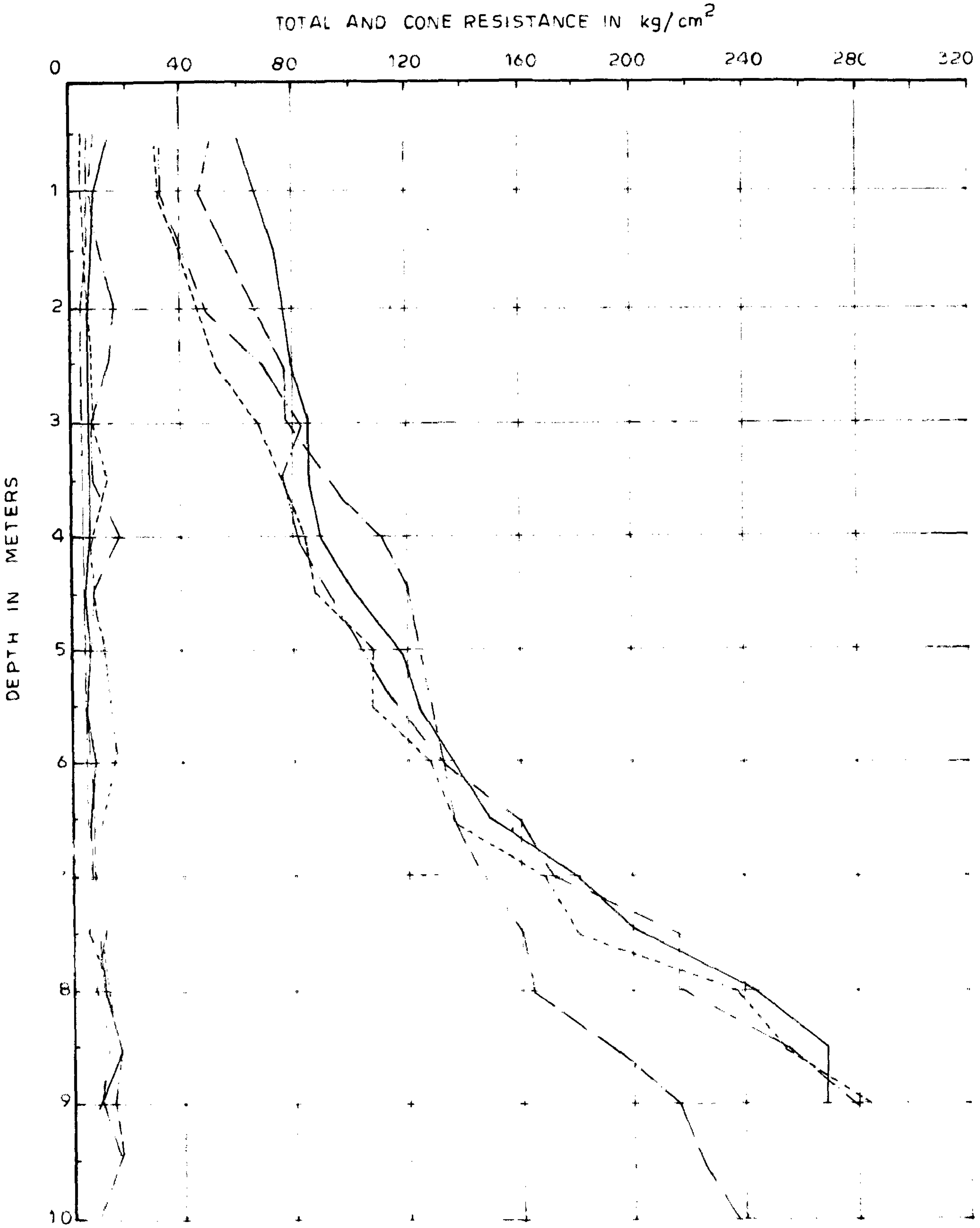


FIG. 1 _ STATIC PENETRATION TESTS



'IG.17- STATIC CONE PENETRATION TESTS (100 m AWAY FROM TEST SITE)

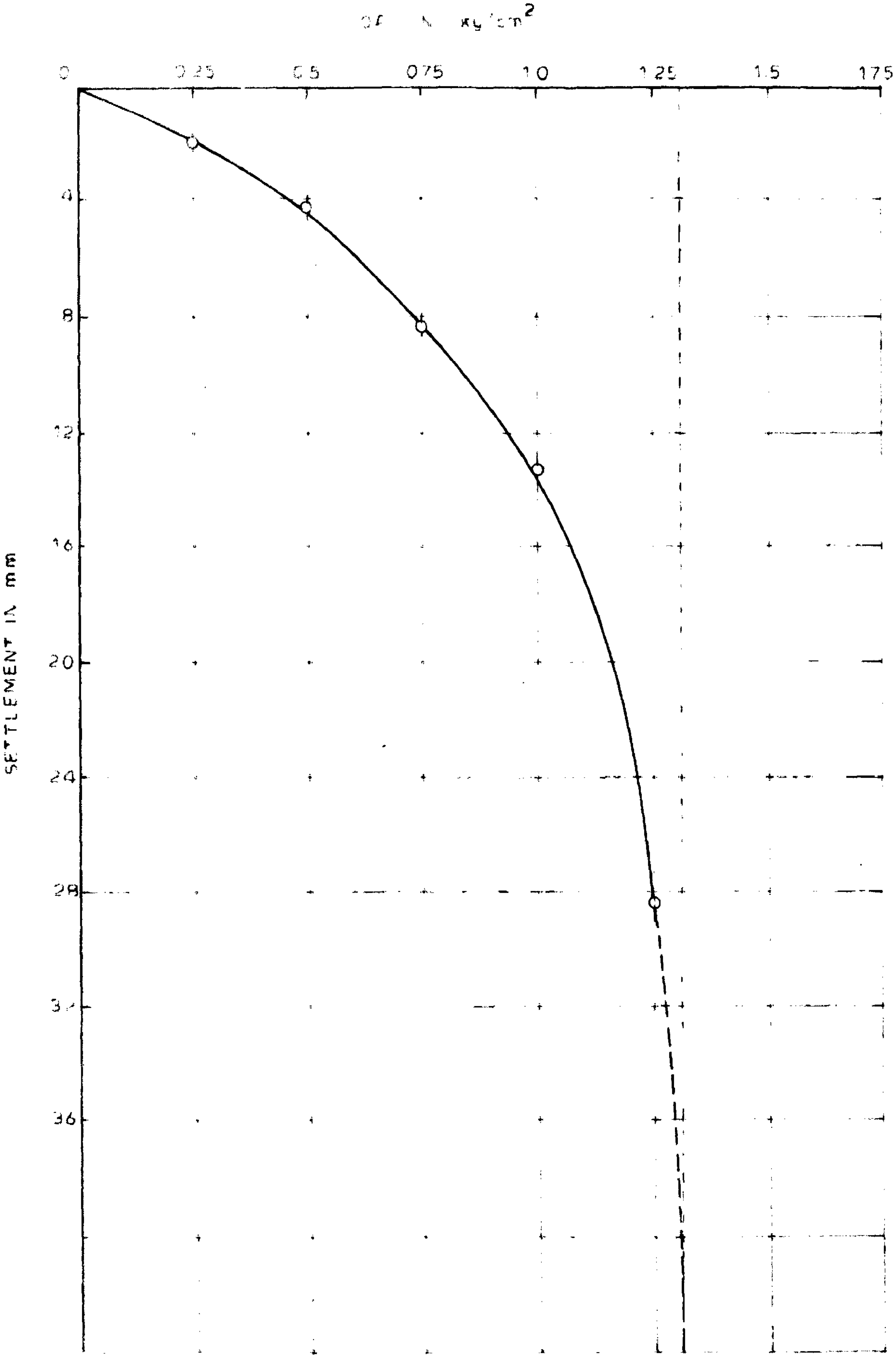


FIG18a. PLATE LOAD SETTLEMENT CURVE

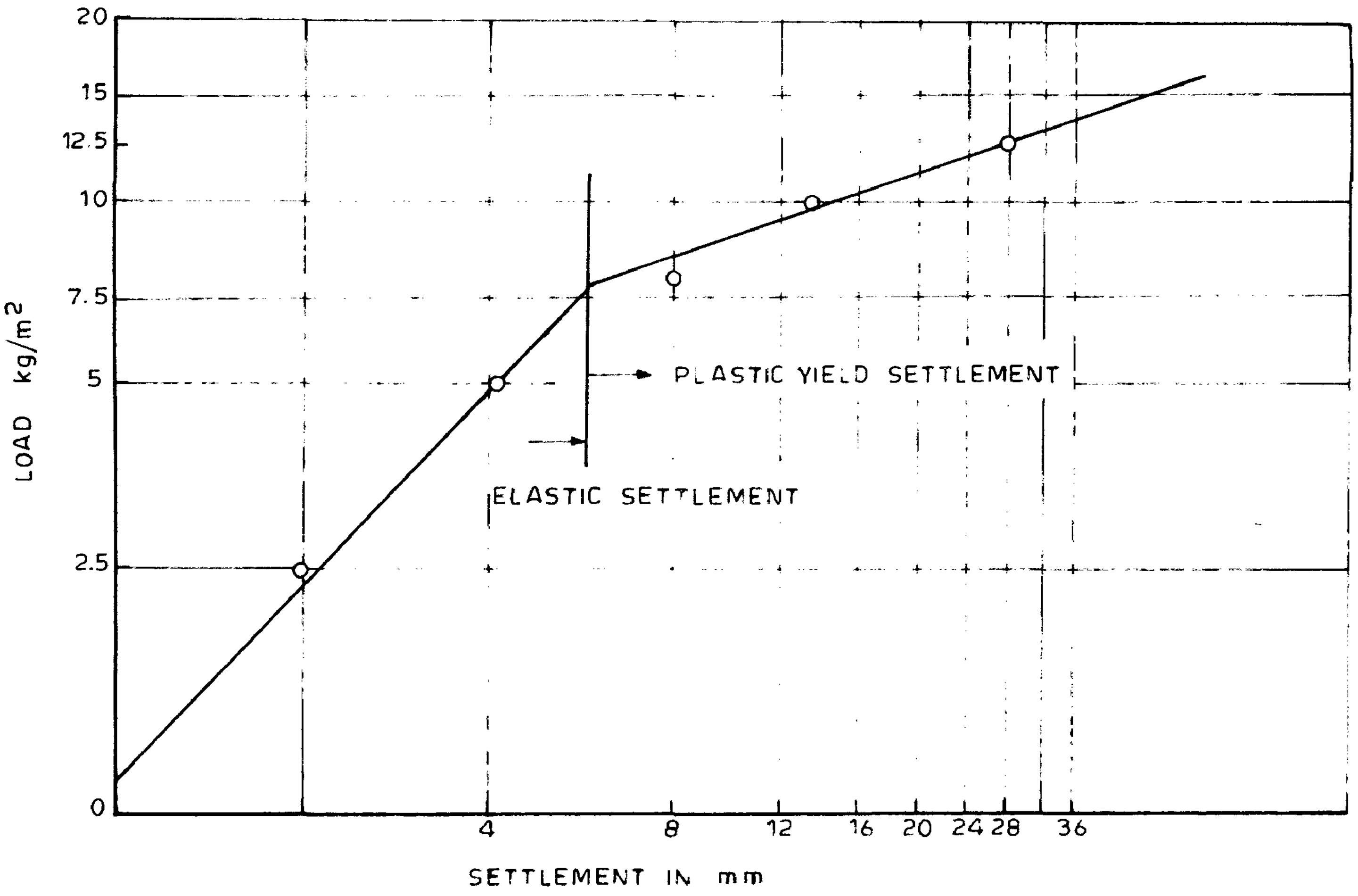


FIG.18b.PLATE LOAD SETTLEMENT CURVE

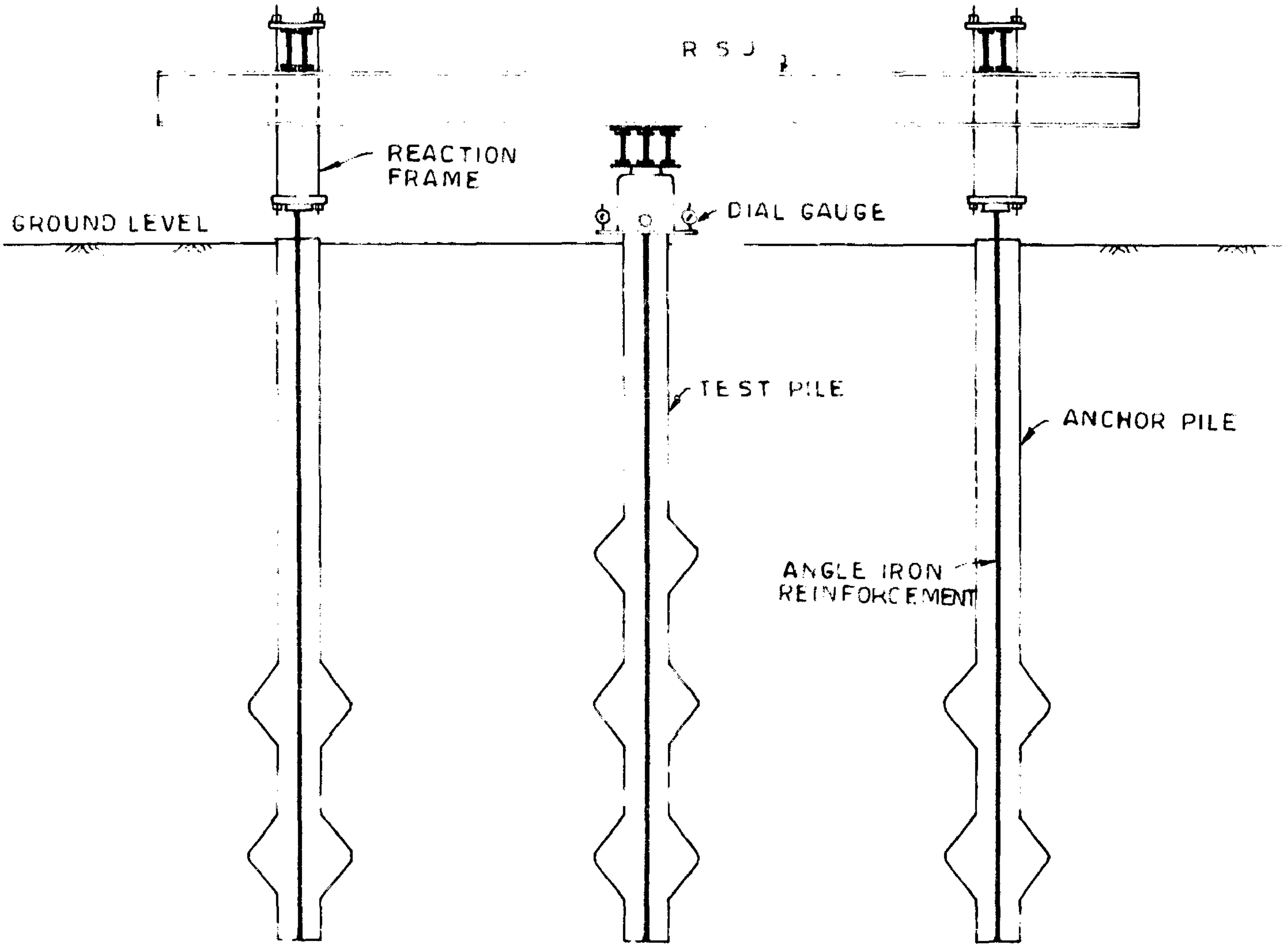


FIG. [illegible] _ TYPICAL PILE LOAD TEST SET-UP

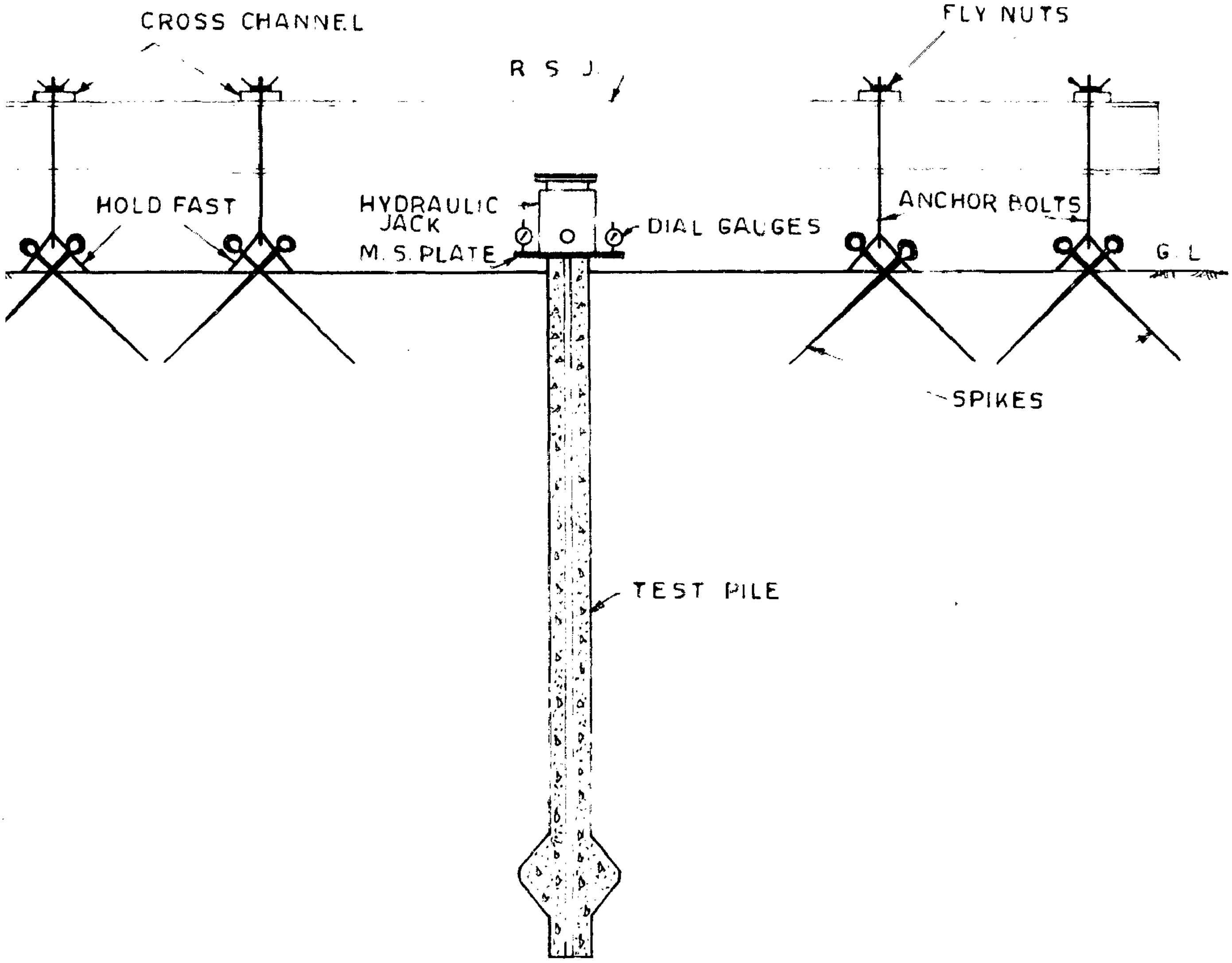


FIG. 2.6 _ TYPICAL LOAD TEST SET-UP WITH HOLD FAST TYPE ANCHORS

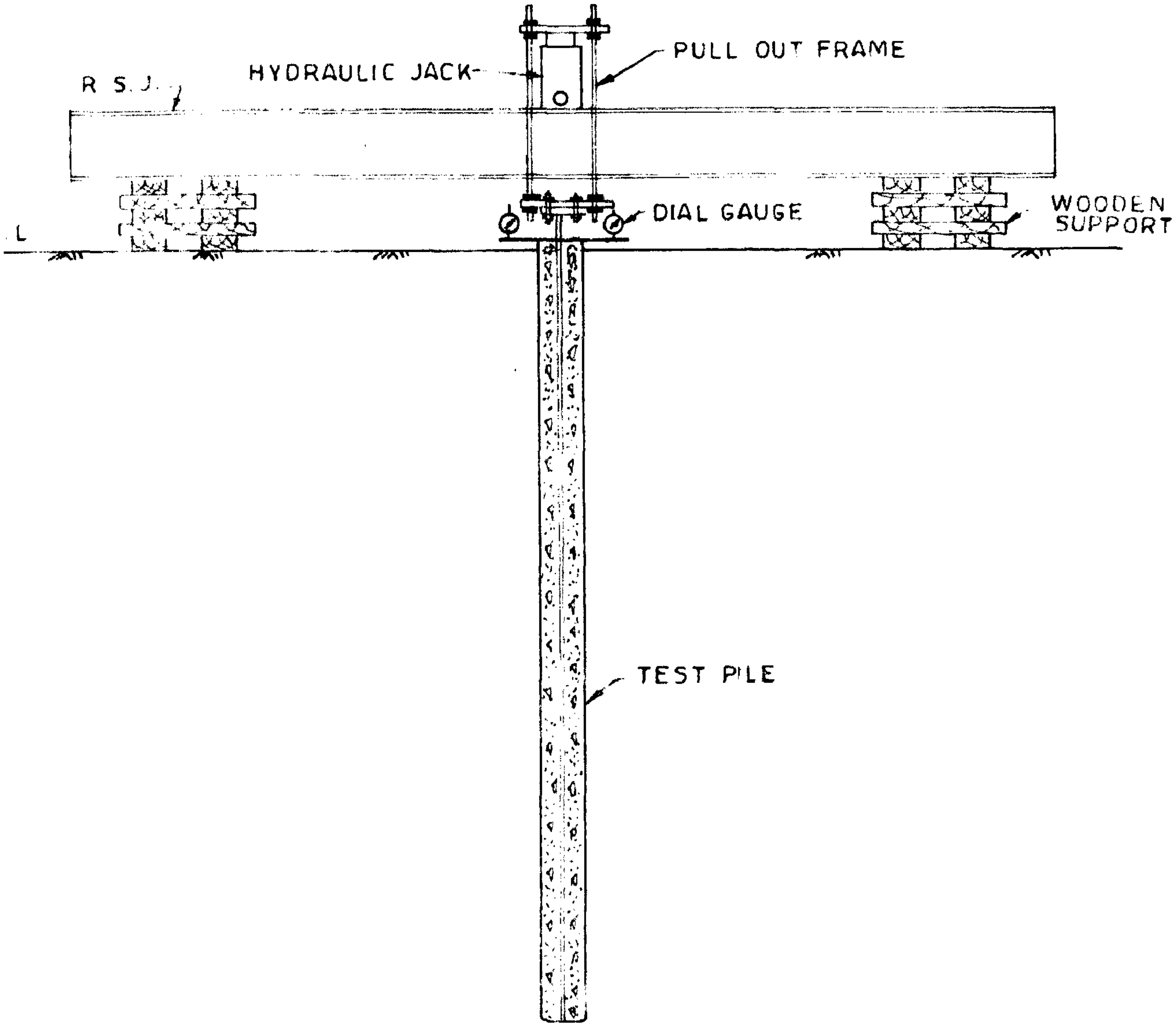


FIG. 21 _SET UP FOR PULL OUT TEST

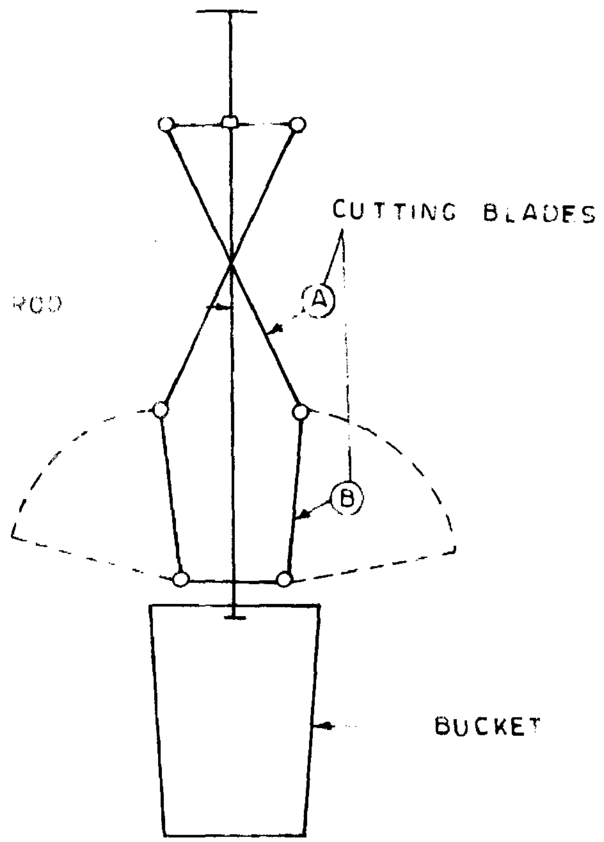


FIG. 22-UNDER-REAMING TOOL

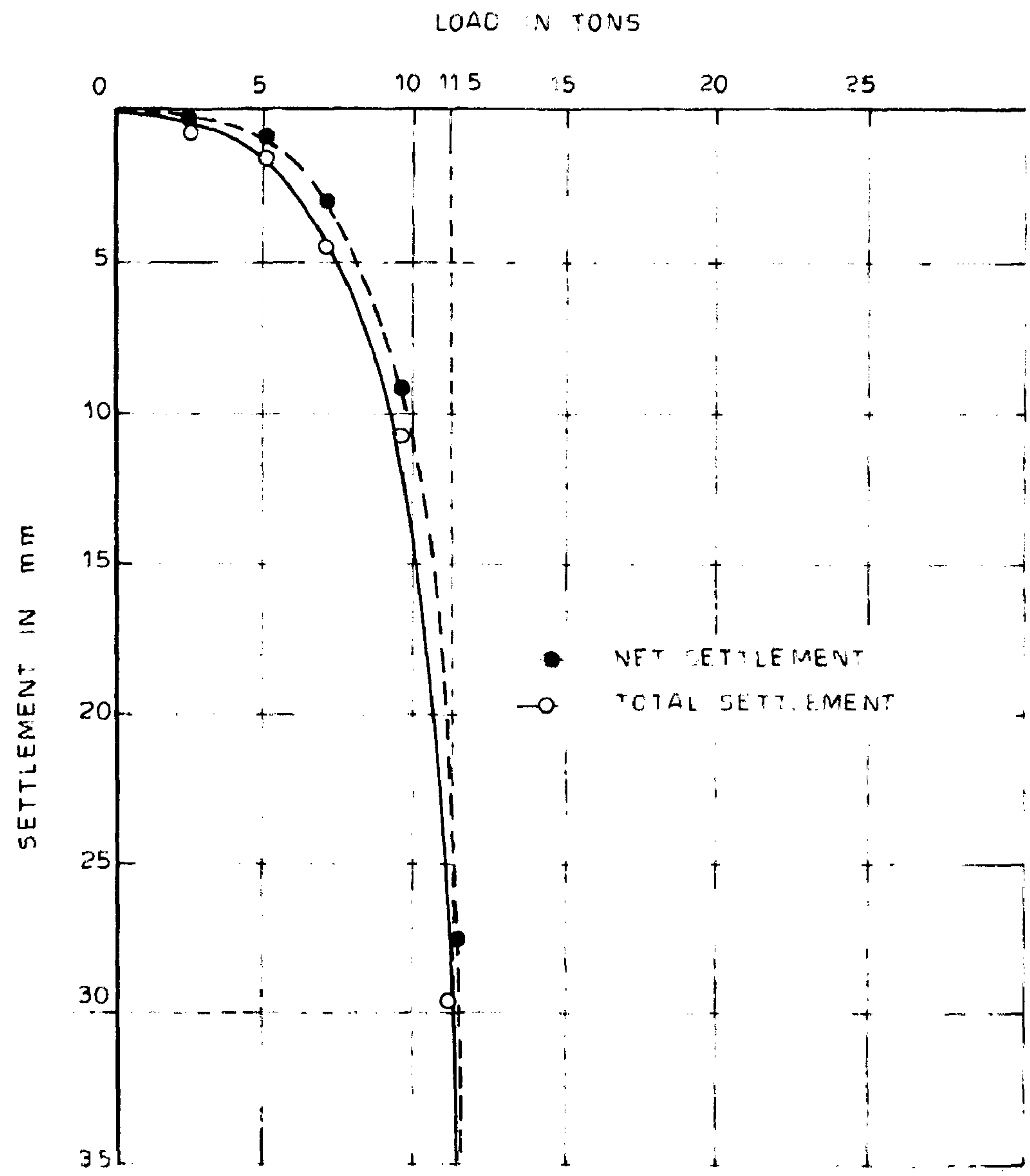


FIG. 23 - LOAD SETTLEMENT CURVES FOR UNIFORM DIAMETER PILE

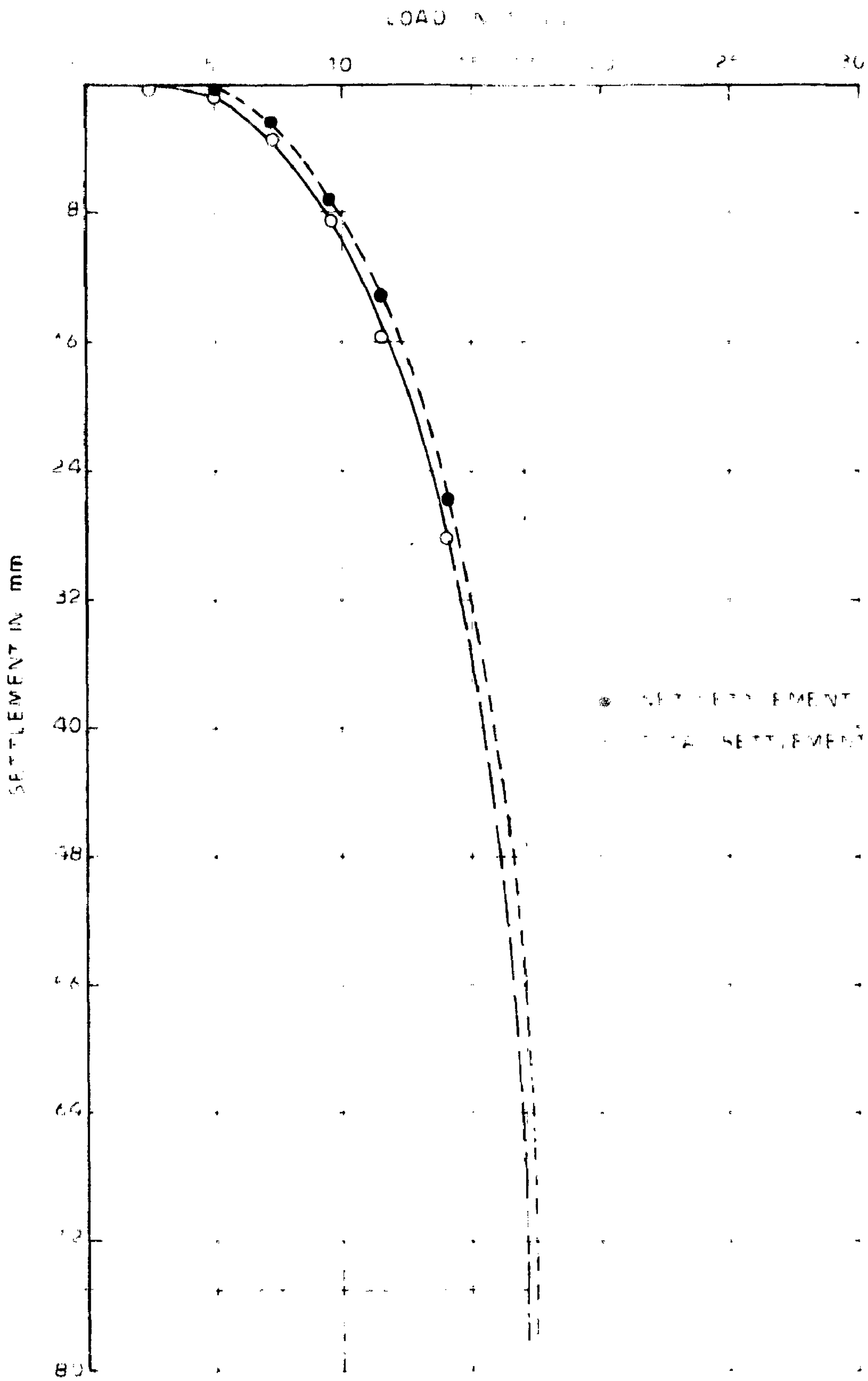


FIG. 24. LOAD SETTLEMENT CURVES FOR SINGLE UNDER REAMED PILE

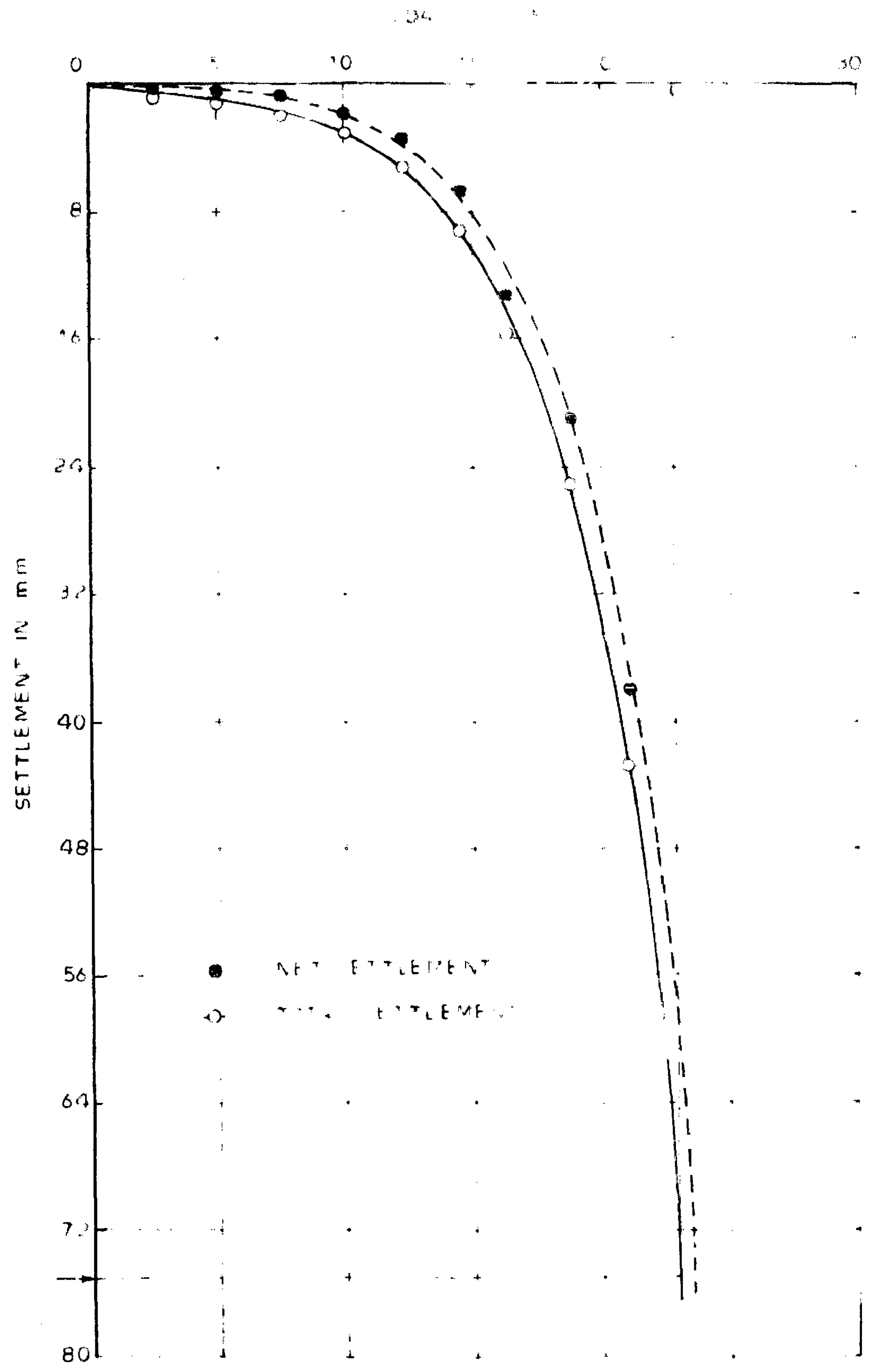
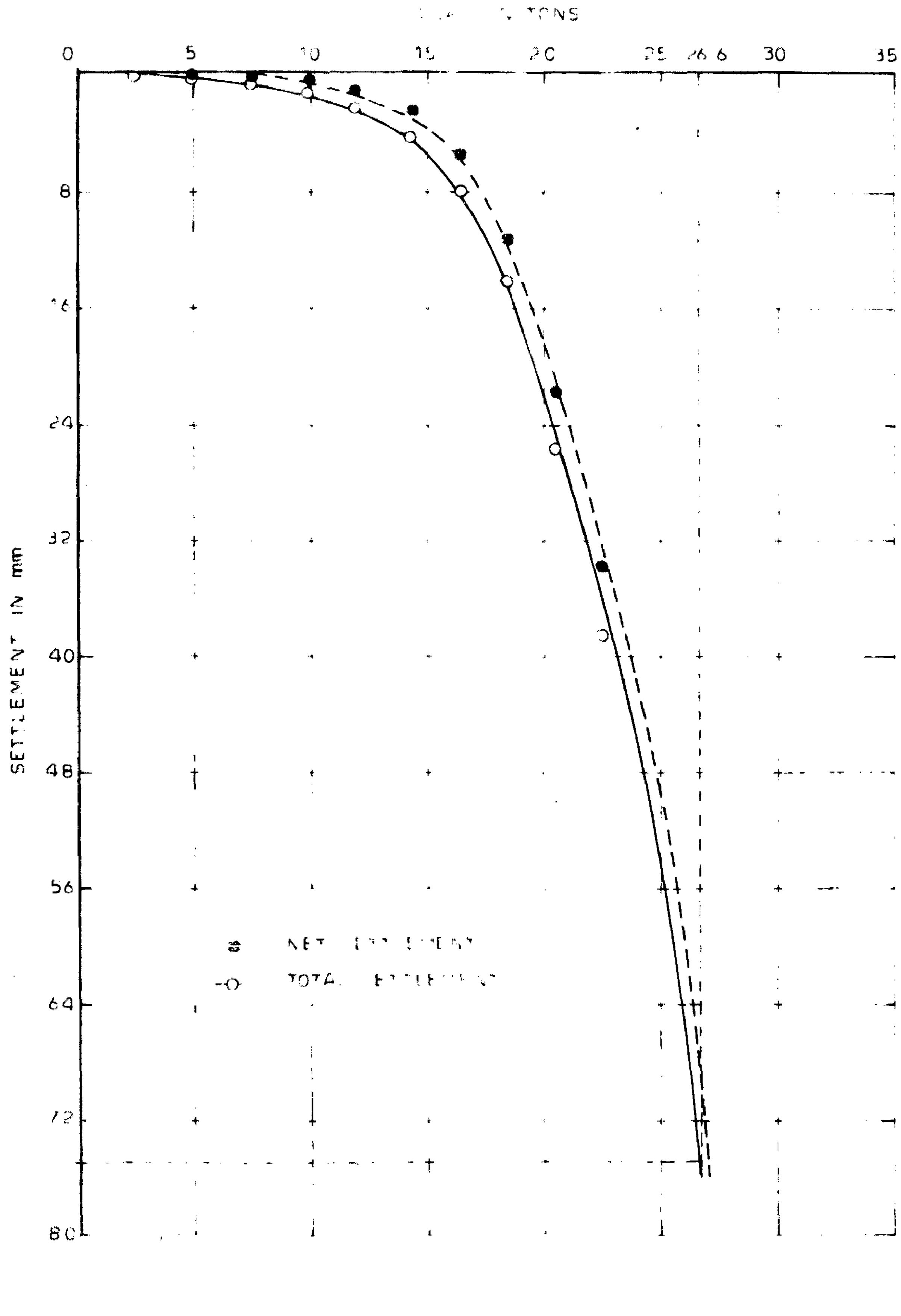


FIG 25 LOAD SETTLEMENT CURVES FOR DOUBLE UNDERREAMED PILES



IG 26. LOAD SETTLEMENT CURVES FOR TRIPLE UNDER REAMED PILE

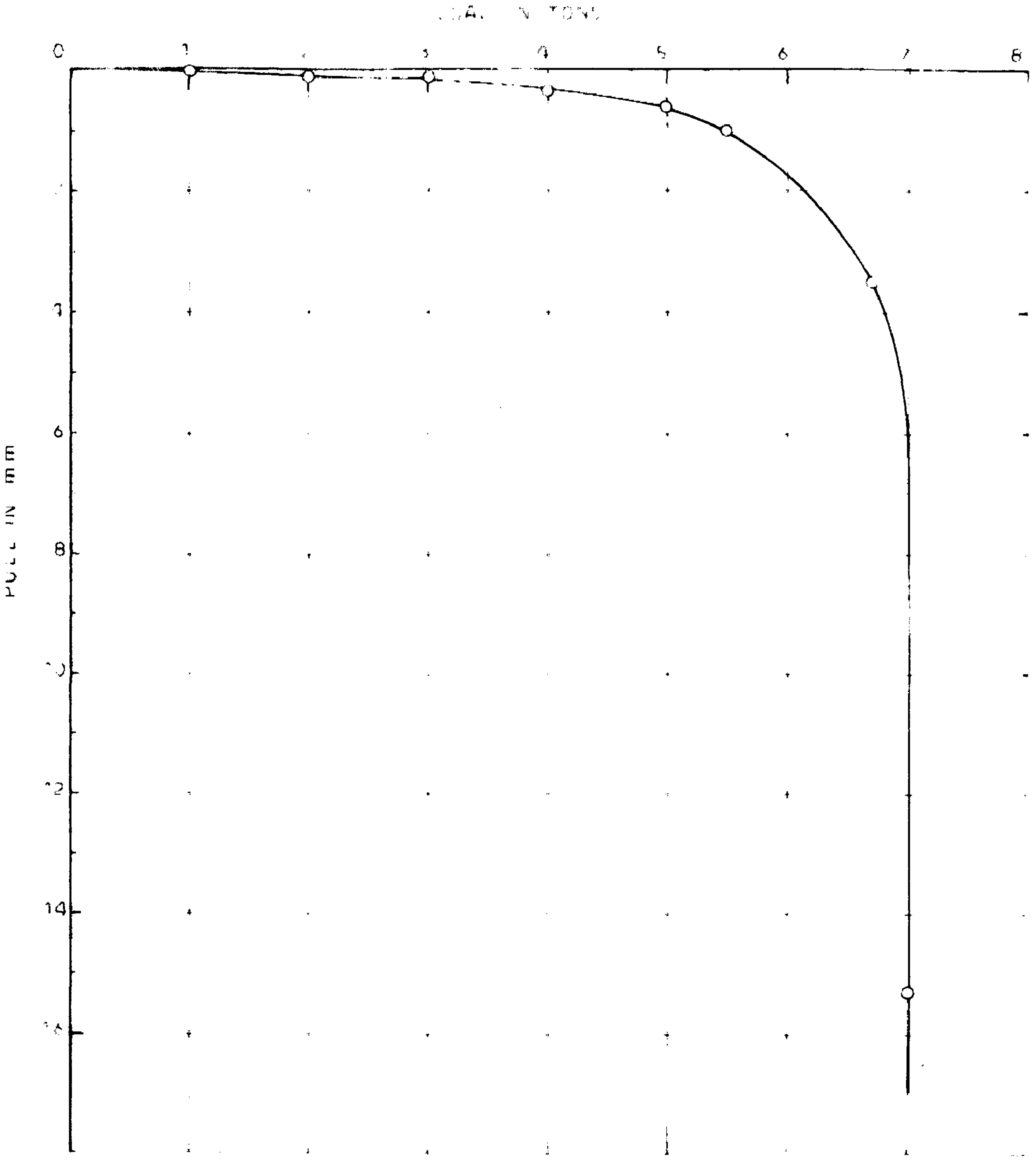


FIG 27. RESISTANCE OF UNIFORM DIAMETER PILE IN PULL OUT

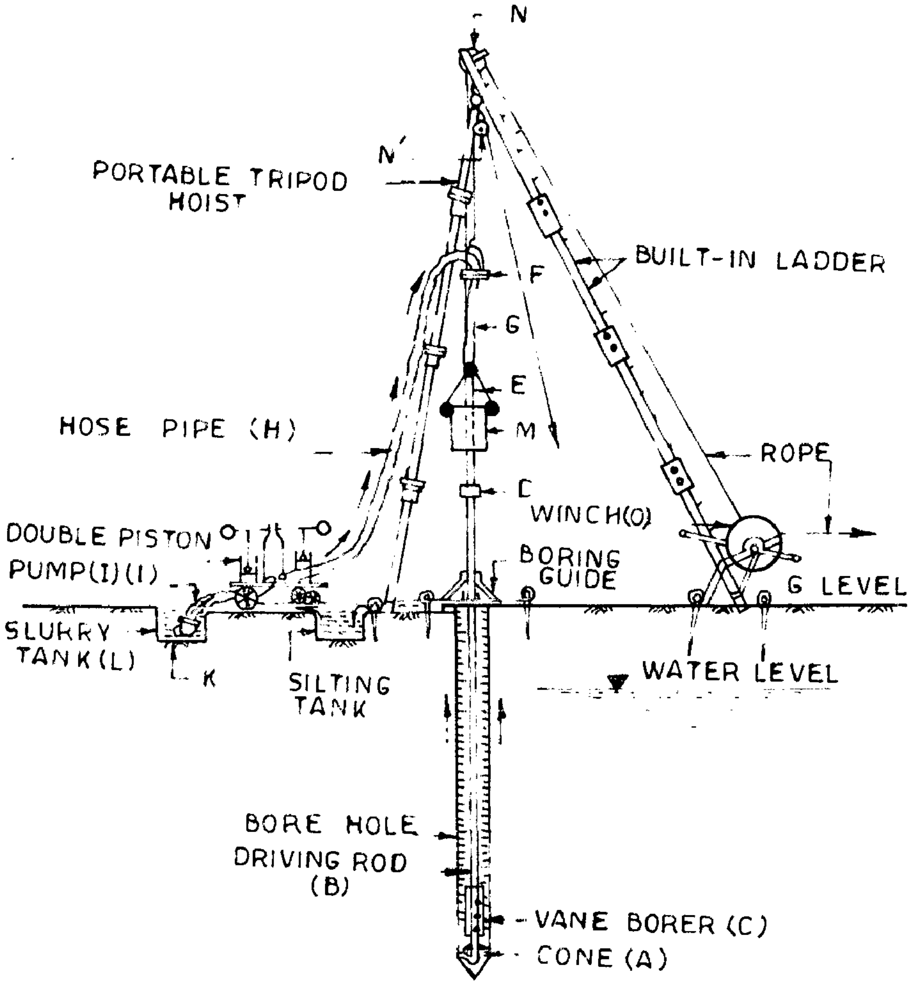


FIG. – DYNAMIC CONE PENETRATION ASSEMBLY

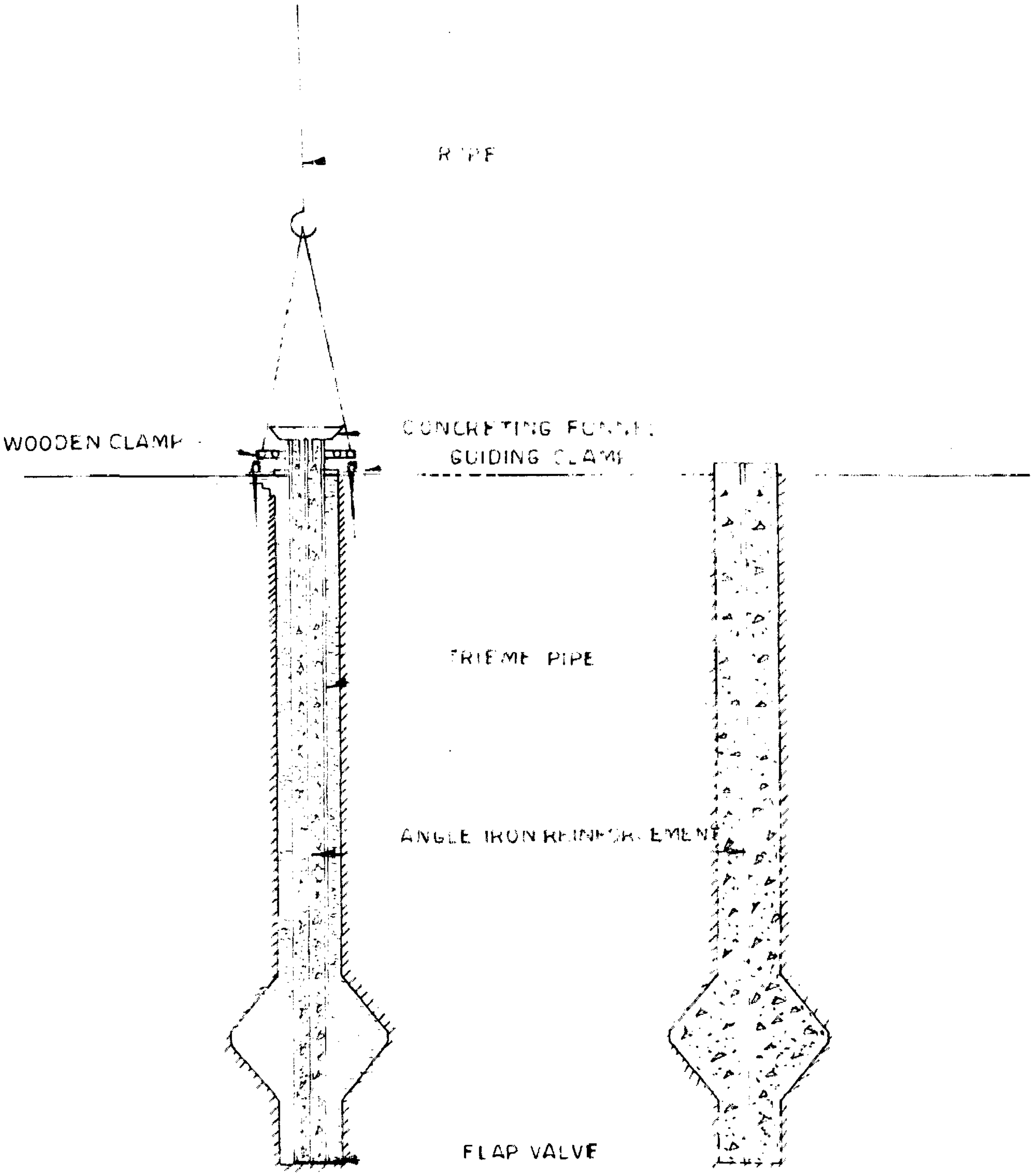
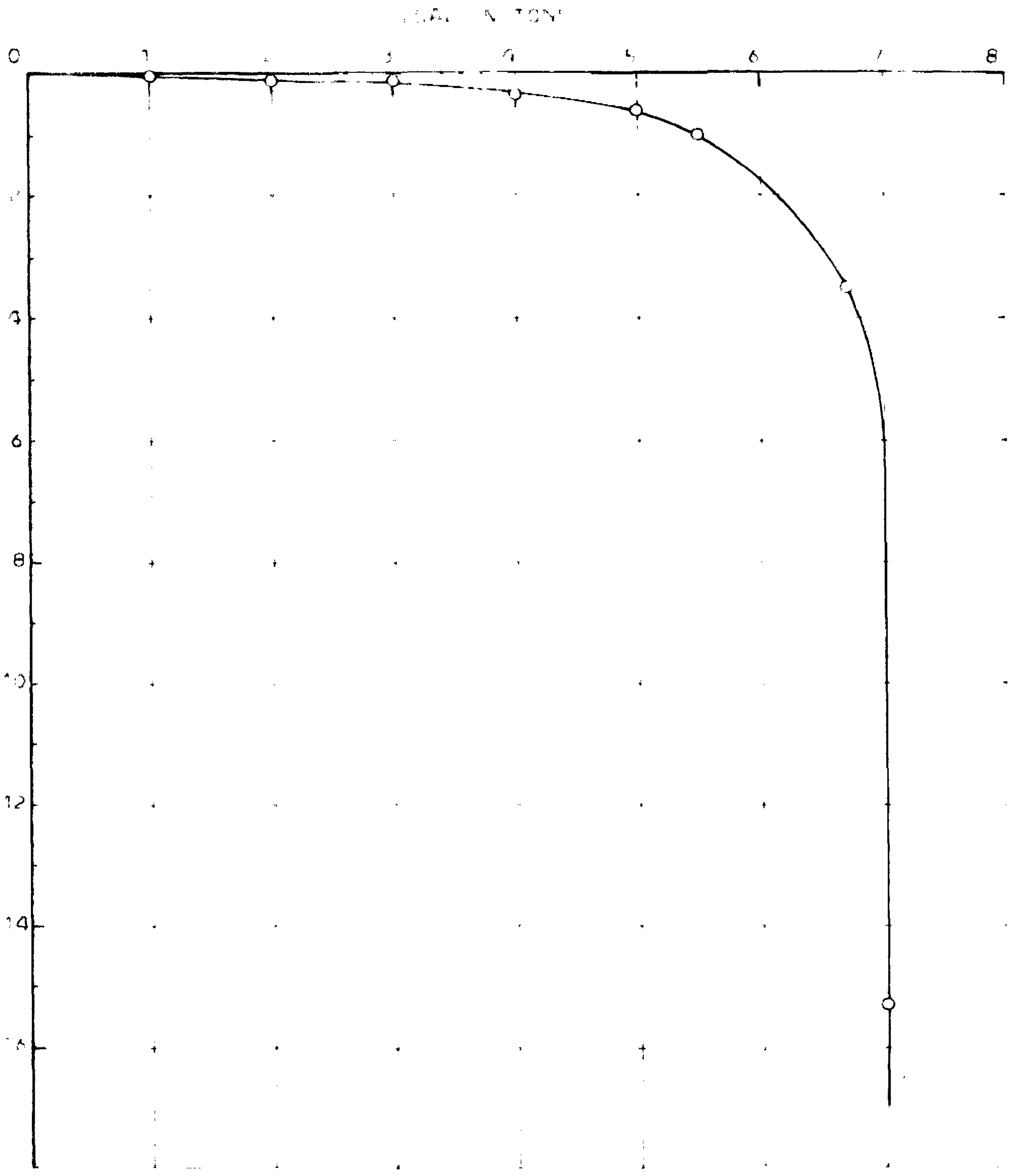


FIG. (CONCRETING PROCESS



IG 27. RESISTANCE OF UNIFORM DIAMETER PILE IN PULL OUT

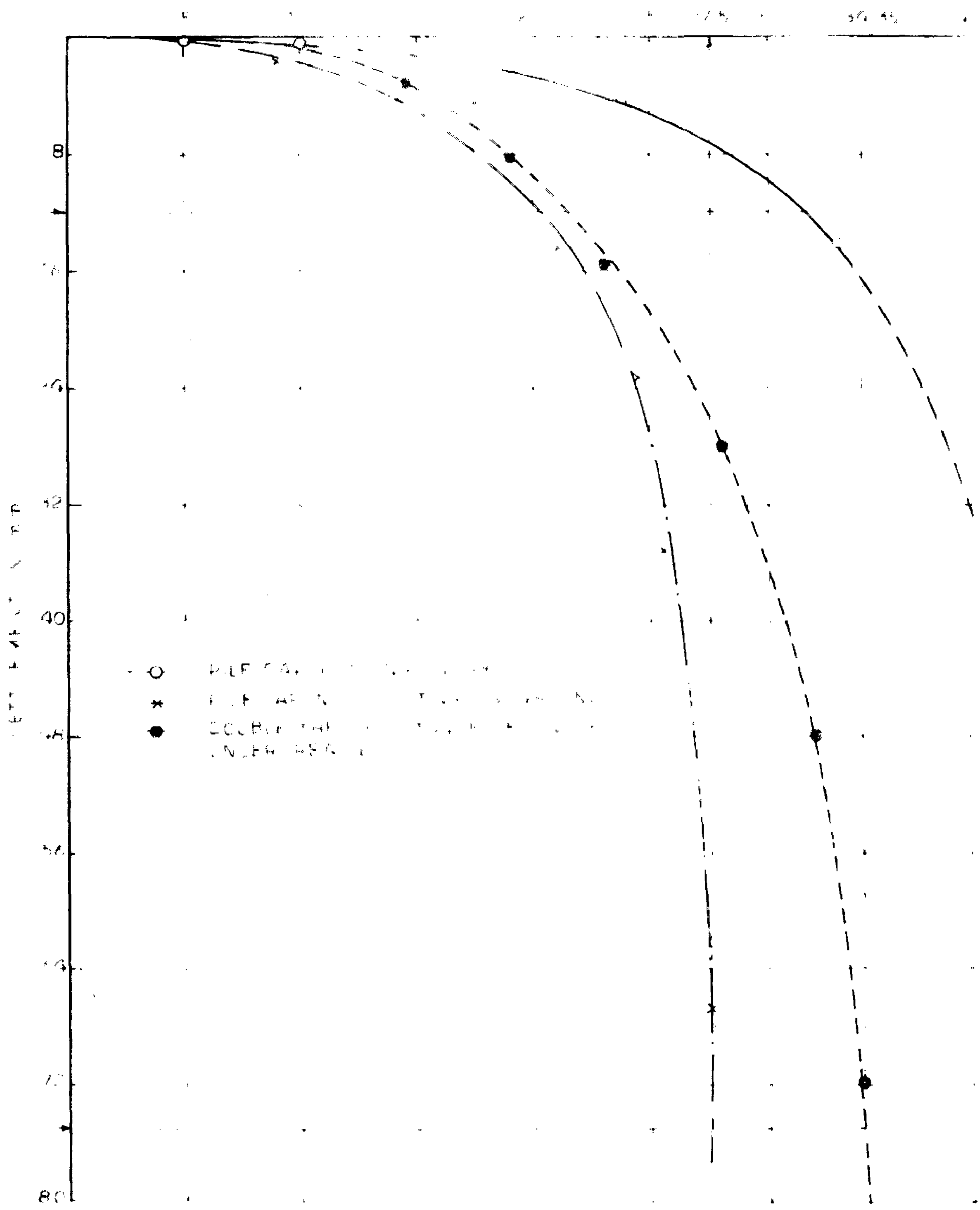


FIG 28 RES·STANCE OF THE GROUP OF TWO SINGLE UNDERREAMED PILES