CHAPTER II

LITERATURE REVIEW

1. The Bearing Capacity of Piles

The bearing capacity of piles is a important factor in the design and construction of foundations. The manual of engineering practice (No 27) defines the term bearing capacity as follows:-

"Bearing capacity may be defined as the load which can be sustained by a pile foundation without producing objectionable settlement or meterial movement - initial or progressive - resulting indamage to the structure or interfering with its use"

The bearing capacity of a pile depends upon

1) Type and properties of the soil
2) Surface and/or ground water requirement
3) Geometry of the pile (solid, hollow and shape)
4) Pile material (timber, concrete or steel)
5) Size of pile (cross section and length)
6) Property of mantle surface of pile (rough or smooth)
7) Methods of embedding the pile into the soil (driving, jacking, jetting, vibrating casting in place)
8) Position of pile (vertical or inclined)
9) Spacing of piles in a pile group

2. The Behaviour of Pile Under Load

The load-settlement relationship for a single pile in clay when subjected to vertical loading is shown in

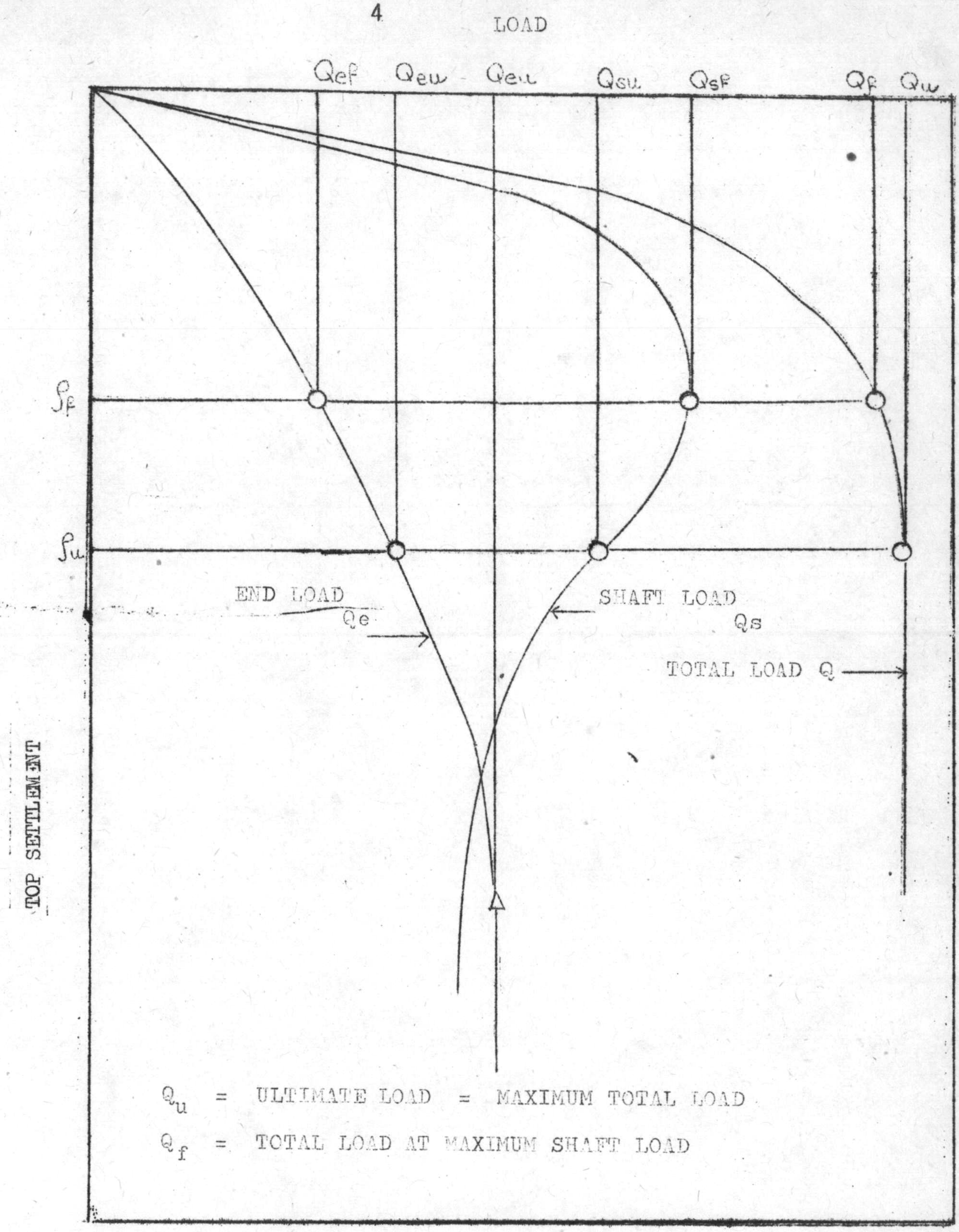


FIG I TYPICAL RELATIONSHIP BETWEEN TOTAL LOAD SHAFT LOAD AND END LOAD TO TOP SETTLEMENT

(FIG I). At the early stage the major part of the load capacity is based on the skin friction. As the load is increased the shaft load is rapidly increased until the mobilization of skin friction is reached. At this stage the shaft load is maximum ($Q_{sm}=Q_{sf}$) when loading is continued the shaft load is dropped while the end load is increased until the ultimate bearing of pile is reached. Therefore the ultimate bearing capacity of pile is the maximum sum of the two components of resistance. After pile failure the shaft load is rapidly decreased and the end load will reach its maximum value. It should be noted that the maximum value of each does not necessarily occur at the same vertical settlement of the pile. The verticle movement of pile to mobilize full end resistance is much greater than that required to mobilize full skin friction (PRACHIT 1968)

3. Methods of Determining Bearing Capacity of Piles

The bearing capacity of piles can be estimated in a number of ways which are all based on one of the following:-

1) Static formula this method, the sub-soil conditions must be known and bearing capacity can be estimated from the shear strength of the supporting soil.

2) Dynamic formula by applying the principle of energy and the bearing capacity can be obtained.

3) The dutch cone static penetration test, which directly reproduces the failure mechanism of a pile.

4) Direct load test on piles.

In this research only 1 static formula and 4 direct load test on piles will be performed and are discussed below.

3.1 Ultimate load from static formula

(a) General formula The static load Q supported by a pile can be thought of as being the sum of the frictional force on the pile shaft Q_s and the load carried by the pile toe Q_e thus.

$$Q = Q_e + Q_s \quad --- \quad 1$$

The load supported by the pile at failure "Q_u" therefore is given by

$$Q_{ult} = Q_{eu} + Q_{su} \quad --- \quad 2$$

where Q_{eu} = end bearing load at pile failure

Q_{su} = shaft load at pile failure

The maximum end bearing load of deep foundation can be expressed in the form

$$Q_{em} = A_e q_{em}$$

$$= A_e (cN_c + \gamma L N_q + 0.5\gamma D N_\gamma) \quad --- \quad 3$$

where A_e = cross-sectional area of pile toe

C = cohesion of soil at pile toe

γ = average unit weight of soil

D = diameter of pile, (width of pile)

L = depth of pile embeded in the soil

N_c, N_q, N_γ = bearing capacity factors for deep foundation

The maximum total shaft friction can be expressed as

$$Q_{sm} = C_a A_s \quad --- \quad 4$$

where C_a = shear strength between soil and pile

A_s = area of embedded pile shaft

(b) End resistance for cohesive soils

In purely cohesive saturated soils, the minimum ultimate load is reached when the pile is loaded under undrained conditions. This is the well known "$\phi = 0$" concept (SKEMPTON 1948) which gives $N_\gamma = 1$ and $N_q = 0$. The maximum end resistance of the pile becomes.

$$Q_{em} = A_e(CN_c + \gamma L) \quad --- 5$$

If the weight of the pile is assumed equal to the weight of soil displaced during installation the net maximum end load is

$$Q_{em} = A_e C N_c \quad --- 6$$

The value of N_c has been determined by many investigator both analytical and experimental method.

Meyerhof (1951) had analyzed the theoretical value and suggested that N_c is between 9.3 and 9.8 depending on the frictional resistance developed at the pile toe.

Skempton (1951) had found from full-scall experiments that $N_c = 9$ was sufficiently accurate for the calculation of the maximum end resistance of the calculation of the maximum end resistance for bored piles in London clay.

In Bangkok area (Chiruppapa) has suggested that the N_c value of soft clay is 6.5 Holmberg (1971) obtained $N_c = 10$ by loading the end of a 58 cm octagonal pile in the stiff clay. Other tests in the soft and stiff clays, however have indicated that at failure of a pile N_c could be much lower than this (BRAND 1970, MUKTABHANT & SUWANAKUL 1971)

(c) Shaft resistance for cohesive soils

The friction on the shaft of a pile in a cohesive soil is found to be directly related to the undrained shear strength of the soil by the relationship.

$$Q_{sm} = \alpha A_s S_u \quad \text{-----------------} \quad (7)$$

where α = adhesion factor

S_u = undisturbed undrained shear strength

The value of α varies from unity to 0.3, its value decreasing with increasing the undrained shear strength. For Bangkok clay this general trend was verified by HOLMBERG. (1971) for a number of different pile materials and soil strengths. (Fig 2) The adhesion factor of soft soil has been found to be about 1.0 (BRAND, 1970 MUKTABHANT & SUWANUKUL 1971)

The explanation of why the adhesion factor is less than 1.0 is simply that the shear strength between soil to the material of pile is less than soil to soil. For stiff clay the value of α is less than soft clay, it is believed that when pile is driving into stiff clay it will cause gap between soil and pile. Therefore if the shear strength of soil is high this hole remains open, and the soil does not flow back to contact the pile body resulting in lowering the adhesion between pile and soil. In the case of soft clay, the soil will flow to fill the space to give complete contact between the pile and soil with a consequent higher adhesion factor.

For bored piles the adhesion factor is less than the driven piles because the lateral pressure is dropped during the process of performing bored pile.

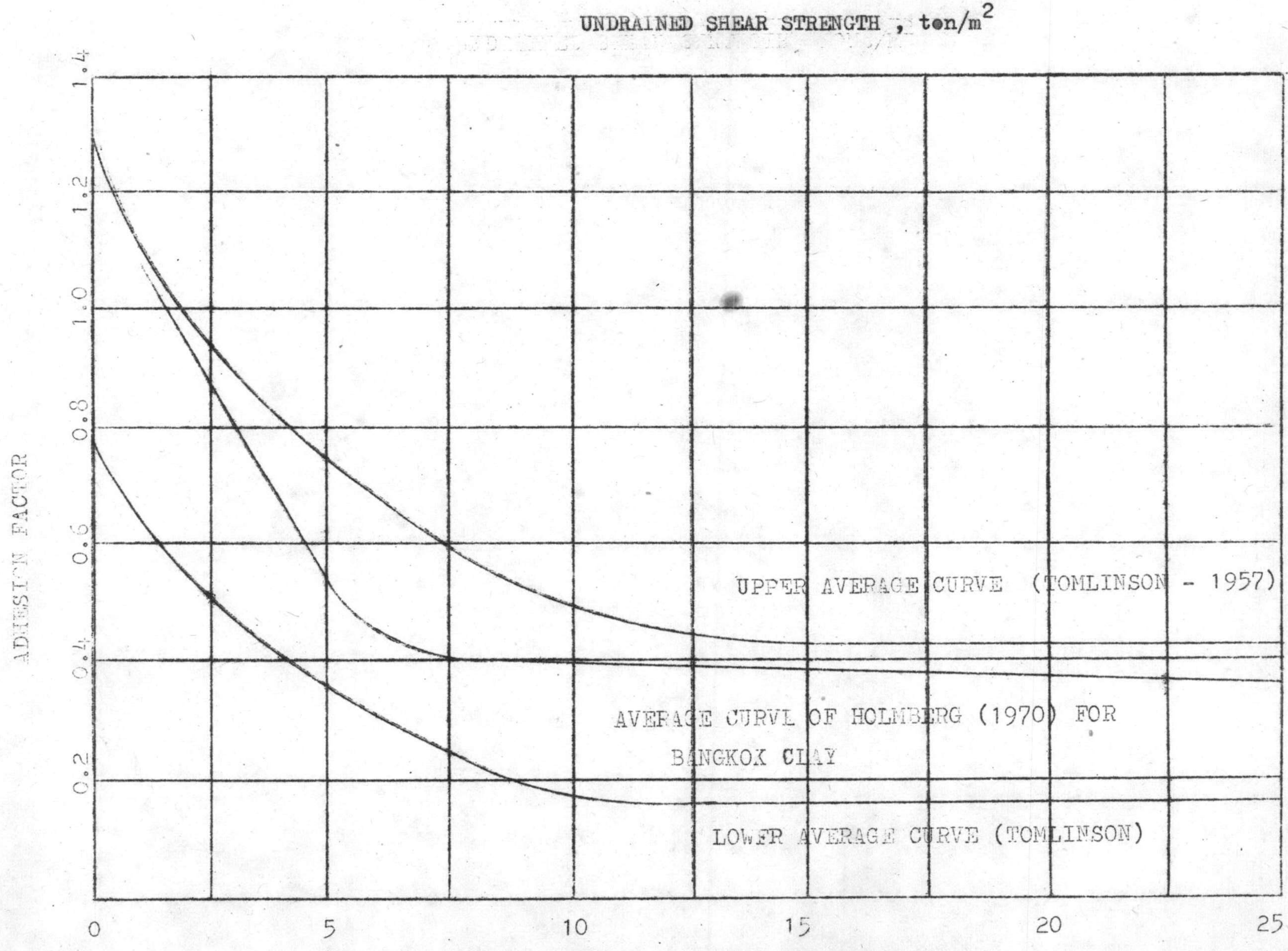


FIG 2 ADHESION FACTORS FOR CLAY

3.2 Ultimate load from pile load tests

The loading to failure of full-scale piles is certainly the most satisfactory basis for the estimation of the ultimate load carrying capacities of other piles installed in similar subsoil conditions. This method of test will give economical design because the working load of pile is maximum. It is regrettable that load tests are all too often tested to a maximum applied load of twice the design load.

PILE LOAD TESTS

4. Criteria of Failure:-

In order to measure, specify or discuss the ultimate load capacity of a pile it is necessary to establish what is to be understood by "failure" where a maximum load is reached which either drops or is sustained as the pile settlements increased. The definition of failure presents no problem as long as the settlement at which this state is reached is tolerable. For many piles, as with most spread footings, this ideal failure criterion cannot be applied (BRAND et al 1972) and it becomes necessary to define failure in terms of some rather arbitrary value of the pile settlement. It is impossible to establish one maximum permissible settlement for all piles under all circumstances, and the many existing criteria of failure based on allowable settlement have generally been established to take account of the worst combination of circumstances.

Terzaghi (1943) suggested that the criterion of failure for a single pile should be taken as a settlement of 0.1 D. This will lead to extremely large settlements however, for large diameter piles under their design loads. Such a criterion also has the disadvantage that it does not differentiate between elastic and plastic settlement,

For piles which are essentially friction piles, it would perhaps seem logical to define failure as the settlement at which the maximum shaft resistance is mobilized. This settelment is generally small compared to that required to mobilize end resistance (BURLAND et al 1966, WHITAKER & Cook 1966, BRAND, 1970)

HORACE E HOY, (Texas Highway Department 1970) recommended that unless failure actually occurs, it would appear resonable to define the point of "failure" by a maximum slope of the load-settlement curve. For example, the failure load could be defined as the load that results in a slope greater than 0.05 in per ton on the gross load-settlement or a slope greater than 0.03 in per ton on the plastic load-settlement curve whichever is smaller (Fig 3). This is still an arbitary definition of failure, but is a more generlized approach. The total criterion would include a maximum allowable gross settlement under the design load, consideration given to elastic shortening of the pile and to safety.

Where failure results from some arbitary criterion, the factor of safety could range from one and one-half to two. When actual failure of the pile soil system is determined by a plunging of the pile into the ground, this factor of safety could range from two to two and one-half.

It should be noted that observed settlements made at the top of the pile may not necessarily indicate downward movement of the pile into the ground. Where high load tests are performed the possibility of local failure of the pile above ground surface, or crushing of the grout under the test plate, should be reconiged as possible factors contributing toward observed "settlements".

The allowable settlement of the pile under the design load is given by many codes. Examples of these criteria for a number of representative codes is given in Table (1). It can be seen that a factor of safety of 2.0 on the working load is commonly specified for the definetion of allowable settlement. Some Codes base their criterion of acceptability on total settlement, some on plastic settlement, and some on a combination of the two.

A logical criterion of acceptability for a pile would take into account the type of pile (end bearing and/or friction in question. Because of the large discrepancy in the settlements at which the maximum end and shaft resistances of piles are mobilized, BURLAND et al (1966) proposed a design criterion established on the basis of separate consideration of end and shaft resistances. This approach will almost certainly lead to safer and more economical foundations in most circumstances as long as the mechanism by which the particular pile carries the applied load is well understood.

City or organization	Settlement criterion	Test load design load	Max per-missible settlement in/ton	Other settlement limits	Time settle-ment, limets	Safety factor	Load test req'd over tons
New York City	Net	200	0.01	1 in gross	0.001 in in 43 hr	2	30
Cleveland Ohio	Gross	200	0.01	1 in max	none in 24 hr	2	60
Chicago	Net	200	0.01	-	none in 16 hr	2	-
Washington D.C.	Net	200	0.01	1 in gross	none in 24 hr	2	40
Baston, Mass	Net	200	0.5 in (total)	-	—	-	-
AASHO	Net	200	0.25 in (total)	-	48 & 60 hr	-	-
Dep of Highways Ohio	Net	up to 300	0.25 in (total)	rate of settlement $<$ 0.003 in/ton	none in 6 hr	Variable	-

TABLE 1 SOME CODES AND SPECIFICATIONS FOR PILE LOAD TESTS

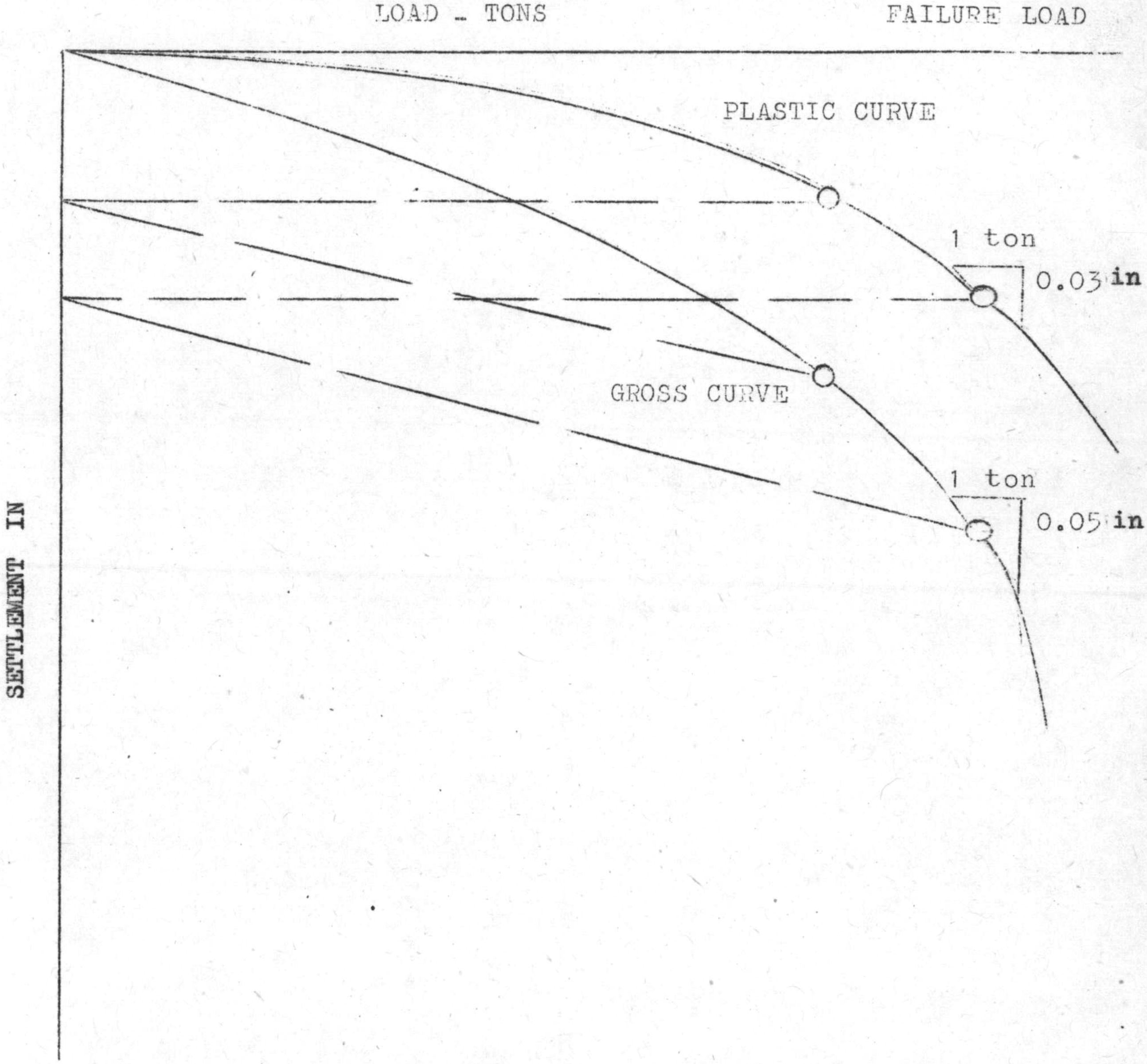


FIG 3 SLOPE CRITERIA FOR DETERMINING "FAILURE" LOAD FROM LOAD-SETTLEMENT CURVES

5. Types of Load Test:-

(a) Maintained load test

one type of load test employed for testing piles, the maintained load (ML) test is by far the most common. The procedure adopted is to apply static loads in increments of the anticipated working load. Increment of 0, 25, 50, 75, 100, 0, 100, 125, 150, 175 and 200% of the working load are often employed. Each load is maintained until the settlement has ceased or has diminished to an acceptable rate or until a certain time period. The working load and twice the working load are maintained on the pile for 24 hours or sometimes longer. If the load is increased to failure, this is done by reducing the increments where failure is imminent so that ultimate load capacity can be accurately measured.

(b) Constant rate of penetration test

This method was first suggested by WHITAKER (1963) In this test the pile is made to penetrate the soil at a constant speed from its position, the force applied at the top of pile to maintain the rate of penetration being continuously measured. The time to reach ultimate bearing capacity should be approximately the same as the time would be taken in making a "quick" shear test of the soil in the laboratory (in the unconfined compression test 0.0012 in/min) but the rate does not significantly affect the ultimate load (Whitaker 1963). In practice a duration of test of about 10 minutes was found suitable for piles in day. As the ultimate load capacity is approached a very little increase in load is required to maintain a constant rate of penetration, and the ultimate bearing capacity is reached when the continous vertical movements result in no increase in the penetration resistance.

Good agreement has been found to exist between the ultimate loads measured by the ML and CRP tests however has been voiced by ELLISON et al. (1971) on the grounds that it does not represent the type of loading to which a pile is subjected during its working life. They also reported that this test tended to overestimate the ultimate load capacity of bored piles in London clay.

It is obvious that the settlement recorded for a given applied load in the CRP test will always be lower than the comparative settlement for the ML test, because no time is permitted for plastic settlement under sustained load this is a disadvantage of the CRP test. Otherwise, the CRP test has the great advantage that it can be carried out very quickly. As long as sufficient experience is gathered with the CRP test in the prevailing subsoil condition and if initial correlation are made the CRP test is generally to be preferred to the ML test.

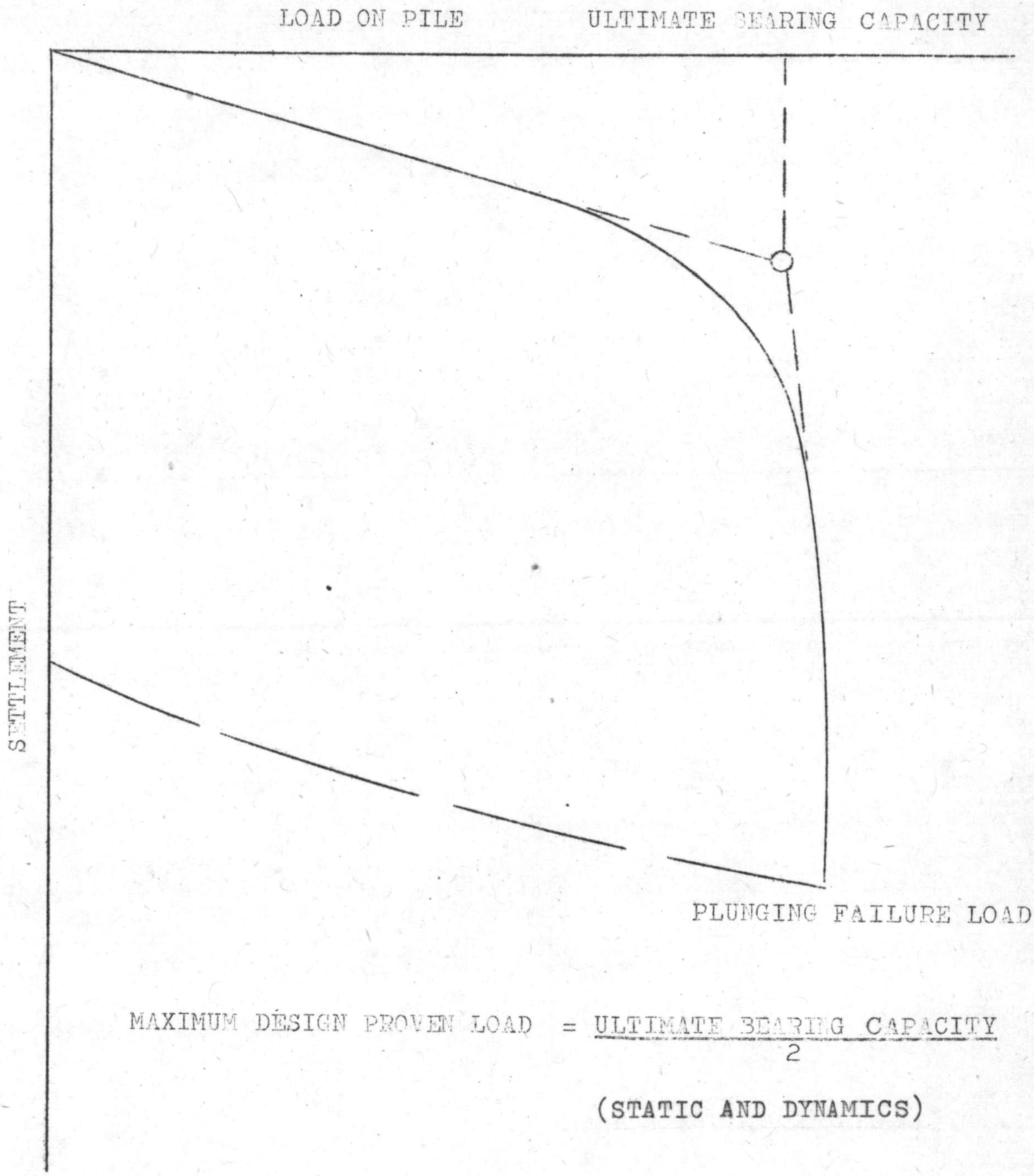
(c) Quick test:-

The CRP test calls for records of time and jacking force to be made at equal interval of movements of the pile head with the rate of jacking being adjusted so that reading occur at equal intervals of time. For convenience and simplicity, the CRP test was modified by the Texas Highway Department to produce the quick test method.

Essentially, it requires that loads be added in increments of 5 or 10 tons with gross settlement readings, loads and other data recorded immediately before and after the application of each increment of load. Each increment is held for $2\frac{1}{2}$ minutes, and the next increment is then applied.

When the load-settlement curve obtained from this test data (Fig 4) shows that the pile is definitely being failed (i. e. the load on the pile can be held only by constant pumping of the hydraulic jack and the pile is being driven into the ground) pumping is stopped. Gross settlement readings, loads and other data are recorded immediately after pumping has ceased and again after intervals of $2\frac{1}{2}$ minutes and 5 minutes. The load on the pile in the case of constant pumping is called plunging failure load. Then all load is removed, and the pile is allowed to recover. Net settlement readings are made immediately after all load has been removed and at intervals of $2\frac{1}{2}$ minutes for a total period of 5 minutes.

All test loads are carried to plunging failure or to the capacity of the equipment. The maximum proven design load is considered to be 50 percent of the ultimate bearing capacity, which is indicated by the intersection of lines drawn tangent to the 2 basic portions of the load-settlement curve as shown in Fig (4).



MAXIMUM DESIGN PROVEN LOAD $= \frac{\text{ULTIMATE BEARING CAPACITY}}{2}$

(STATIC AND DYNAMICS)

FIG 4 INTERPRETATION OF RESULTS (TEXAS HIGHWAY DEPARTMENT).
QUICK TEST METHOD

6. Separation of End And Shaft Loads

To enable a full understanding of pile behavior it is essential to know the division of total load Q sustained by a pile into end load Qe and shaft resistance Qs this will enable realistic selection of the design parameters adhesion factor and bearing capacity factors Nc, Nq for a given soil condition and pile type. A variety of methods have been used for ascertaining Qe and Qs from direct measurement to methods based solely on the load-settlement curves for the pile

6.1 Direct measurement Probably the most satisfactory way of determining the end and shaft load on a pile at a particular total load is by direct measurement. This can be accomplished by means of a load cell placed in the pile toe or by devizing an arrangement which endbles the end and shaft to be loaded separately

In the Bangkok area a load cell was used by Muktabhant & Suwanakul (1971) in their tests on driven castin-situ piles while HOLMBERG (1970) loaded separately the end and shaft of a driven octagonal pile Direct determination of Qe and Qs has the disadvantage that it tends to be rather an expensive procedure

6.2 Indirect measurement indirect measurement of load distribution along the length of a pile can be achieved by means of strain gauges of some kind they may take the form of electrical resistance gauges, either affixed to the surface of the pile or cast into the pile if concrete or mechanical gauges of steel rods or tensioned wires fixed at various depths in the pile strain readings are taken at the top of the pile when loads are applied

Gauges of the electrical resistance type have been used in larged bored piles in Bangkok by PROMBOON et al (1972) with little success, steel rod of the type suggested by HANNA (1970) (telltale) have not previously been used locally but thin wires were used to determine the load distribution along the 45 m bored piles supporting the piers of the Tha Chang Bridge (PROMBOON et al, 1972)

Strain gauging of driven and cast-in-situ piles has been used to show that a load applied to a pile head is transferred gradually from the pile shaft to the toe, the loads supported by each depending on the soil conditions and the pile type (HANNA, 1970 VESIC 1970, COLOMBO 1971 PROMBOON et al, 1972)

a) <u>The settlement and the elastic shortening of pile</u>

The settlement of pile top ρ_{top} involves

1. The elastic shortening of pile Δ_p
2. The elastic compression of subgrade Δ_s
3. The plastic settlement of pile ρ_d

$$\rho_{top} = \Delta_p + \Delta_s + \rho_d$$

The toe settlement of pile is the sum of the elastic compression of subgrade and the plastic settlement of pile

$$\rho_{toe} = \Delta_s + \rho_d$$

$$\rho_{top} = \Delta_p + \rho_{toe}$$

The elastic compression of pile Δ_p consists of two components

(1) The elastic compression of pile due to end load Δ_{pE}

(2) The elastic compression of pile due to skin friction Δ_{ps}.

$$\Delta_p = \Delta_{pE} + \Delta_{ps}$$

The top settlement ρ_{top} of top can be measured directly from pile top. At any substained load if the load is rebound, the plastic settlement is remained and the elastic rebound (Δ_r) is equal to the top settlement minus the plastic settlement and can be expressed as

$$\Delta_r = \rho_{top} - \rho_d$$

$$\Delta_r = \Delta_p + \Delta_s$$

Finally the top settlement can be expressed in the form

$$\rho_{top} = \Delta_p + \rho_{toe} = \Delta_p + \Delta_s + \rho_d = \Delta_r + \rho_d \quad \text{— — — 8}$$

The elastic shortening of pile can be measured directly if the toe settlement is known.

Other methods to find the elastic shortening of pile is to assume the load distribution of skin resistance

b) <u>The relationship of the mode shape of frictional resistance and the influence factor of pile</u>

The elastic compression of the pile can be divided to two parts (1) compression due to skin friction Δ_{ps} (2) compression due to end bearing Δ_{pE}

$$\Delta_{pE} = \frac{Q_e L}{AE_p} \qquad \text{--- --- -- } 9$$

E_p is the modulus elasticity of pile

To evalulate the elastic compression of the pile due to frictional resistance, the mode shape of frictional resistance must be known.

Let the λ be the influence factor of the shaft load. The elastic shortening due to friction will be equivalent equal to the shortening that caused by the load λQ_s in axial

$$\Delta_{ps} = \frac{\lambda Q_s L}{AE_p} \qquad \text{--- --- -- } 10$$

From fig 5 the total frictional load at any distance y

$$Q - Q_y = \int_0^y f(y)\,dy$$

Therefore $$Q_s = \int_0^L f(y)\,dy$$

the frictional elastic shortening of pile of slice dy

$$d\Delta_{ps} = \frac{\left\{Q_s - \int_0^y f(y)\,dy - f(y)\,dy\right\}(dy)}{AE_p}$$

neclect the load f(y)dy

$$\Delta_{ps} = \int_0^L \frac{Q_s\,dy}{AE_p} - \frac{\int_0^L \int_0^y f(y)\,dy\,dy}{AE_p}$$

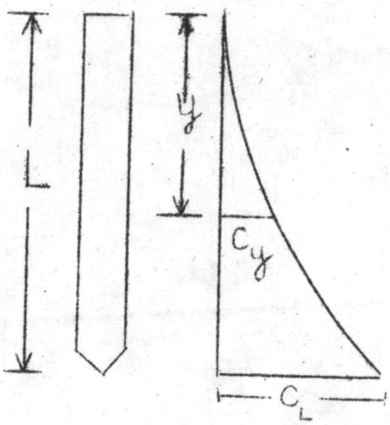
$$\Delta_{ps} = \frac{Q_s L}{AE_p} - \frac{\int_0^L \int_0^y f(y)\,dy\,dy}{AE_p}$$

substituting Δ_{ps} for eq (10)

$$\frac{\lambda Q_s L}{AE_p} = \frac{Q_s L}{AE_p} - \frac{\int_0^L \int_0^y f(y)\,dy\,dy}{AE_p}$$

$$\lambda = 1 - \frac{\int_0^L \int_0^y f(y)\,dy\,dy}{Q_s L} \quad \text{---} \; 11$$

Example :- Let the frictional resistance mode shape in the form of parabolic function



$$f(y) = C_y = \left(\frac{y^2}{L^2}\right) C_L$$

$$Q_s = \int_0^L \left(\frac{y^2}{L^2}\right) C_L\,dy = \frac{1}{3} C_L L$$

$$\int_0^L \int_0^y f(y)\,dy\,dy = \frac{1}{12} C_L L^2$$

$$\lambda = 1 - \left(\frac{C_L L^2}{12}\right)\left(\frac{3}{C_L L^2}\right)$$

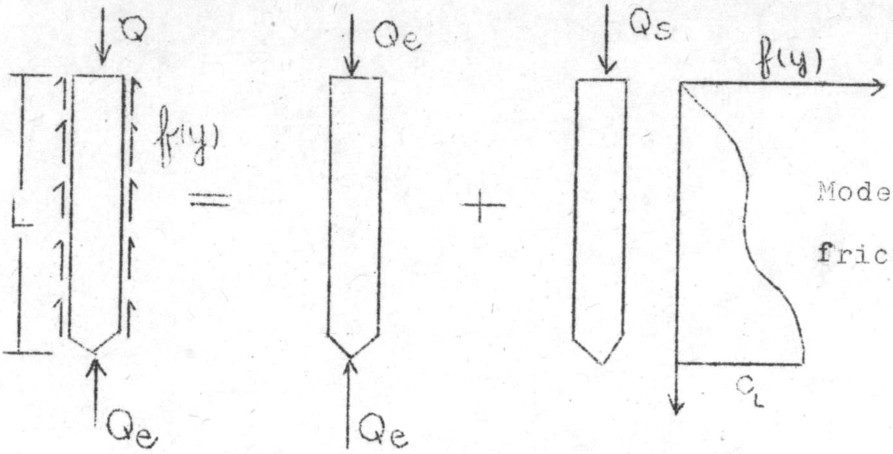
$$\lambda = \frac{3}{4}$$

c) The load distribution of pile

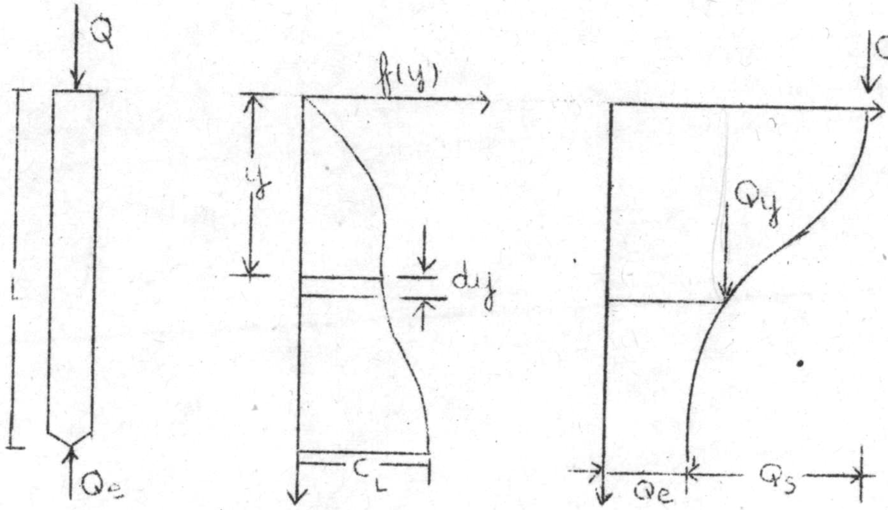
The important point in separating end load and shaft load in indirect method is to find the load distribution of pile the strain of pile at various level can be obtained by fixing electrical gauge or strain rod. Then by plotting the curre between depth of pile and cumulative deformation of pile, the load distribution can be obtained. The mathematical form of this is presented in Fig 7

The disadvantage of this method is that

1. If the pile is precast prestressed concrete the modulus of elasticity (E_p) cannot be found accurately. In common if E_c of concrete and E_s of steel is known the value of E_p



Mode shape of frictional resistance

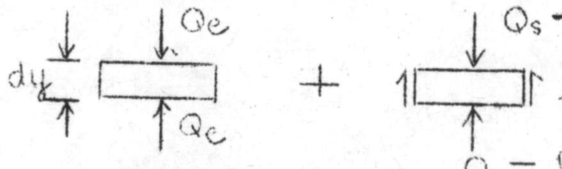


$$Q_y = Q - \int_0^y f(y)\,dy$$

$$Q_y = Q_e + Q_s - \int_0^y f(y)\,dy$$

Frictional resistance

Load distribution curve



$$Q_s - \int_0^y f(y)\,dy$$

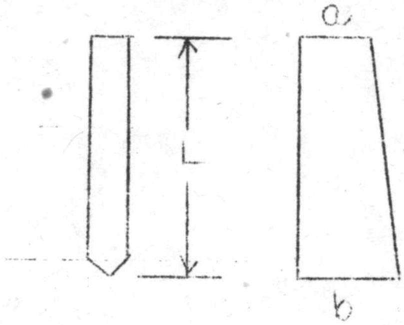
$$Q_s - \int_0^y f(y)\,dy - f(y)\,dy$$

$$d\Delta_p = d\Delta_{pE} + d\Delta_{pS}$$

$$= \frac{Q_e\,dy}{AE_p} + \frac{\left(Q_s - \int_0^y f(y)\,dy - f(y)\,dy\right)dy}{AE_p}$$

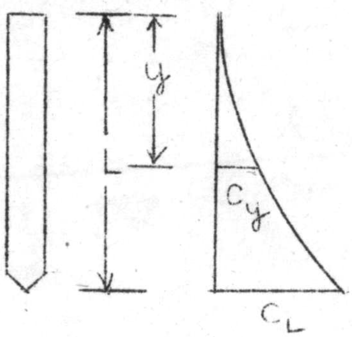
FIG 5 THE RELATIONSHIP OF THE ELASTIC SHORTENING OF PILE TO THE FRICTIONAL RESISTANCE

Mode I   Assumed linear distribution of Frictional resistance



$$\text{If } a = 0 \quad \lambda = \frac{1}{3}$$
$$a = b \quad \lambda = \frac{1}{2}$$
$$b = 0 \quad \lambda = \frac{2}{3}$$

Mode II   Assumed parabolic distribution of frictional resistance



$$C_y = \frac{y^2}{L^2} C_L$$
$$\lambda = \frac{3}{4}$$

FIG 6   The evaluation of the influence factor of pile by assuming mode shape of frictional resistance can be expressed as

$$E_p = E_c + (E_s - E_c) \frac{A_s}{A} \qquad \text{------- (12)}$$

$$A = A_c + A_s$$

A_c = area of conorete

A_s = area of steel

2. The effect of temperature on settlement of pile must be cautioned. Toeliminate this effect the settlement of dummy pile should be set up

The application of the load distribution of pile can be summarized as follows:-

1) At any particular total load, the shaft load and the end load is known

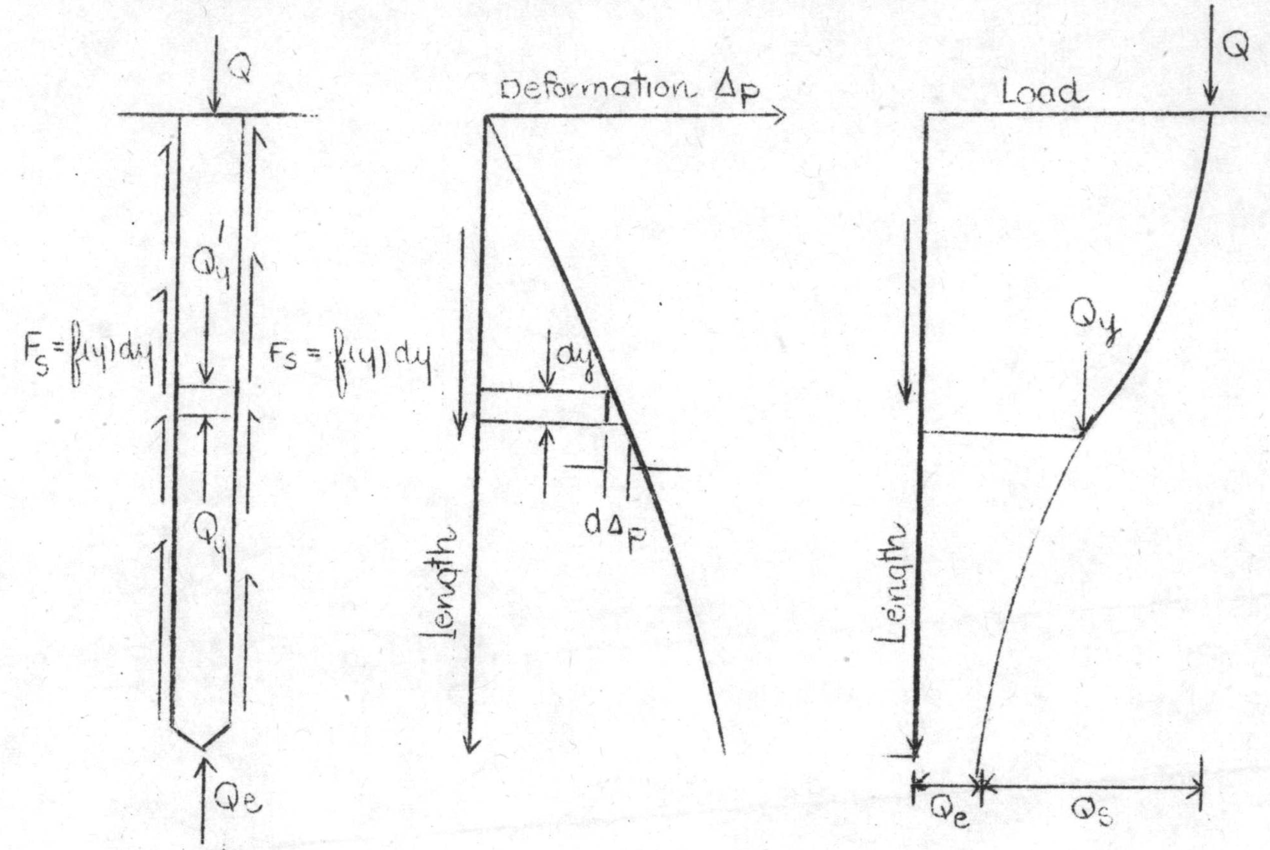
2) The skin resistance of any segments of pile at any particular total load can be found out. If the total load is the ultimate load the adhesion of any layer of soil is also known

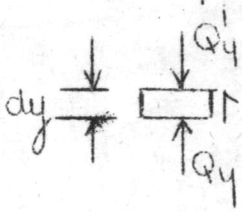
3) The mode shape of the frictional resistance and the influence factor of pile at ultimate load can be calculated from the load distribution curve

4) The settlement for mobilization skin friction can be found if the settlement and skin resistance is plotted

6.3 Graphical method

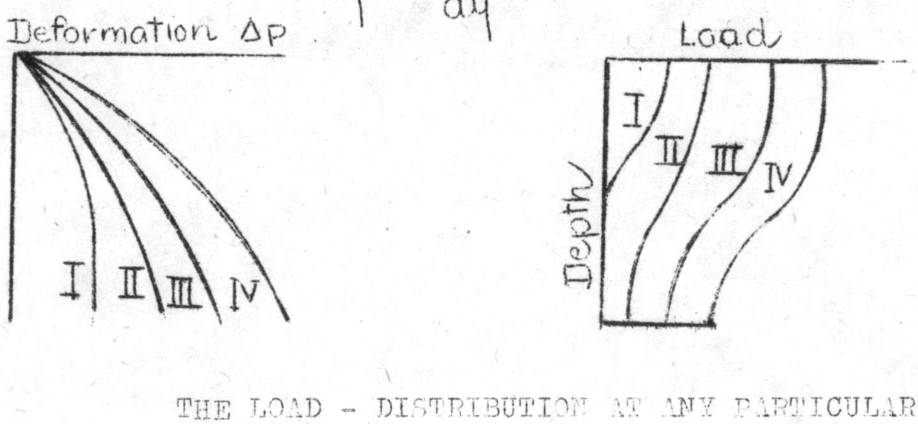
If no strain is measured along the pile, the graphical method can also separate the end load and shaft load from the load-settlement curve.





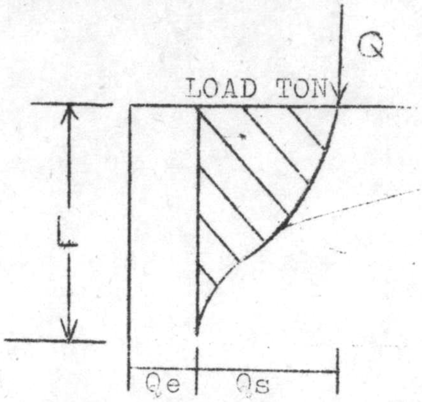
$$F_s = f(y)dy \qquad d\Delta = \frac{(Q_y' - F_s)dy}{AE} = Q_y \frac{dy}{AE}$$

$$Q_y = \frac{d\Delta}{dy} AE$$



THE LOAD - DISTRIBUTION AT ANY PARTICULAR LOAD

FIG 7 LOAD TRANSFER ALONG PILE

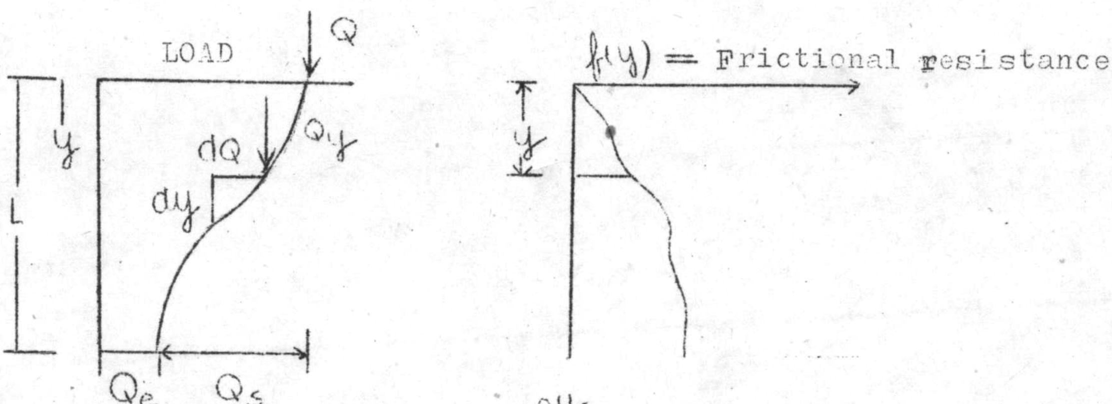


$$\lambda = 1 - \frac{\int_0^L \int_0^y f(y)\,dy\,dy}{Q_s L}$$

λ = ratio of shaded area per product of $Q_s L$

LOAD DISTRIBUTION CURVE

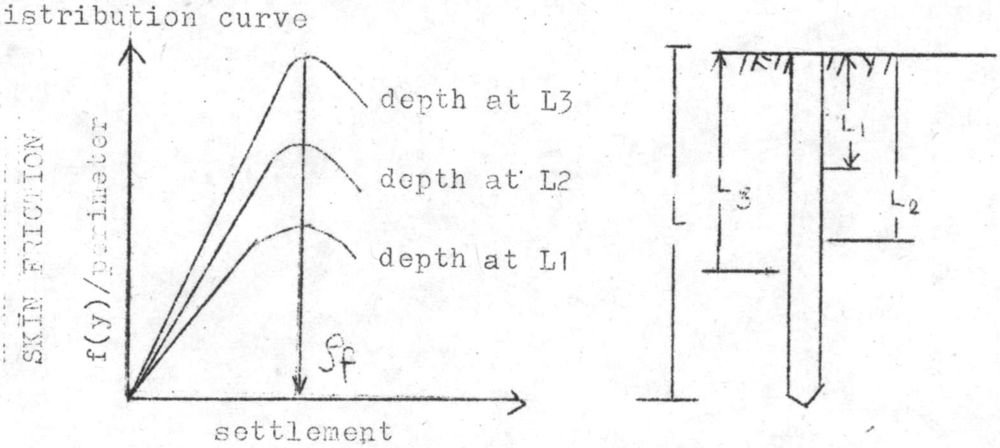
a To determine λ from the load distribution curve



$$Q - Q_y = \int_0^y f(y)\,dy$$

$$\frac{d(-Q_y)}{dy} = f(y)$$

b To deternmine the mode of frictional resistance from the load distribution curve



c To find the settlement for mobilization of skin friction (ρ_f)

FIG 8 THE APPLICATION OF LOAD DISTRIBUTION

VAN WEELE 1957 observed from the field load test that

(1) The shaft friction remained nearly constant beyond a certain pile load (that is after the mobilization of maximum shaft friction)

(2) The relationship between the load on pile end and the elastic compression of the subgrade Δ_s was linear as shown in Fig (9) the elastic compression of this case is considered to occur at the pile end and it is equal to the elastic recovery of pile end

(3) The relationship between the load on pile top and the elastic compression of subgrade was linear after the mobilization of the maximum friction, and it is parallel to the line obtained from (2) this was because after the maximum friction, any increase of load at the pile top is the same as the increase at the pile end (Fig 10)

(4) The combination of (2) and (3) will be possible to separate the skin friction and end load. First by plotting the total load V.S. elastic compression of pile toe and a line through the origin parallel to the straight portion of the curve then represents the end load the difference between the total and end load is the shaft load at any settlement.

In order to separate end bearing and shaft friction by the method of VAN WEELE it is necessary to load pile to failure and to measure elastic rebound after each load increment

The elastic compression of the subgrade is equal to the elastic rebound minus the elastic shortening of the pile. Therefore the problem is how the elastic shortening of pile can be found. The elastic shortening can be found by direct measurement if we fix a strain rod (telltale) at the base (The elastic shortening of pile is equal to the settlement of pile top minus the settlement of pile base) In case of the elastic shortening of pile is not measured the calculation is obtained from a deep-sounding (static cone penctration) or from an assumed distribution of frictional resistance along pile length.

BULLEN (1958) assumed a linear distribution of skin friction with depth and found that the influence factor varies from $\frac{1}{3}$ to $\frac{2}{3}$ He suggested therefore that elastic shortenings computed on the basis of an axial load equal to one half of the shaft load give value which are acceptable estimates of actual elastic compression

JAIN AND KUMAR (1963) combined VAN WEELE'S hypothesis and Bullen's assumption, then proposed the graphical method for separating the shaft friction and the end bearing capacity of pile

It is first assumed that there is no compression in the pile body and a curve is drawn between the elastic recovery of the pile top and the load on pile top. From this curve, values for the end resistance and the shaft friction are obtained values for the elastic compression of the subgrade are now worked out, and a second curve is drawn as shown in Fig. 13

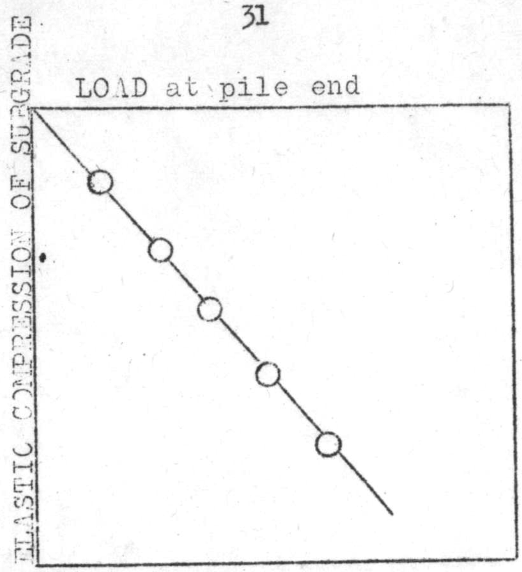


FIG 9

Relationship between end load and elastic compression of subgrade

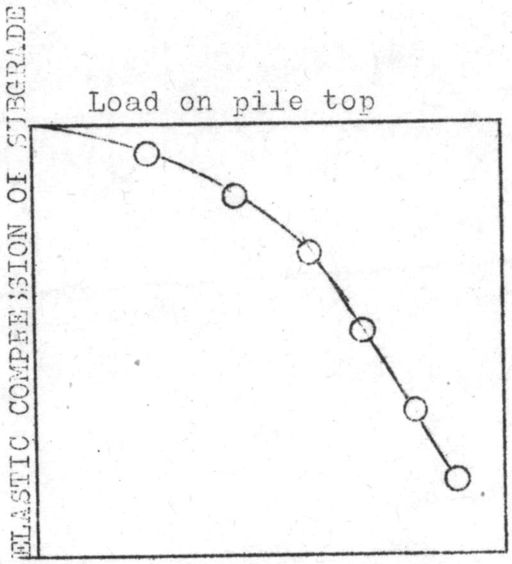


FIG 10

Relationship between load on pile top and elastic compression of subgrade

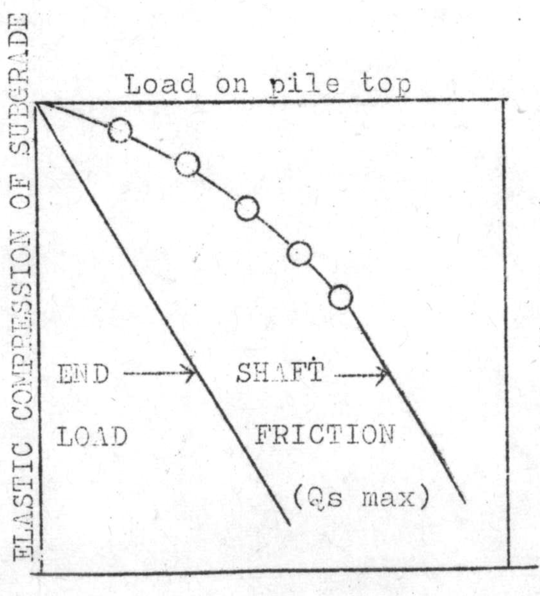


FIG 11

Relationship between load on pile top, end load, maximum shaft friction and the elastic compression of subgrade

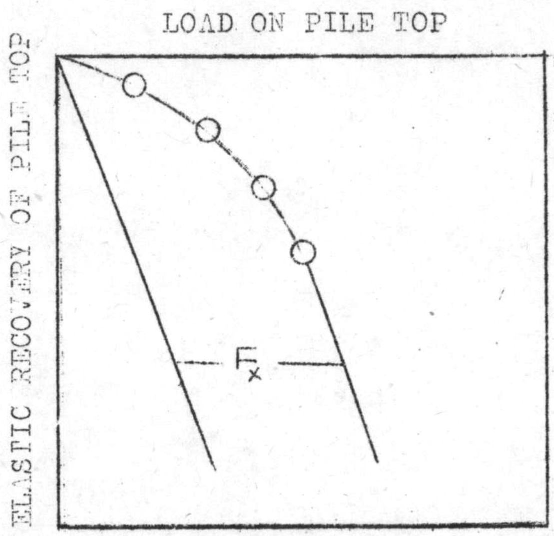


FIG 12

Relationship between load on pile top and elastic recovery of pile top

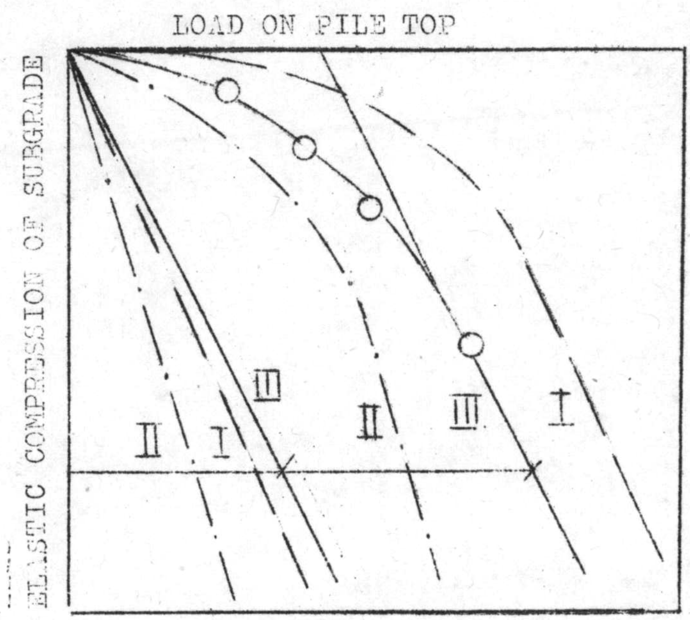


FIG 13

Separating of shaft friction and end bearing

The process is repeated and a third attempt gives nearly constant values if the fourth attempt is made, the two curves will normally coincide

7. Disturbance Caused by Pile Driving

When a pile is driven into a soft clay deposit it causes a great deal of disturbance this disturbance is of two kinds

(1) Remoulding of the clay in the vicinity of the pile to result in a decrease in strength

(2) Pore pressure increases in the soil surrunding the pile which result in decreasing in effective stress and consequently in shear strength

It is well-known that high excess pore pressures can be induced by pile driving in soft clays (CUMMINGS et al, 1950, BJERRUM & JOHANESSEN 1960) and at least one stability failure is known to have resulted from this cause in the Bangkok area (BRAND & RASAESIN 1970) Measurements of pore pressures at the face of a driven pile by KOIZUMI & ITO (1967) showed that the effective normal stress during driving was virtually zero because of the development of high excess pore pressure this liquifaction effect has also been reported by HANNA (1967) The dissipation of these pore pressure is rapid however, partly because of the convenient vertical drainage path provided by the pile surface the pore pressures induced at a distance from the pile tend to dissipate laterally with time because of the higher permeability of most sediments in the horizontal plane

The recent investigation into the effects of pile driving on soft Bangkok clay which was carried out at the plant yard of gammon (Thailand) Ltd at Lad Prao.

Susisuwan (1972) concluded that :-

1) The soil adjacent to the pile is not completely remolded but, on the contrary, is disturbed only to an extent that may be called partially remolded

2) The shear strength lost approximately 28 % (based on field yane test)

3) About one pile diameter from the pile, the undrained shear strength decrease appreciably

4) The thixotropic hardening does occur both in both weathered and soft Bangkok clay the effect is most evident in the soft clay and less pronounced in the weathered clay

5) The soil regined its strength within 14 days after pile driving. The strength regained is due to the thixotropic hardening process and the dissipation of pore water pressure

6) Within the region of local shear failure zone, the induced pore pressure are maximum outside this zone, the pore pressure decreases rapidly with distance. At a distance of approximately 16 pile diameter (8 m), the pore pressure is practically negligible

7) The time lag does occur in pore pressure measurement since the maximum pore pressure was recorded at different times after the pile driving

8) The pore pressure was observed to increase with depth

9) A rapid decrease in pore pressure was recorded when the soil changes from local to general shear failure zone

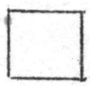
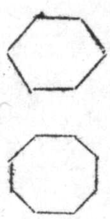
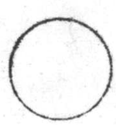
10) Most of the excess pore pressures dissipate with one month after pile driving

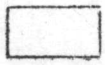
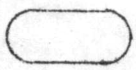
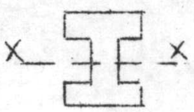
8. Pile Tests in Bangkok Clay

The two previous researches had been carried out to study the performance to failure of cast-in situ piles by CHIRRUPPAPA (1968) and SUWANAKUL (1969) The results had been dicussed by BRAND (1970) and MUKTABHANT & SUWANAKUL (1971) A number of pile test results were also collected and analyzed by VONGTHIERES (1967). HOLMBERG (1970) had investigated the ultimate load, shaft end and end load of an 58 cm octagonal piles and presented the adhesion factor of soil for Bangkok clay. Two 45 m-large diameter bored piles were load tested by PROMBOON et al (1972) but these were not taken to failure. The useful data obtained by SASISUWAN (1972) in his work on the effect of pile driving on soil disturbance and pore pressure. Jata-surivonse (1972) had carried out a good research which showed the performance of some driven and cast-in-situ piles.

Short wooden piles had also been tested singlely and in groups of four by TAECHATHUMMARAK (1970) and WATTANAVAHA (1971) Other work on groups of short piles reported by MUKTABHANT & TENGAMUAY (1970). The uses of bamboo as pile foundation had been studied by CHIRRUPPAPA (1973).

9. The Comparison of Some Shapes For Prestressed Concrete Piles

Shape	Advantages	Disadvantages
	Highest ratio of skin-friction perimeter to cross-sectional area. **Low manufacturing cost.**	Low bending **strength**
	Good ratio of skin-friction perimeter to cross-sectional area. **Low manufacturing cost. Good bending strength on major axes.**	
	**Approximately equal bending strength on all axes. Good penetrating ability. Good column stability (1/r ratio)**	Surface defects on top sloping surface as cast are hard to avoid
	Equal bending strength on all axes. No sharp corners-aids appearance and durability Minimum wave and current loads. Good column stability (1/r) ratio.	Manufacturing cost generally higher surface defects on upper surfaces are cast are hard to avoid

Shape	Advantages	Disadvantages
 or 	May be used where greater bending strength is required around one axis especially if minimum surface to lateral wave and current forces is desired.	Difficulty in maintaining orientation during driving
	High bending moment about both axes in relation to cross sectional area	High cost of manufacture difficulty of orientation
	High bending moment about axis X - X in relation to cross sectional area	High cost of manufacuture difficulty of orientation