# Separation of Skin Friction and Point Resistance in R.C.C. Piles—A New Approach

*by*

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## 1. Introduction

LOAD tests on full size piles are recommended for determining the ultimate bearing capacity of piles by Terzaghi (1948). The customary factors of safety for calculating safe working load range between 1·5 to 2.5 (Chellis, 1951 & I.S., 1964). The principal uncertainties in the load test method lies in the interpretation of the load-settlement curves as part of the lcad is transferred to the soil as skin friction and partly as bearing. Cyclic load test has been suggested (Jain and Kumar, 1963) for separating skin friction and point bearing. The problem becomes more complicated when the piles are deriven through a number of starta few of them being compressible. In such cases a portion of skin friction offered by the compressible layers during the course of testing is not available later. The present paper gives a new interpretation to cyclic load test results which is different than the interpretation given by Jain and Kumar (1963). It is also possible in this case to calculate separately the skin friction offered by each layer through which the pile is passing.

## 2. Simplifying Assumptions

Because of wide variations in the soil conditions, methods of driving or casting the pile and also in the elastic and plastic properties of pile material itself; any attempt to assess the safe load in pile by the Theory of Elasticity needs a number of simplifying assumptions. The main assumptions made in this paper are as follows :—

(*a*) The load on the pile top is resisted by the skin friction developed in the surrounding soil due to the elastic shortening of the pile.

(*b*) The skin friction per unit of a contact area in a particular *stratum is constant throughout its depth.*

(*c*) The load is transferred to the 'pile point' only after the test load exceeds the ultimate value of skin friction.

(*d*) The pile follows Hook's law during the course of the test.

## 3. Derivation of Working Formulae

The load bearing R.C.C. piles may be divided in the following principal categories :—

(*i*) Piles driven in the loose sand.

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(*ii*) Piles resting on sound rock.

(*iii*) Pile point embedded in the dense sand.

### 3.1 PILES DRIVEN INTO LOOSE SAND

The ultimate bearing capacity of such piles usually named as 'Friction Piles' can be expressed as a sum of skin frictional resistance $Qr$ and point resistance $Qp$, i.e.,

$$Qd = Qr + Qp \qquad \ldots(1)$$

Let the load on the pile top be $P$ and the force in the pile at a depth $x$ be $Px = P - prx$. Under this force an infinitesimal element $dx$ is shortened by $d\epsilon$. According to Hooks *law:*

$$d\epsilon = \frac{Px}{AE}dx$$

Where $E$ is Young's Modulus of Elasticity of the pile material, $p$ is perimeter of pile and $r$ is the skin friction per unit area.

The deformation of pile length for a length $l$ is given by

$$\epsilon = \frac{1}{AE}\int_0^1 Px\,dx = \frac{1}{AE}(Pl - \tfrac{1}{2}prl^2)$$

Also we have $P = prl$, i.e., $l = P/rp$

Therefore, $\epsilon = \dfrac{1}{AE}\left(Pl - \dfrac{Pl}{2}\right) = \dfrac{Pl}{2AE}$

i.e.,
$$\left(\frac{\frac{P+O}{2} \times L}{AE} = \frac{\text{Av. Force} \times L}{AE}\right)$$

$$= \frac{P^2}{2rpAE} = \frac{P^2}{4K}$$

i.e.,
$$4K\epsilon = P^2 \qquad \ldots(2)$$

Where $K = 1/2\ AErp$, a constant for a particular pile in a particular stratum.

When $P = Qr$ full friction gets mobilized and as $P$ exceeds this value the additional load is shared by the 'point bearing' and continues to take the load till $P$ becomes equal to the ultimate bearing capacity of the pile $Qd$. At this stage the pile starts settling and $\epsilon$ becomes constant. The maximum value of $\epsilon$ can be given by the expression

$$\epsilon = \frac{\frac{1}{2}(Qr + 2Qp)}{AE} L = \frac{L}{2AE}(Qr + 2Qp)$$

But
$$Qr = prL$$

i.e.,
$$\epsilon = \frac{Qr}{2AEpr}(Qr + 2Qp) = \frac{Qr}{4K}(Qr + 2Qp) \qquad \ldots(3)$$

The $P-\epsilon$ curve is shown in Figure 1.

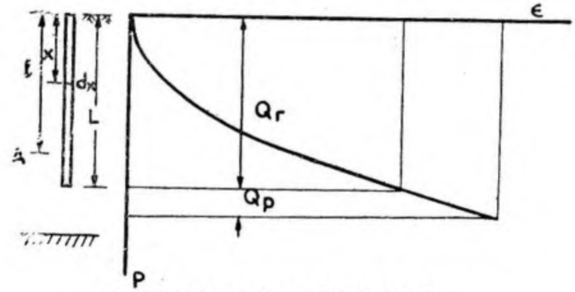


**FIGURE 1 : Failure of Friction Pile.**

### 3.2 PILE RESTING ON SOUND ROCK

3.2.1 *Pile passing through Single Layer of Soil*

Under ideal conditions such piles driven to sound rock act like piers and their settlement does not exceed the elastic shortening of the pile. In practice, however, these piles also show plastic deformations.

The settlement behaviour of this pile is quite similar to the 'Friction Pile' except when the load $P$ exceeds the total frictional resistance $Qr$. In that case the pressure at the pile bottom becomes $P-Qr$ and the average pressure compressing the pile becomes $(P-\frac{1}{2}Qr)$.

Therefore,
$$\epsilon=\frac{(P-\frac{1}{2}Qr)L}{AE}$$
$$=\frac{(P-\frac{1}{2}Qr)Qr}{AEpr}$$
$$=\frac{(P-\frac{1}{2}Qr)\,Qr}{2K}$$

Hence,
$$P=\frac{2K}{Qr}\epsilon+\tfrac{1}{2}Qr \qquad \ldots(4)$$

The Equation (4) represents a straight line tangent to the parabola $P^2=4K\epsilon$ *at the point* $P=Qr$.

It will, therefore, be seen that these piles have very large strength and invariably fail by the crushing of concrete (Figure 2).

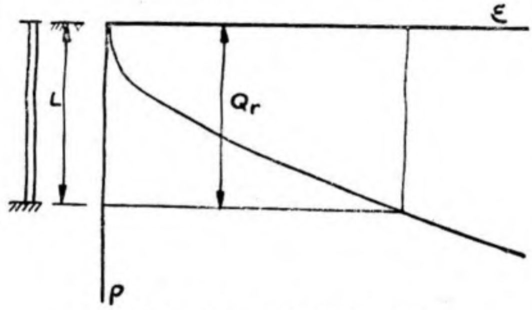


**FIGURE 2 : Failure of Bearing Pile.**

3.2.2 *Pile passing through Multi-layer System*

We will now consider a pile passing through two layers of soil with characteristic $r_1$ and $r_2$. Then we have,

$$P = pr_1L_1 + pr_2L = Qr_1 + pr_2L$$

Therefore, $$L = \frac{P - Qr_1}{pr_2}$$

The average pressure in the top layer is $\left(P - \frac{Qr_1}{2}\right)$ and in the middle layer is $\frac{1}{2}(P - Qr_1)$.

Hence, $$\epsilon = \frac{(P - \frac{1}{2}Qr_1)L_1}{AE} + \frac{\frac{1}{2}(P - Qr_1)L}{AE}$$

$$= \frac{(2P - Qr_1)Qr_1}{4K_1} + \frac{(P - Qr_1)^2}{4K_2}$$

Here $4K_1 = 2AEpr_1$ and $4K_2 = 2AEpr_2$ and are constant for a given pile and a particular stratum. On re-arranging we get,

$$(P + \alpha Qr_1)^2 = 4K_2\epsilon + \alpha(1 + \alpha)Q^2r_1 \qquad \ldots(5)$$

Where $1 + \alpha = K_2/K_1$. The above equation represents a parabolic curve with shape factor '$K_2$' and apex at $(-\alpha Qr_1, -\frac{\alpha(1+\alpha)}{4K_2}Q^2r_2)$. It is also evident that this parabola crosses the Equation (2) at the point where

$$\epsilon = \frac{Q^2r_1}{4K_1}$$

It will, therefore, be seen that the friction offered by each layer can be represented by different parabolic curves with different shape factors $K_1$ and $K_2$ depending on the frictional properties of these layers.

The same principle can be extended to a multi-layer system wherein a number of parabolas will represent the $P-\epsilon$ curve. Each parabola will represent a different stratum and its shape will depend upon the friction characteristic of the individual stratum.

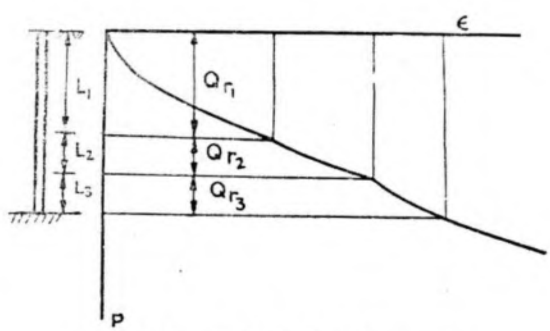


**FIGURE 3 : Bearing Pile (3-layer system).**

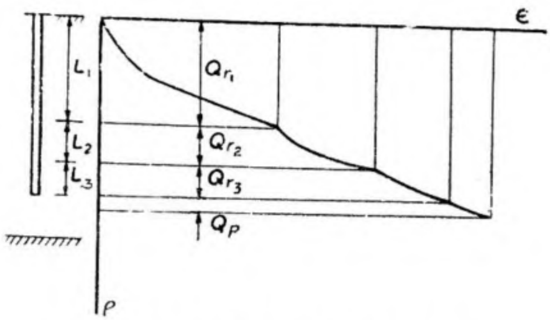


**FIGURE 4 : Bearing Pile (bearing on sand).**

### 3.3 PILE POINT EMBEDDED IN DENSE SAND

The behaviour of these piles is a combination of the behaviour of friction pile and point bearing pile. The $P-\epsilon$ curve shall also be a combination of the curve in Figure 3 and the one in Figure 1. A typical failure curve for such a pile should be of the shape given in Figure 4.

## 4. Interpretation of $P-\epsilon$ Curve

From the curves derived in the preceding para it is evident that there is a marked 'kink' in the $P-\epsilon$ curve at the boundary of two strata The curves on both the sides of the 'kink' is parabolic with the shape factor $k$ in each layer being different and depending upon the friction property of the particular layer. This property of the curve when studied in relation to the sub-soil profile will give us immediately the ultimate frictional resistance offered by each individual layer. The straight line portion of the curve gives us directly the effective point resistance offered at the tip of the pile.

The author was associated with a series of cyclic load tests conducted on R.C.C. driven piles of 45 cm and 50 cm diameter. These piles were driven in the Naval Project area at Vishakhapatnam, as a part of an investigation programme for assessing the load carrying capacity of the piles. $P-\epsilon$ curves were drawn for test pile Nos. C-27/1, M-14, M-30 and M-47 and their behaviour was compared with the 'Pile driving resistance record' and the theoretical curve derived in preceding para. Clearly defined 'kinks' were observed at every change of strata as predicted by analytical assessment.

## 5. Conclusion

The failure curves derived analytically for piles passing through compressible layers and embedded in sand (Figure 4) compare very well with the test results (curve A-4). This property of the $P-\epsilon$ curve can be used to calculate the total frictional resistance as well as the frictional resistance offered by the individual stratum of soil.

This information has been used in Annexure-A for calculating the safe load carrying capacity of the piles passing through compressible strata and resiting on dense sand.

**Notations**

$A$ = Cross-sectional area of pile

$dx$ = Differential $x$

$dE$ = Differential $E$

$E$ = Young's Modulus of Elasticity of Pile

$K, K_1, K_2$ = Skin friction constants

$L, L_1, L_2$ = Total length of pile, thickness of strata

$l$ = Variable length

$P, Px$ = Load on pile, residual load on pile at depth $x$.

$Qd$ = Ultimate bearing capacity of pile

$Qp$ = Ultimate point resistance of pile

$Qr$ = Ultimate frictional resistance of pile

$r, r_1, r_2$ = Skin frictional resistance of soil per unit area of pile

$x$ = Unknown length parameter

$\epsilon$ = Elastic longitudinal strain in pile

$\alpha$ = Perimeter of pile

ANNEXURE—A

**Load Test Results**

**OBSERVATIONS**

Cyclic load tests were done on Pile Nos. C. 27/1, M-14, M-30 and M-47. The load tests were conducted in accordance with the procedure detailed in clause C-4 of IS : 2911 (Part I) of 1964. The settlement and rebound at the pile top were noted for each increment of load and these are tabulated in Tables I & II.

The driving record of the steel casing tube is record in Figure 5 and Table III. Curve for load versus total settlement of pile top are given in Figure 6.

*(A) Load-Elastic Settlement Curve*

The $P—\epsilon$ curves drawn in Figure 7 indicate the following :

(*a*) Kinks are observed in the curve and these closely correspond to the curve analytically derived as per para 3.3.

(*b*) the curves in between the kinks are well defined.

(*c*) marked changes in the net settlement (plastic settlement) at pile top observed in net settlement versus load curve (Figure 8) occur almost at the same loads at which kinks are observed in $P—\epsilon$ curve.

(*d*) The number of soil layers as observed in driving record (number of blow per unit penetration versus depth) closely correspond to the number of layers observed in $P—\epsilon$ curve.

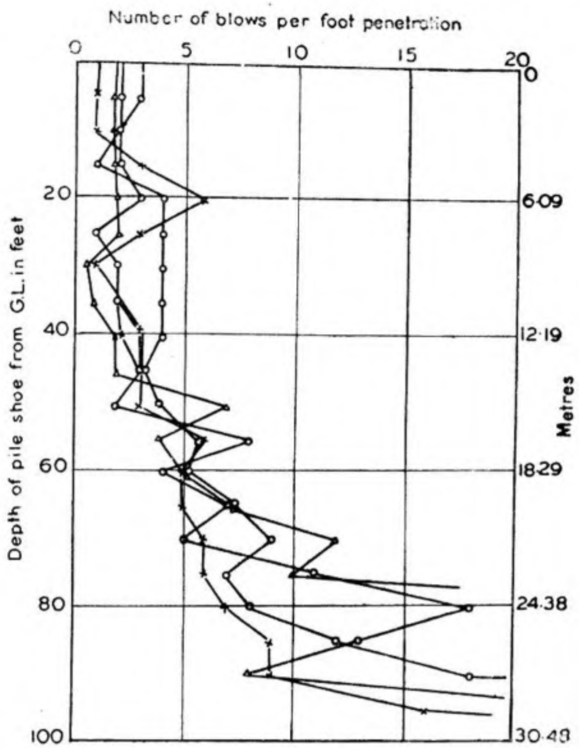


FIGURE 5.

**TABLE I**

**Load Test Records for Test Pile Nos. C-27/1—M-14.**

| Load on pile top in tonnes | Test Pile No. C-27/1 | | | Test Pile No. M-14 | | |
|---|---|---|---|---|---|---|
| | Gross settle-ment mm | Net settle-ment mm | Elastic recovery mm | Gross settle-ment mm | Net settle-ment mm | Elastic recovery mm |
| 12 | 0·263 | 0·235 | 0.030 | 0·210 | 0·025 | 0·185 |
| 24 | 0·490 | 0·300 | 0·190 | 0 570 | 0·050 | 0·520 |
| 36 | 0·653 | 0·333 | 0·320 | 0 740 | 0·070 | 0·670 |
| 48 | 0·953 | 0·340 | 0·619 | 1·090 | 0·133 | 0·957 |
| 60 | 1·326 | 0·413 | 0·913 | 1·440 | 0·200 | 1·240 |
| 72 | 1·703 | 0·373 | 1·330 | 1·890 | 0·203 | 1·687 |
| 84 | 2·240 | 0·473 | 1·767 | 2·450 | 0·250 | 2·200 |
| 96 | 2·850 | 0·640 | 2·210 | 3·150 | 0·435 | 2·715 |
| 108 | 3·700 | 1·080 | 2 620 | 4 010 | 0·820 | 3·190 |
| 120 | 4·870 | 1·830 | 3 040 | 5·270 | 1·360 | 3 910 |
| 132 | 6·380 | 2·780 | 3·600 | 7 340 | 2·560 | 4·780 |
| 144 | 9·010 | 4·740 | 4·270 | 10·190 | 4·240 | 5 950 |
| 156 | 12·290 | 7·520 | 4·770 | 14·430 | 8·000 | 6·430 |
| 168 | 43·810 | 36·000 | 7·810 | 22·280 | 15·010 | 7·270 |
| 180 | 54·030 | 45·680 | 8·350 | 34·690 | 27·200 | 7·490 |

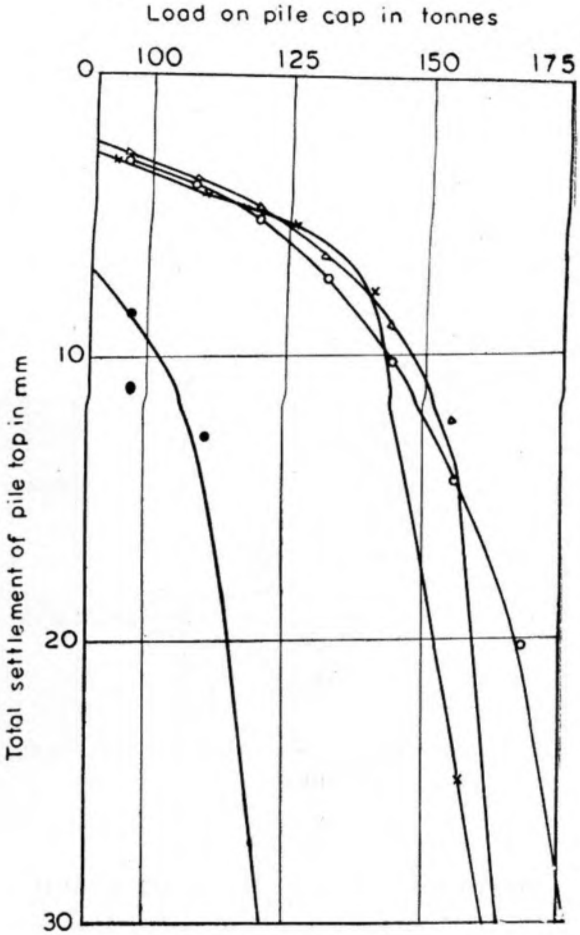


FIGURE 6.

TABLE II

**Load Test Records f›r Test Pile Nos. M-30—M-47.**

| Land on pile top in tonnes | Test Pile No. M-30 | | | Load on pile top in tonnes | Test Pile No. M-47 | | |
|---|---|---|---|---|---|---|---|
| | Gross settle-ment mm | Net settle-ment mm | Elastic recovery mm | | Gross settle-ment mm | Net settle-ment mm | Elastic recovery mm |
| 13·88 | 0·381 | 0·001 | 0·380 | 15·70 | 0·210 | 0·000 | 0·210 |
| 27·76 | 0·890 | 0·000 | 0 890 | 31·40 | 0·540 | 0·020 | 0·520 |
| 41·64 | 1·800 | 0·070 | 1·730 | 47·10 | 1·100 | 0·080 | 1·020 |
| 55·52 | 2·640 | 0·100 | 2·540 | 62·80 | 1·650 | 0·045 | 1·605 |
| 69·40 | 3·780 | 0·150 | 3·630 | 78·50 | 2 200 | 0·075 | 2·125 |
| 83·28 | 4·310 | 0·120 | 4·190 | 94·20 | 3·080 | 0·250 | 2.830 |
| 97·16 | 8·400 | 1·340 | 6·060 | 109·90 | 4·350 | 0·660 | 3·690 |
| 97·16 | 11·000* | 3·700* | 7·300* | 124·60 | 5·410 | 1·100 | 4·310 |
| 111·04 | 12·700 | 5·570 | 7·130 | 141·30 | 7·750 | 2·460 | 5·290 |
| 124·92 | 38·600 | 29 650 | 8·950 | 157·00 | 24·940 | 16·770 | 8·170 |
| 138·80 | 57·500 | 43·280 | 14·220 | 172·70 | 42·430 | 32·200 | 10·230 |

*Settlement values after keeping a constant load on pile for 24 hours.

TABLE III

**Record of Driving Resistance of Steel Casing of the Test Piles.**

| Driving Resistance in blows per ft. of penetration | For a depth in feet for test pile Nos. | | | |
|---|---|---|---|---|
| | C-27/1 | M-14 | M-30 | M-47 |
| Up to 5 | 0-53 | 0-57 | 0-68 | 0-60 |
| Up to 10 | 53-68 | 57-82 | 68·88 | 60-90 |
| More than 10 | 68-73 | 82-96 | 88-95 | 90-95 |

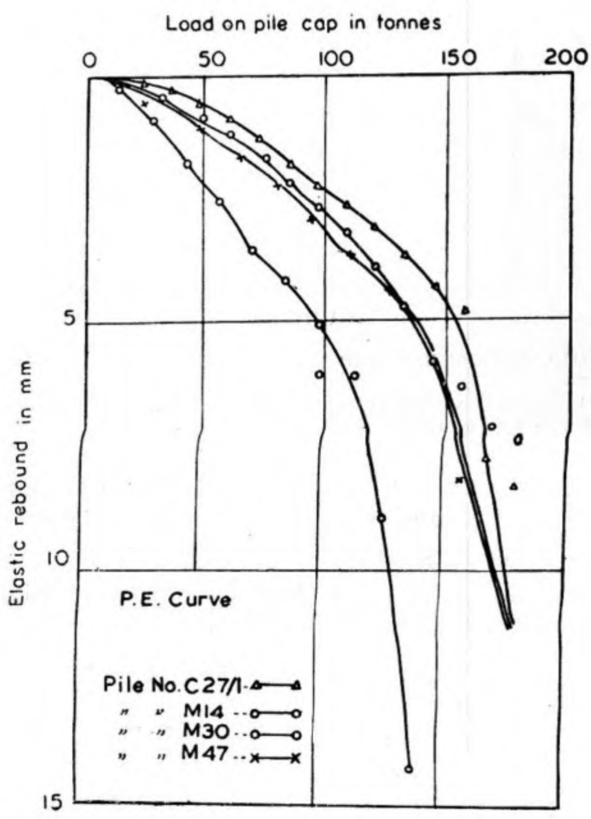


FIGURE 7.

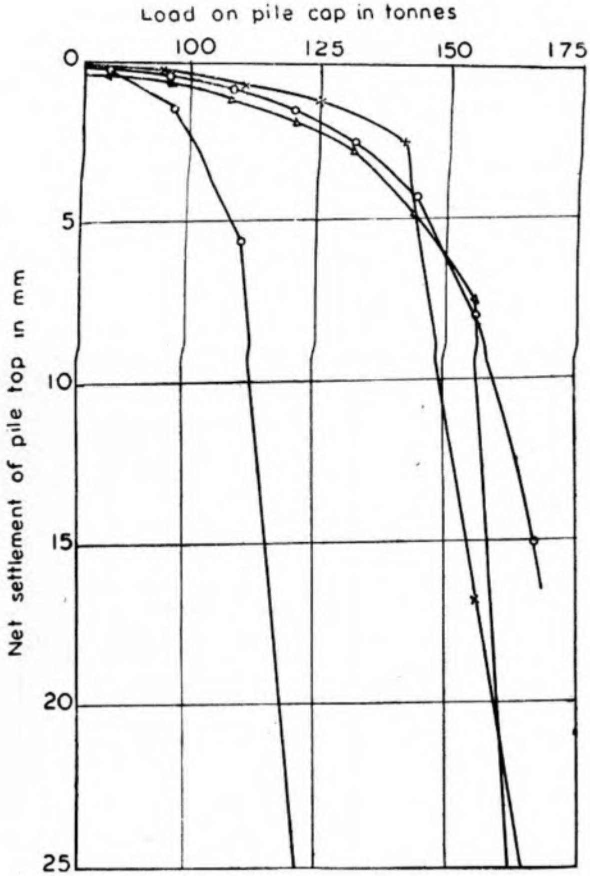


**FIGURE 8.**

### (*B*) *Load-Plastic Settlement Curve*

Further the load versus settlement curve indicates that the rate of net settlement of pile top increases very rapidly after certain stage of loading. Beyond this load, even small increments, in load produce large plastic settlements. Also in one case (Pile M-30) when the load was maintained for 24 hours the net settlement actually increased under constant load.

The test results indicate that :

(*a*) The definition of failure load (or ultimate load) as the load causing settlement of one-tenth the diameter of pile at pile top is not rational.

(*b*) More appropriately the ultimate (or failure) load would be considered as the load at which the rate of plastic settlement observed at pile top is quite large. It is proposed that the failure load be taken as the load causing a rate of net settlement of 1 mm per ton increment in pile load.

## COMPUTATION OF SAFE BEARING CAPACITY OF PILE

Based on the load tests carried out and the theoretical analysis explained earlier it is suggested that the safe bearing capacity of pile be computed from cyclic load tests as under :

(*a*) Draw the $P-\epsilon$ curve and compute skin friction afforded by each layer of sub-soil.

(*b*) Ultimate capacity of the pile will be reconed as the load at which the rate of net settlement becomes 1 mm per ton of load increment.

(*c*) Calculate the total values of skin friction and point bearing corresponding to this load from $P-\epsilon$ curve.

(*d*) From the total values of skin friction deduct the friction resistance of compressible strata as obtained from the $P-\epsilon$ curves, to arrive at the ultimate effective frictional resistance.

(*e*) Calculate the safe capacity of pile by applying a factor of safety of 2·5 for ultimate point bearing valve and 2 for ultimate effective skin friction valve.

An example of calculation of safe capacity for pile based on above method is given in Table IV.

**TABLE IV**

| Type | | | Resistance offered by Soil in tonnes | | | Remarks |
|---|---|---|---|---|---|---|
| | | | Pile No. C-27/1 | Pile No. M-14 | Pile No. M-30 | Pile No. M-47 | |
| Ultimate Resistance | | 1 | 156 | 168 | 111 | 141 | From Annexure A-3 |
| Frictional Resistance | STRATA-I | 2 | 48 | 52 | 53 | 55 | From Annexure A-4 |
| | STRATA-II | 3 | 45 | 40 | 31 | 34 | |
| | STRATA-III | 4 | 57 | 63 | 26 | 42 | |
| | Total | 5 | 150 | 155 | 110 | 131 | |
| Ultimate Point Resistance (1)—(5) | | 6 | 6 | 13 | 1 | 10 | |
| Ultimate Effective Frictional Resistance (5) — (2) | | 7 | 102 | 103 | 57 | 76 | The Strata I was found to be soft marine clay of recent origin |
| Safe Capacity $=\frac{(6)}{2\cdot5}+\frac{(7)}{2}$ | | 8 | 53·4 | 56·7 | 28·9 | 42 | Average Value =44·25 tonnes. |

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