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An Experimental Study on Pull-out Behaviour of Pile Groups in Sand

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Introduction

Deep foundations in the form of piles or piers are used when soft compressible strata exist near the ground surface and extend upto large depths. Piles and pilegroups are often subjected to uplift, when they are used for foundations of transmission towers, mooring systems for ocean surface or submerged platforms etc. Vertical or inclined upward load in these piles develop as a consequence of horizontal loads or wave actions. During uplift, distributed stresses are developed along the pile and with the progress of load application the surrounding soil mass experiences relatively less deformation and failure occurs all of a sudden, unlike failure in compression, which may occur gradually.

The behaviour of single pile under uplift has been reported in a few research papers, as has been discussed in the next section. Attempts have been made by the researchers to study the characteristics of failure plane, the ultimate uplift capacity of pile and the like. In case of pilegroups under uplift, not much has been published as yet and only a few semiempirical formulations have been reported. Generally, when a pile group is subjected to a moment due to horizontal load or wave action, some piles of the group experience uplift and others undergo compression. However, the worst possible case may arise, when all the piles of a group are subjected to uplift, that is, when the group is subjected to a vertical pull. Pile groups constructed under offshore platforms are typical examples of those in which all the piles are subjected to uplift. The present investigation aims at gaining an insight in the behaviour of pile groups under uplift, through an extensive experimental study.

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A Brief Review of Past Works

Meyerhof and Adams (1968) developed an approximate theory with some simplifying assumptions in respect of failure plane. The theory was derived for uplift capacity of strip or continuous footings on sand and was then modified for circular and rectangular footings. It was then extended for piles and pile groups also. In sand, the experimental uplift capacity coefficient increased with angle of internal friction of soil (ϕ) and ratio of depth to width of footings (D/B). However, the experimental uplift capacity coefficient remained constant for large values of D/B for circular footings. For long rectangular footings it was independent of ϕ.

Vesic (1971) presented a state of the art study on breakout resistance of objects embedded in ocean bottom. Solution by Vesic gave radial pressure (q_o) needed to breakout a cylindrical or spherical cavity of radius (R) placed at depth (D) below the surface. The earth pressure theory used in the solution for expansion of cavities was recommended for prediction of breakout forces. He also proposed that the study of uplift capacity could be carried out taking into consideration the contribution of soil adhesion, ocean bottom slope, inclination and eccentricity of load etc.

Hanna (1972) identified some of the factors, e.g. geometry of anchor or group of anchors nature of loading and embedment depth to anchor diameter ratio, affecting the behaviour of anchors and groups of anchors in sand through a comprehensive laboratory testing programme. The general study indicated that anchors in groups interacted and ultimate group efficiencies were less than 100%. The ratio of embedment length to anchor diameter was kept from 6 to 12.

Das and Seeley (1975) carried out pull out tests on three vertical model anchor plates of sizes 51×51 mm. 51×153 mm, 51×510 mm, and of thickness 3.2 mm. The plates were kept embedded into the sand vertically and the pull was applied. Embedment ratio varied between 1, 3 and 5. The load versus displacement relationship was approximated by a rectangular hyperbola and expressed as $\overline{\Delta}/\overline{P} = a + b\,\overline{\Delta}$. The plot of $\overline{\Delta}/\overline{P}$ vs. $\overline{\Delta}$ yielded $a = 0.15$ and $b = 0.85$. They suggested further investigation on the influence of soil properties on values of "a".

Here, $\overline{P} = \dfrac{P}{Pu}$ and $\overline{\Delta} = \dfrac{\Delta}{\Delta u}$

Pu, Δu were ultimate pullout load and corresponding displacement and Δ was the displacement when the applied load was P.

Das and Seeley (1976) carried out model tests for studying uplift capacity of buried single pile and groups of piles in a box full of sand. For groups of piles, efficiency vs. spacing relationship was determined for group sizes : 1×2, 1×3, 1×4, 2×2, 2×3, 3×3. A general conclusion could be drawn that the isolation spacing is about 4 to 6 times pile diameter. Isolation spacing here refers to the pile spacing at which the piles tend to behave as individual ones.

Chattopadhyay and Pise (1986) proposed an analytical model for predicting uplift capacity of piles in sand based on a few assumptions regarding the failure surface. Failure surfaces for different values of angle of friction (ϕ) and pile friction angle (δ) were evaluated. Net uplift capacity factor was evaluated for $\phi = 40°$ and different values of δ and λ by numerical integration (λ = slenderness ratio = Embedded depth/pile diameter). Reasonable agreement was observed between the experimental and theoretical results. Also, field tests results agreed well with the predicted values. The proposed analysis was capable of predicting the nonlinear variation of dimensionless load factor with λ, whereas Meyerhof's analysis (Meyerhof, 1973) predicted linear variation.

Saran, Ranjan and Nene (1986) carried out an analysis for predicting critical pullout and breakout loads. They used hyperbolic stress strain curves of soil for predicting load- deformation characteristics of anchors. They attempted to analyse strip, square and circular anchors. They also compared available experimental results with analytical data and found good agreement between the two.

Ruffier and Mahler (1989) worked with a finite element simulation of the uplift of plates and foundations. The program used joint elements for soil structure interface and 2-D axisymmetric elements with four nodes for soil and foundations. Several field tests were simulated. Soil nonlinearity and plastification were taken into account. The results suggested that the proposed procedure could be used for prediction of pullout resistance of similar foundations.

Sarac (1989) presented a method for uplift capacity calculation for shallow anchors pulled in vertical direction using general logarithmic spiral surface of failure. Design charts were also given. In sand, the comparison of experimental and theoretical results indicated very good agreement.

Nene and Garg (1991) carried out an analysis by limit equilibrium method for shallow plate anchors in reinforced cohesive soil. According to them, the breakout load for shallow plate anchors (q_o) was given by

$$q_o = c\,F_c + \gamma\,H\,F_\gamma \qquad (1)$$

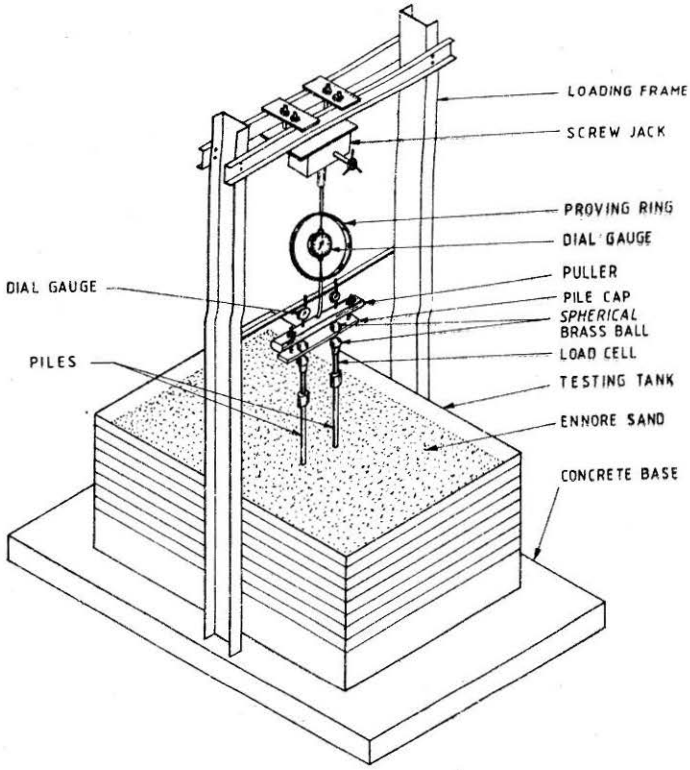

FIGURE 1 : Line Sketch of Experimental Setup

where,
c = unit cohesion,
γ = unit weight of soil,
H = depth of anchor,
F_c, F_γ = nondimensional factors,
$F_c = 4 \lambda (1 + \lambda \tan \phi)$,
$F_\gamma = 1 + 2 \lambda \tan \phi + 4/3 \lambda^2 \tan^2 \phi$, and
λ = total depth/total width.

They also conducted model tests to investigate the effects of geosynthetics on load displacement behaviour of square and circular plate anchors. There was a fairly good agreement between the theoretical and experimental results.

The foregoing literature review shows that detailed behaviour of pilegroups under uplift in respect of the distribution of load in individual piles, transfer of load to the soil from the pilegroup along depth and formation of failure surface were not studied in details by the previous researchers.

Scope of the Work

1. Experiments with different pile groups were carried out, varying parameters e.g. (a) pile spacing, (b) arrangement of piles in a group and (c) ratio of embedment length of pile to its diameter. In each test, load shared by each pile, the total load taken by the group, the pile group movement and also the variation of load along the depth of a pile were observed. An attempt was also made to observe the failure plane developed inside sand mass by placing some fragile material around the pile group.

2. In order to represent the influence of different parameters on uplift capacity of pile group a parametric study was carried out.

Test Set Up and Equipment

The complete experimental set up is shown in Fig. 1, it includes the following components.

Test tank

A segmented aluminium tank of size of 90 cm × 90 cm × 110 cm deep was fabricated. A loading frame was used to suit to the requirements of the test.

Model piles

Aluminium model piles of 25 mm O.D. and 21 mm I.D. were cut into required length of 1.1 m. Their bottoms were plugged by rubber corks and their tops were threaded for fitting the load cells.

Pile caps

10 mm thick aluminium plates were cut into sizes for use as pile caps. Holes were made in these plates as per requirement of the arrangement of piles in a group and pile spacing. Details of fixing piles with pile cap has been shown in Fig. 2 and pile caps with different configurations of piles have been shown in Fig. 3.

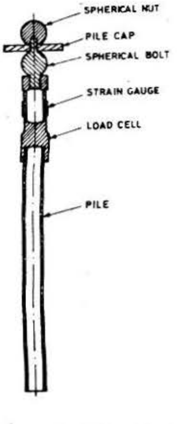

FIGURE 2 : Sketch Showing Fixing Arrangement of Pile and Pile Cap

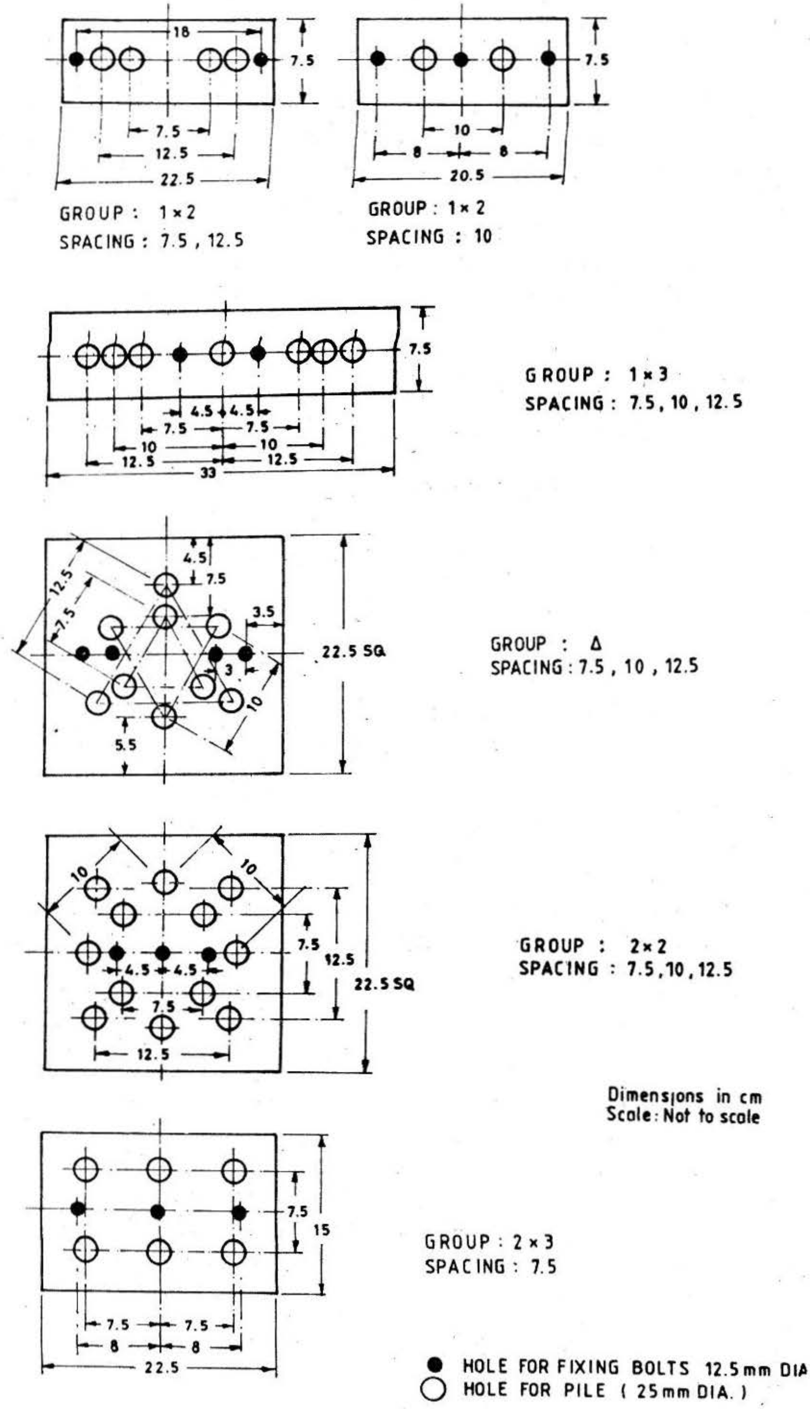

FIGURE 3 : Pile Caps for Different Pile Groups with Varying Spacings

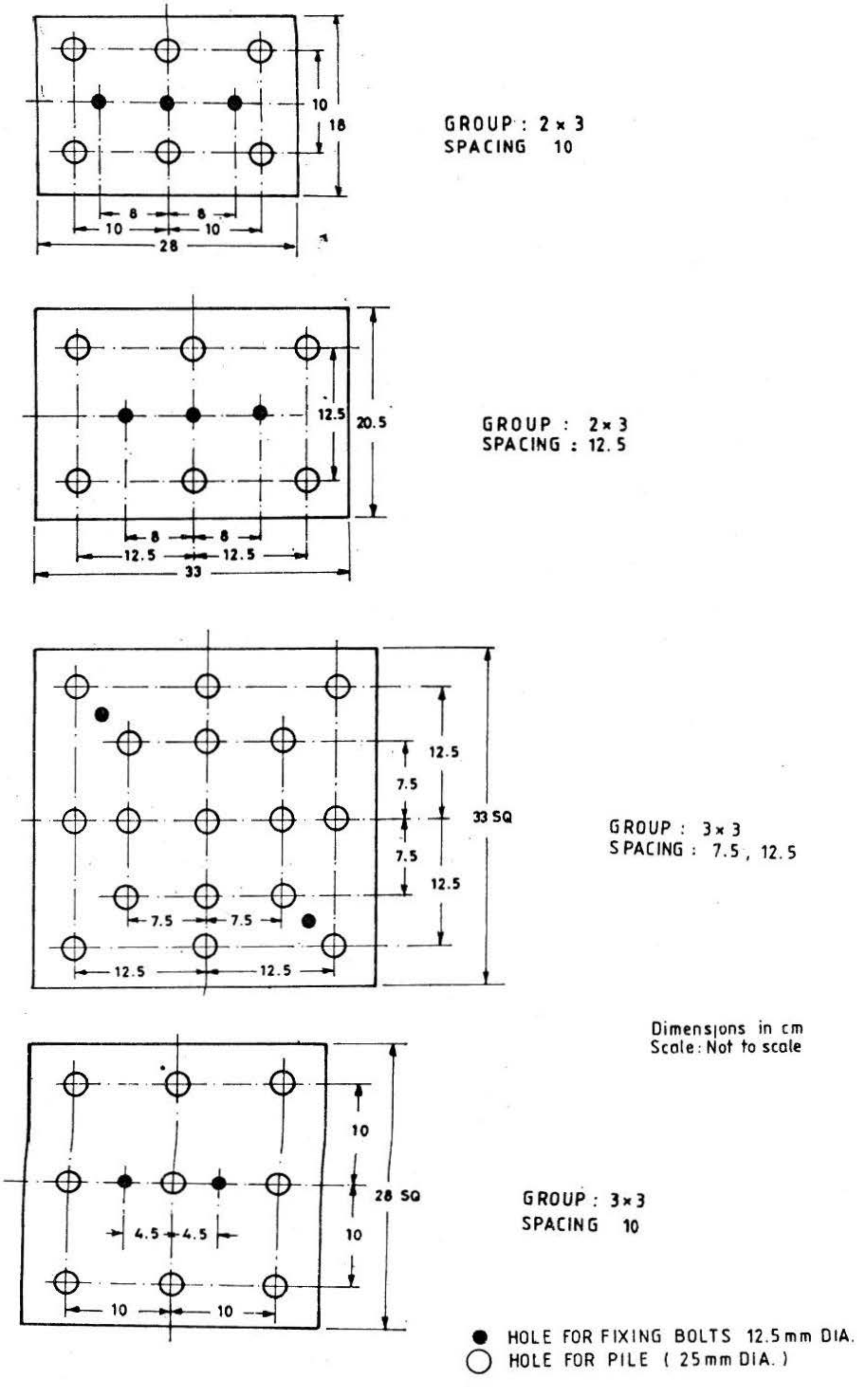

FIGURE 3 : Pile Caps for Different Pile Groups with Varying Spacings

Spherical brass balls

Spherical brass balls with internal threads were fabricated for fixing piles. They helped in supporting the piles under uplift as the pile cap moved upward and the friction component was also minimised.

Load cells

In between the pile and the pile cap a brass load cell was placed to measure the load carried by each pile. These load cells were fabricated by cutting a brass cylinder. The middle portion of the load cell consisted of two very thin legs on which electrical strain gauges were mounted. The load cells were calibrated by direct loading method within a range of load varying from 0 to 25 kg.

Instrumented pile

An instrumented pile was made by mounting 9 number of strain gauges on the surface of one model pile at intervals of 10 cm along its length. This was used as one of the piles in the group during each test, in order to obtain distribution of pulling load along depth of pile for a particular location in the group. The instrumented pile was calibrated for load ranging for 0 to 20 kg by direct loading method.

Ennore sand

Ennore sand obtained from Tamil Nadu, Madras was used as the foundation medium. Its engineering properties were found out which are as follows :

Uniformity coefficient = 1.1
Specific gravity = 2.64
Density = 1.62 t/m^3
Relative density = 0.88, and
Angle of internal friction (ϕ) = 37°.

The engineering properties of Ennore sand and its grain size distribution curve is given in Fig. 4. The technique of sand placement for preparation of the sand bed for tests is described below :

> The technique of sand placement plays an important role in the process of achieving reproducible density. The reliability of results would definitely depend upon the uniformity of density of the foundation medium. After proper placement of the single pile/pilegroup under test sand was poured continuously by rainfall

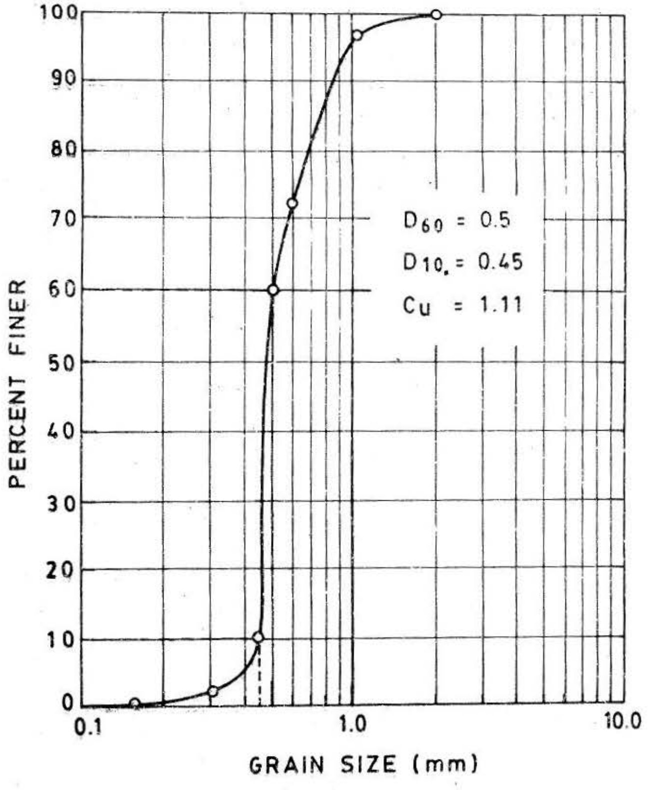

FIGURE 4 : Grain Size Distribution Curve of Ennore Sand

technique through the slit of a hopper keeping constant 60 cm height of fall of sand in order to achieve uniform density. Before starting the test programme the same height of fall was used to fill up a calibration chamber of size 21.45 x 20.6 x 11.9 cm depth. A uniform density of 1.62 t/m^3 was achieved in the calibration chamber even with a good number of repetitions. The constant height of fall was maintained by using a small weight hanging from a thread, acting like a pulmbbob. However, there is possibility of variation of density within the group because of the interference of piles.

Test Programme

A total number of 57 tests were carried out by varying different parameters as mentioned in Table 1.

TABLE 1
Parameters for Tests with Pile Groups

No. of piles in a group	Pile spacing (cm)	Embedded length of pile (cm)	Pile arrangement
1	–	60, 75, 90	single
2	7.5, 10, 12.5	,,	line (1x2)
3	,,	,,	line (1x3)
3	,,	,,	Triangular
4	,,	,,	Square (2x2)
6	,,	,,	Rectangular (2x3)
9	,,	,,	Square (3x3)

Test Procedure

All the model pile tests were carried out following the same procedure described here. When piles were fixed in position alongwith the pile cap and load cells, the arrangement looked like as shown in Fig. 1. The pile cap with the puller was fixed to the tension proving ring attached to the screwjack. The piles with the load cells fitted at the top were then attached to the pile cap with the help of brassballs (nuts), while the threaded head of the load cell passed through the holes made earlier suiting to the requirement of the group arrangement. The verticality of the piles were then checked by pulumb bob. With the pile/piles in position sand pouring was started by rainfall technique to achieve the required placement density of 1.62 t/m^3. At each 10 cm, interval along the length of pile, vermicellis were placed around the pile/pile-group for observing the failure plane after the test. To maintain the proper spacing between the piles, wooden spacers were inserted in between the piles at different levels. With the progress of sand filling the spacers were removed when sand reached the spacer level. When sand pouring was complete the top surface was made levelled. The dial gauges were then fixed. The test was then started. The uplift load was applied gradually in stages. At every stage, movement of the pile/pile-group was recorded by dial gauges. The strain gauge readings of load cells and the instrumented pile were recorded by digital strain indicator. The test was continued till failure.

After the test was over, all the attachments of the top including the pile cap was removed. The aluminium boxes were then taken out one by one. After taking out one box, the sand was removed slowly, until the broken ends of the vermicellis at that level could be seen. The distances of

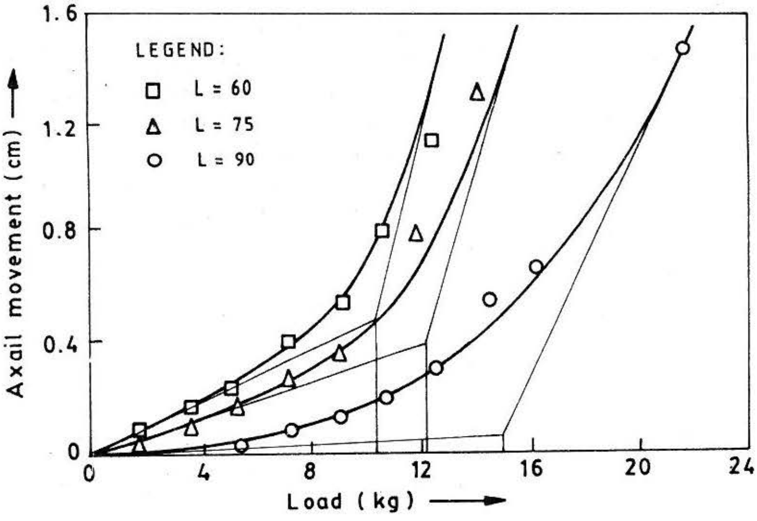

FIGURE 5 : Axial Movement vs. Uplift Load Diagram (Single Pile)

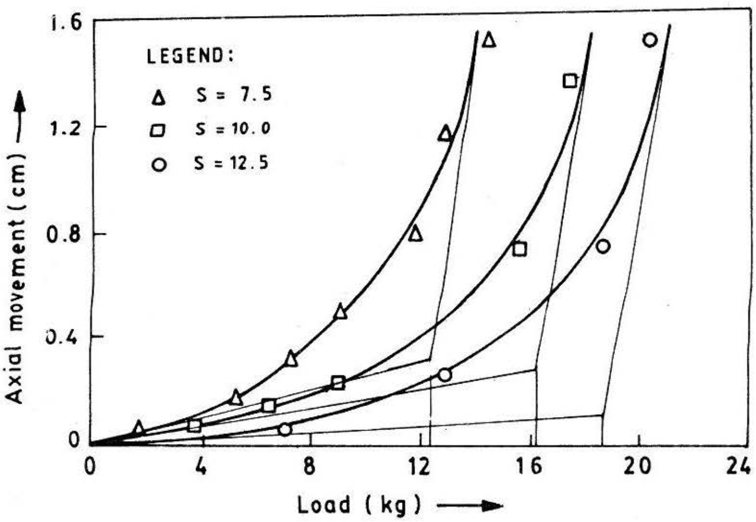

FIGURE 6 : Axial Movement vs. Uplift Load Diagram (Group Size : 1 × 2, L = 60 cm)

the broken ends of vermicellis were then measured from the outer periphery of the pile/pilegroup. This was continued, till all the boxes were taken out except the last one. At a particular level, the average distance of the broken ends of vermicellis from the outer periphery of pile/pilegroup was considered to be representative extent of the failure surface at that particular level. Uplift load vs. axial movement of the pile group relationships were plotted (shown in Figs. 5 and 6). The failure surfaces for different pile groups were also plotted and shown in Figs. 14 and 15.

Test Results and Discussion

Ultimate Uplift Capacity

For each test the axial movement of the pile/pile group recorded by dial gauges have been plotted against applied uplift load measured by the proving ring. Such typical graphs are shown in Figs. 5 and 6. It has been found that, for each test the curve becomes ultimately asymptotic to the movement axis. The ultimate load has been obtained by double tangent method. In 70% of the cases the load settlement curves were curvilinear throughout the ranges except for very small portion at the end and hence it was difficult to pin point the ultimate load. So double tangent method is resorted to (only in the case of sudden failure the load settlement curve is asymptotic to the movement axis, in which case the ultimate load can easily

TABLE 2
Ultimate Uplift Capacity in Kg

	Embedded Length of Pile								
	L = 60 cm			L = 75 cm			L = 90 cm		
Single Pile	10.5			12.2			15.0		
Arrangement of pile in Group	Spacing			Spacing			Spacing		
	7.5	10.0	12.5	7.5	10.0	12.5	7.5	10.0	12.5
1 × 2	12.3	16.2	18.6	14.0	18.3	21.5	17.0	22.0	26.1
1 × 3	16.7	21.3	25.9	18.3	24.5	29.7	22.1	29.1	36.0
Triangle	17.5	23.1	27.4	19.9	26.5	31.7	24.0	32.0	38.5
2 × 2	21.6	26.9	33.6	23.6	32.4	38.5	28.4	37.2	46.8
2 × 3	29.7	38.4	47.2	33.8	43.5	53.5	39.8	52.0	65.2
3 × 3	43.0	55.8	68.5	47.5	63.7	78.2	57.5	75.6	94.5

TABLE 3
ρ / B* and ρ / S* Ratios for Different Groups

Group	ρ / B ratio	σ / S ratio
1 × 2	0.28 – 0.48	0.056 – 0.141
1 × 3	0.32 – 0.62	0.064 – 0.144
Triangular	0.06 – 0.21	0.075 – 0.288
2 × 2	0.10 – 0.20	0.128 – 0.266
2 × 3	0.036 – 0.12	0.044 – 0.16
3 × 3	0.044 – 0.09	0.096 – 0.21

ρ = axial movement of the group
B = group size
S = pile spacing

be determined by single tangent method). For all the tests the ultimate uplift capacities have been presented in Table 2. It has been observed that, the ultimate uplift capacity increased with spacing for a particular embedment length of piles, in case of a particular arrangement of group. It increased with embedded length for a given spacing and for a particular group arrangement. The variation of axial movement (ρ) to group width (B) ratio and this movement (ρ) to spacing (S) ratio for different groups at the time of failure is presented in Table 3. In general, for line groups the relative movements are found to increase with number of piles. This effect has been noticed in case of relative movements calculated either on the basis of minimum dimension in cross section or spacing. While the trend noticed is as above in terms of non-dimensional quantities the absolute values indicate no definite trend. However, it is difficult to explain the reasons of such variations.

The group efficiencies have been obtained by dividing the group capacity (Q_g), by number of piles (n) times single pile capacity (Q_s), without any consideration of displacement. The efficiency vs. (L/D) / (L/B) plots for various S/D ratio (L = embedded length, D = pile diameter, B = group width, S = spacing) have been presented in Fig. 7. The figure shows that efficiency (n) decreases with L/D but increases with S/d for a particular group arrangement. At higher spacing, the efficiencies are almost same for line and other groups because the piles may tend to act individually. At closer spacing line groups have been observed to be more efficient as far as uplift is concerned.

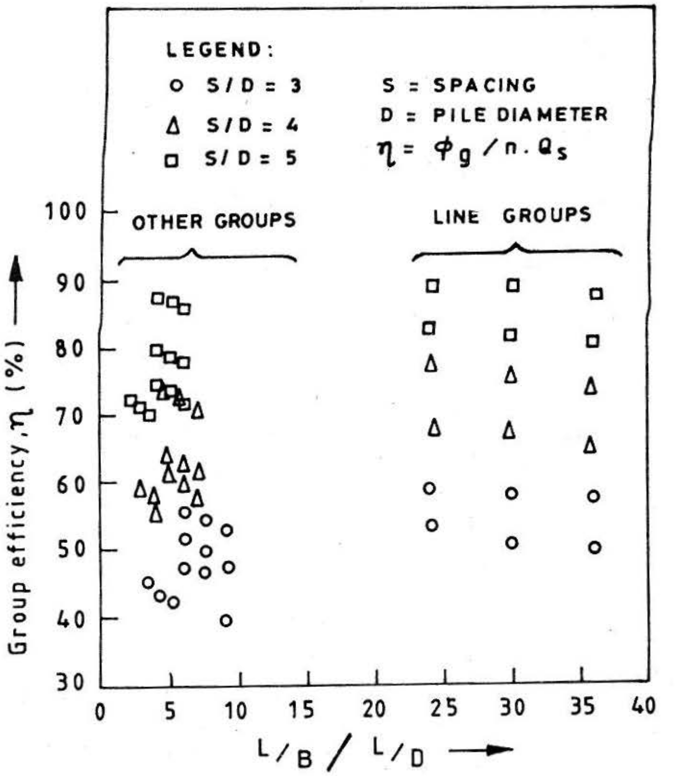

FIGURE 7 : Axial Movement Vs. Uplift Load Diagram (Group Size : 1 × 2, L = 60 cm)

Transfer of pulling load

With the help of instrumented pile the load carried by the piles at different levels is found out and typical load distribution is represented by Figs. 8(a) and 9(a). Typical variation of load shared by the soil i.e. load transferred to the soil at different levels is indicated in Figs. 8(b) and 9(b). It may be observed that, major portion of the load has been transferred to the soil at the upper reaches of the pile with reduction in the downward direction. Figures 8(a) and 9(a) represent the plots of loads along the axis. The entire embedded depth has been divided into segments as indicated in the figure. The mid level of each segment is indicated by a line. Between any two sections (i.e. top and bottom of any segment) the difference in the ordinates of the cumulative load diagram indicates the amount of load transferred from pile to the soil. The relevant loads are plotted in Figs. 8(b) and 9(b) at the middle level of the segments. Figures 8(b) and 9(b) have been drawn to represent the distribution of load transferred to the soil along the depth.

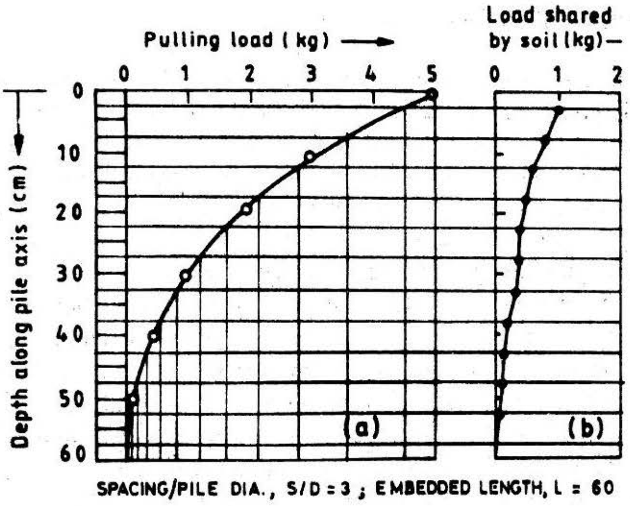

FIGURE 8 : Distribution of (a) Pulling Load along Pile Axis
(b) Load shared by Soil
(Group Size : 2 × 3)

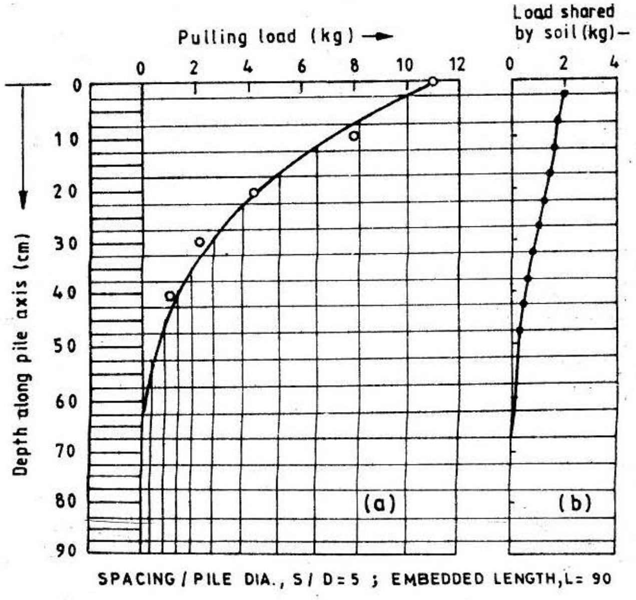

FIGURE 9 : Distribution of (a) Pulling Load along Pile Axis
(b) Load shared by Soil
(Group Size : 1 × 2)

Load shared by individual piles in a group

Typical sharing of load by individual piles at different movement levels in a group is presented in Fig. 10 to 13. It was observed that for 1×2, 2×2 and triangular groups all the piles of a group carried equal loads and hence an average graph has been drawn. With larger spacing, individual piles of these groups were found to carry higher loads. For 1×3, 2×3, 3×3 groups the corner piles were found to carry higher load with the progress of the test. At the beginning, all the piles carried almost equal loads. The lesser load carried by the internal pile might be due to mobilisation of less shear strength, because the relative movement of the enclosed sand with respect to the central pile is likely to be less, when compared to the sand surrounding the group and the corresponding shear mobilisation. In addition, the reduced capacity of the central pile seems to be as a result of interaction between the piles in a group. The load carried by the individual piles of a group is expressed in percentage of the group load at different movement levels and indicated in the figures (Fig. 10 to 13). In general, percentage of load carried by internal piles were found to decrease and that for external piles were

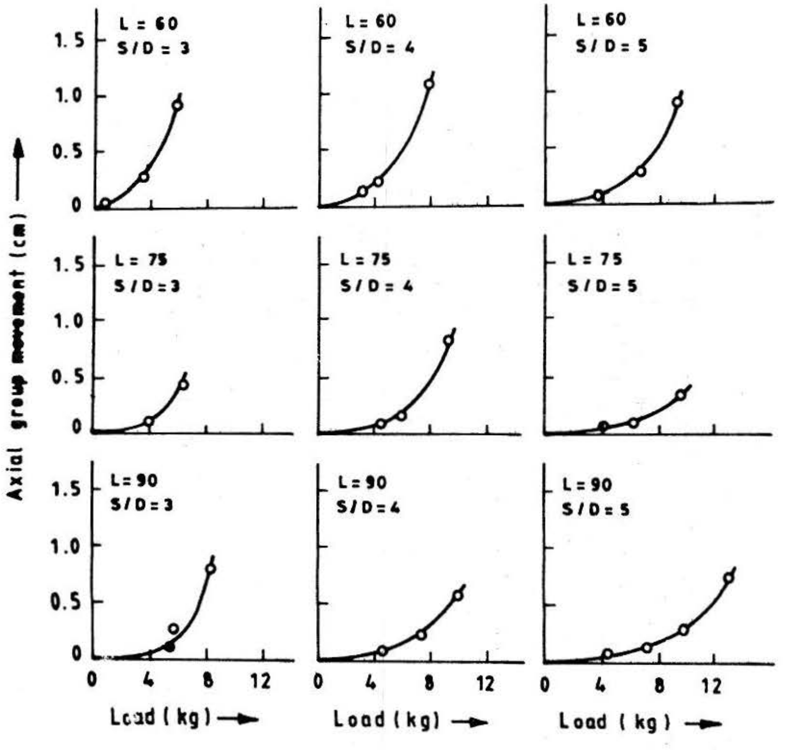

FIGURE 10 : Load Shared by Piles in a Group (Group Size : 1 × 2)

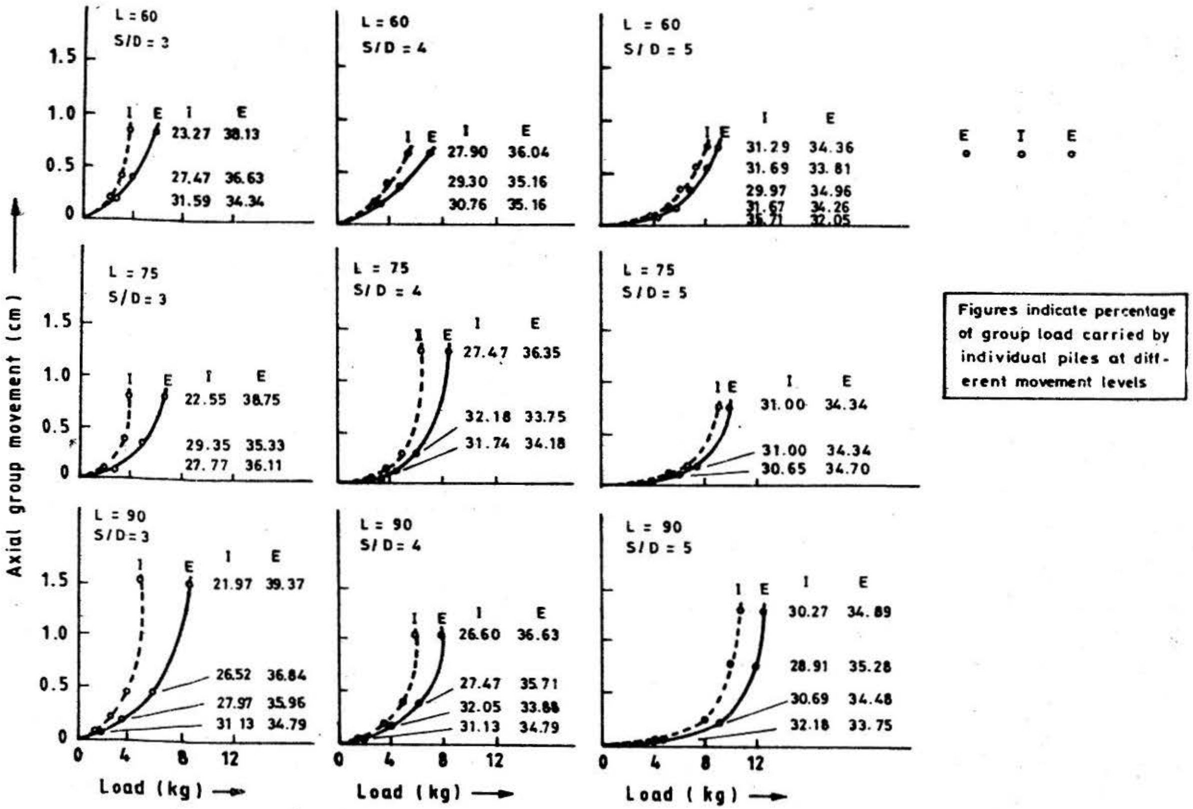

FIGURE 11 : Load Shared by Piles in a Group (Group Size : 1×3)

found to increase with the progress of the test. In the case of 3×3 pile group the percentage of load taken by central and middle piles are found to become more and more with spacing. But the corner piles are found to share lesser percentage of load with increase in spacing. With increase in embedment length though the total load taken by the group increases, there is no perceptible change in the load shared by each of the piles in the group. In the case of line group (1×3) the middle pile is found to take up more and more load with increase in spacing while the end ones are sharing less and less load. With increase in embedment length, though the total load taken by the group increases, no perceptible change in the load shared by each pile of the group is observed.

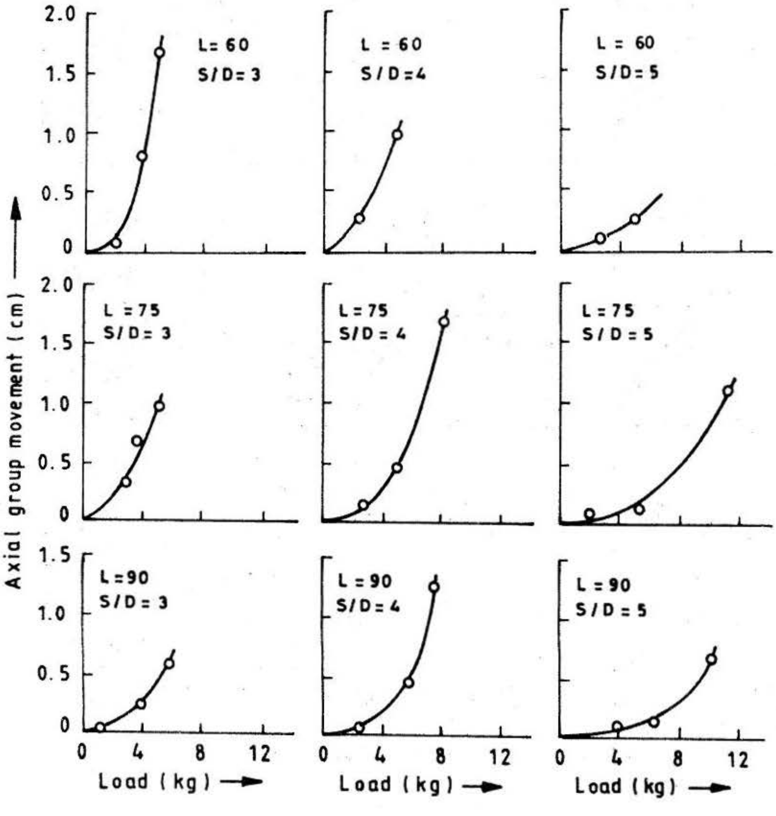

FIGURE 12 : Load Shared by Piles in a Group (Group Size : 2 × 2)

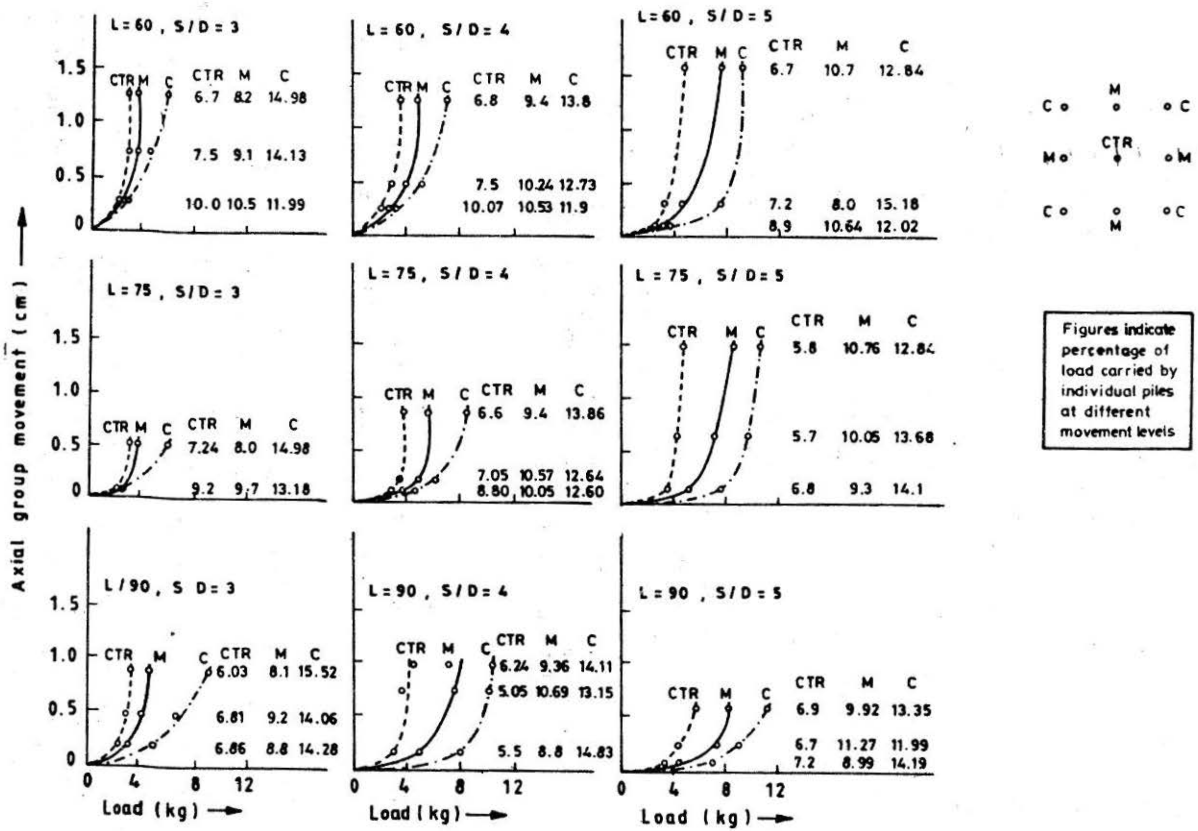

FIGURE 13 : Load Shared by Piles in a Group (Group Size : 3 × 3)

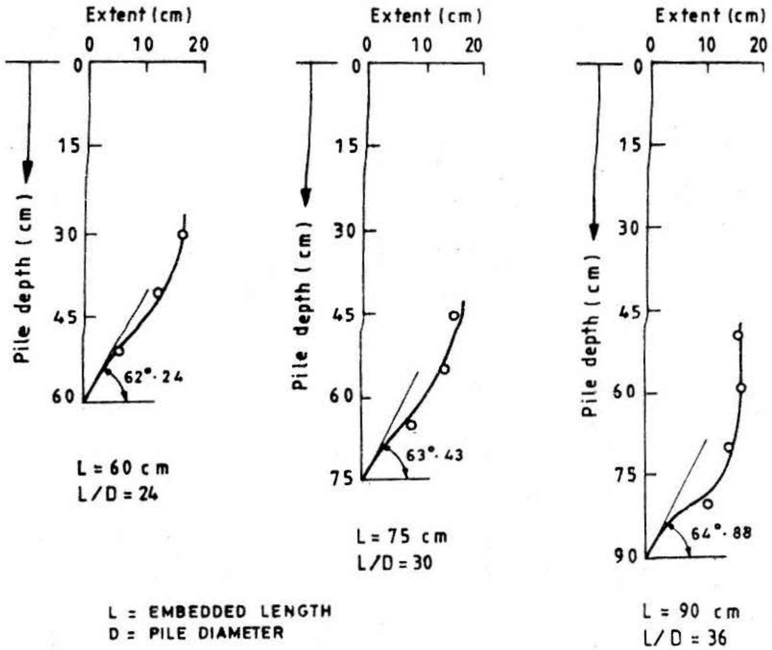

FIGURE 14 : Failure Plane Developed for Single Pile (Note : Extent is the Average of Distances of Breaking Points of Vermicellis from the Periphery of the Pile)

Failure surface

A few typical failure surfaces as obtained from tests have been shown in Fig. 14 and 15. The following salient features could be observed in respect of the failure surface.

(a) As L/D ratio became less than 6, the failure surface tended to reach the surface with a convex curvature towards the pile. For L/D more than 6, the curvature appeared to be just the opposite and they were not found to reach higher levels.

(b) All the failure surfaces started near about the pile tip with an angle of, approximately $(45° - \phi/2)$ (ϕ = angle of internal friction of soil) with the vertical. This indicates that the direction of major principal stress is nearly vertical and along the axis of the pile. It might also be inferred that since all the angles were not exactly $(45° - \phi/2)$, there remained a possibility that coefficient of earth pressure (K) might have changed during the test. However, this aspect needs further investigation.

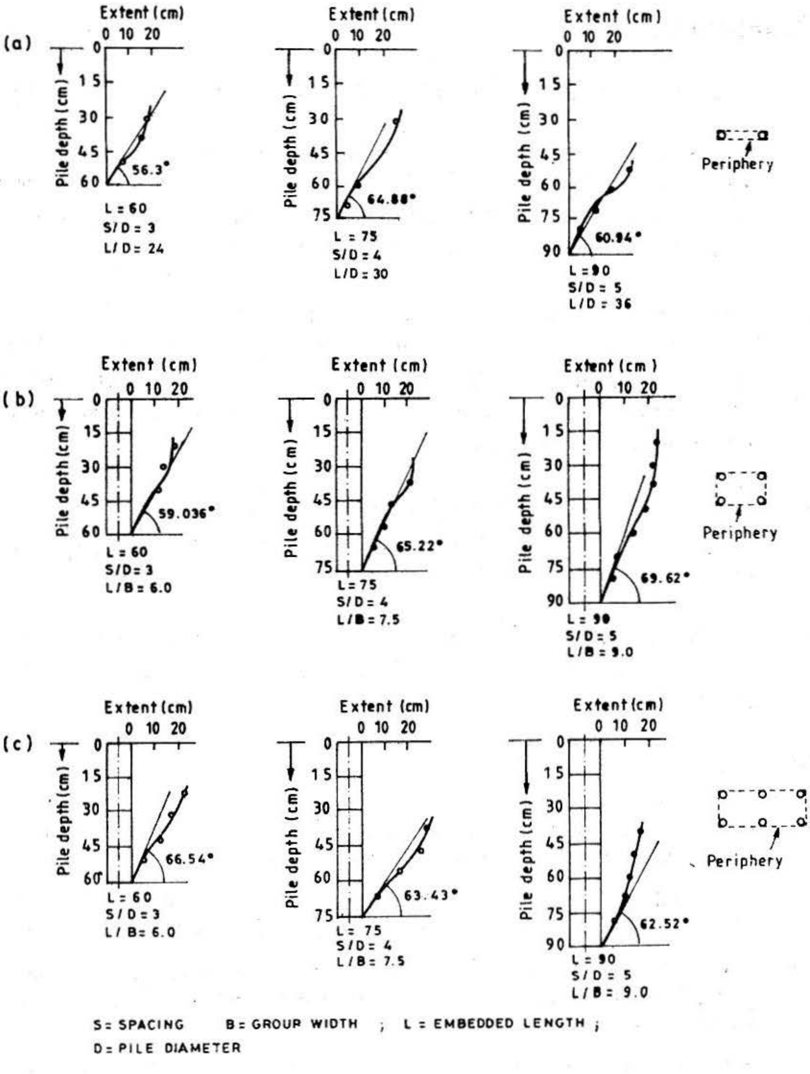

FIGURE 15 : Failure Planes for Pile Groups
(a) Group Size : 1 × 2
(b) Group Size : 2 × 2
(c) Group Size : 2 × 3
(Note : Extent is the Average of Distances of Breaking Points of Vermicellis from the Periphery of the Pile)

Conclusions

The following conclusions may be drawn from the present study.

1. *For a particular embedment length of piles and a particular group* arrangement, the ultimate uplift capacity of piles increases with spacing and it increases with embedded length of piles for a given spacing, in case of a given arrangement of piles in a group.

2. The group efficiency decreases with increasing (L/D) / (L/B) but increases with increasing S/D or S/B for a particular group arrangement (L = embedded length, S = spacing, D = diameter of piles, B = minimum dimension of the group in cross section) within the range of test parameters.

3. In a group, the central piles carry the least load and the corner piles carry the highest load at failure, although in the beginning all the piles carry almost equal loads.

4. All the failure surfaces start near the pile tip approximately at an angle of $(45° - \phi/2)$ with the vertical, indicating that the direction of major principal stress is nearly vertical.

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