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SYNOPSIS: Lateral loading tests on instrumented model piles in saturated sand in a centrifuge are described. Piles were tested in isolation and in groups of two, three and six piles. Data were obtained on deflections and bending moments in single piles and on interaction factors between piles. Response of single piles compared favourably with data from Mustang Island field tests. Experimental interaction factors generally differed from those commonly used which are derived from elastic theory. The difference depends on pile spacing.

INTRODUCTION

One of the major technical problems facing the offshore oil industry is the design of piles to resist wave loading on offshore structures. With wave heights approaching 100 ft, large cyclic lateral components of load can be expected in addition to the already great vertical components. Previous experience in designing piles under lateral load has been confined mainly to relatively small piles on land and the extrapolation of current design techniques to meet the requirements of large piles and severe offshore conditions involves many uncertainties. There is an obvious need for development of better analytical methods based on experimental data from full scale tests on piles.

This paper demonstrates the use of centrifugal modelling techniques to provide experimental data on piles under lateral loading, under controlled conditions and with adequate instrumentation.

Although much valuable information can be gained from a full scale testing programme, data from one installation cannot be applied easily to other sites and the cost is so high as to preclude its being carried out on a regular basis. The alternative of testing large piles on land is still costly, and the loading conditions expected offshore cannot be reproduced easily. Additionally, soil conditions in the field are often uncertain and the range of parameters which can be varied is restricted.

It is far preferable to carry out a full parametric study under controlled conditions for a variety of soil conditions, pile sizes and loading configurations. Only in this way can a consistent set of design rules be developed. The modelling of piles at small scale in the laboratory allows better control over soil and pile parameters at much lower cost. However, standard laboratory testing techniques cannot take full account of the insitu stress conditions of the soil and hence do not model correctly non-linear soil-pile interaction. The technique of centrifugal modelling of geotechnical structures can be used to test a model at a small geometric scale while simulating the same stress history in the soil as would occur in the corresponding full scale prototype. This method provides, therefore, a low cost technique for studying the behaviour of piles under lateral loading at small scale in a controlled reproducible soil medium, while retaining the non-linear pressure-dependent features of soil response. A centrifugal model test can be regarded as an event in itself, or the centrifugal modelling laws can be applied to interpret the results in terms of a case history of a prototype at a scale equivalent to a pile offshore.

The objectives of this study included the identification of fundamental mechanisms of soil-pile interaction for long flexible piles in dense saturated sand, and a parametric study of some factors influencing pile response to static and cyclic loading.

TEST PROCEDURE

Model pipe-piles were tested under static and slow-cyclic lateral loading in the Cambridge University Geotechnical Centrifuge. Details of the centrifuge and the principles of centrifugal modelling are given by Schofield (1980).

A strain-controlled lateral loading system was adopted (Barton, 1982), and is shown in Fig. 1. Instrumentation was provided to measure lateral load, deflection and rotation at the pile head and bending moments at several points down the length of the pile. The notation adopted for the experimental data is shown schematically in Fig. 2. The model aluminium alloy piles ranged in diameter from 9.5 mm to 16 mm, and were tested at centrifugal accelerations in the range 30 g to 120 g. The

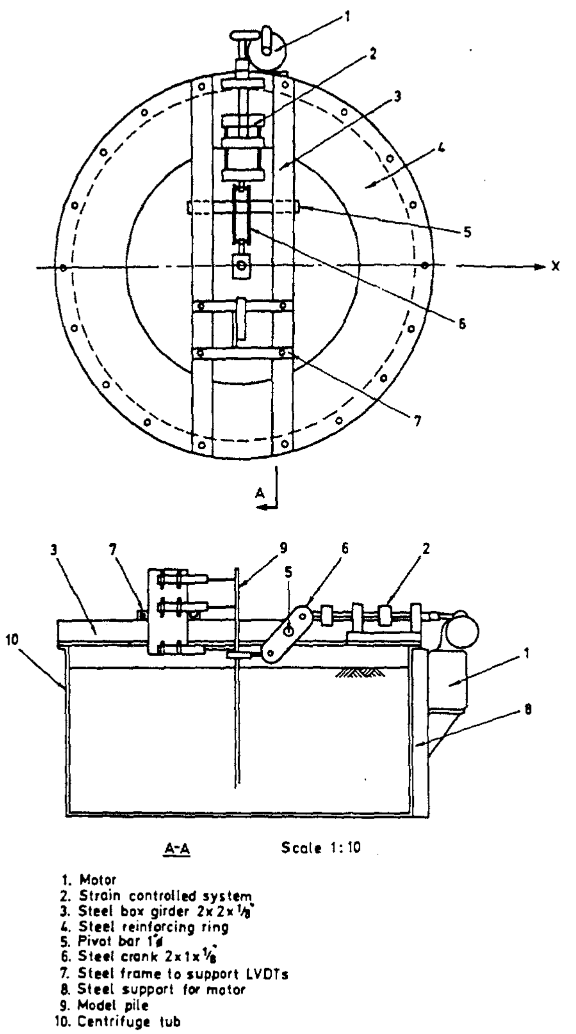

FIG. 1 Strain Controlled Apparatus - General Arrangement.

loading system was designed to accommodate pile groups of up to six piles.

The sand samples used were all of fine sand prepared under water to a relative density of 79% and peak friction angle of 43°. The samples were contained in circular steel tubs of 850 mm diameter and 400 mm depth. Saturated soil conditions were maintained in all tests, and the influence of water table location was investigated by comparing tests where the soil surface was inundated with free water with those where the water table was drawn down below the pile toe.

MUSTANG ISLAND FIELD TESTS

To establish the validity of using centrifugal model data to predict the behaviour of full scale events, it is useful to make comparisons with field data. Cox, Reese and Grubbs (1974) have carried out full scale tests on single 24 in diameter steel piles driven into a fine uniform sand deposit at Mustang Island. The

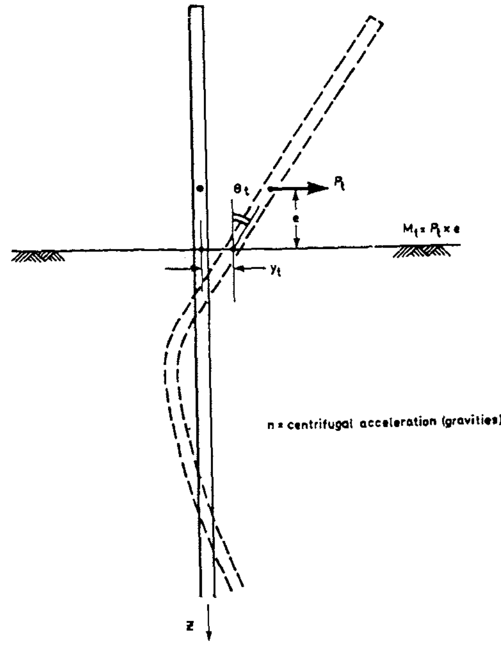

FIG. 2 Notation for Centrifuge Data from Tes on Single Model Laterally Loaded Pile

sand was of grading 120-200, had a relative density of about 0.8, a friction angle of 39° and was saturated. Static and slow cyclic lateral loads were applied to two piles and measurements taken of pile head deflection an rotation, and of bending moments down the piles.

The results from this test series have formed the basis of the American Petroleum Institute (API) recommendations for the design of laterally loaded piles in sand. In particular, th currently accepted method for computing data for p-y analysis has been developed from empirical relationships derived from these tests (API, 1982).

In order to compare the results of centrifugal modelling with those from full scale piles, model parameters, including soil properties and pile geometry, have been chosen to achieve similarity between the Mustang Island tests and centrifuge model.

CENTRIFUGAL MODEL TESTING

If results from a parametric study are valid, then the centrifugal modelling laws must be obeyed. It is important to ensure that model events scale correctly with each other before the results can be interpreted with confidence

To demonstrate this, cyclic lateral load tests were carried out on three different sizes of model pile at respective centrifugal accelerations to represent the same full scale event in each case. Table I lists the model and equivalent prototype test configurations for each case.

TABLE I

ACCELERATIONS	MODEL		PROTOTYPE	
N Gravities	Diameter (mm)	EI (Nm^2)	Diameter (mm)	EI (Nm^2)
40	15.875	95.5	635	$245x10^6$
50	12.70	42.8	635	$268x10^6$
66.7	9.525	14.4	635	$277x10^6$

By scaling load with N^2 and deflection with N, the test data for the appropriate g-levels can be plotted as prototype load versus prototype deflection, as shown in Fig. 3. The correlation between the results is good and verifies that the modelling technique is self-consistent and obeys the scale modelling laws. It should now be possible to apply the model test results to full scale events.

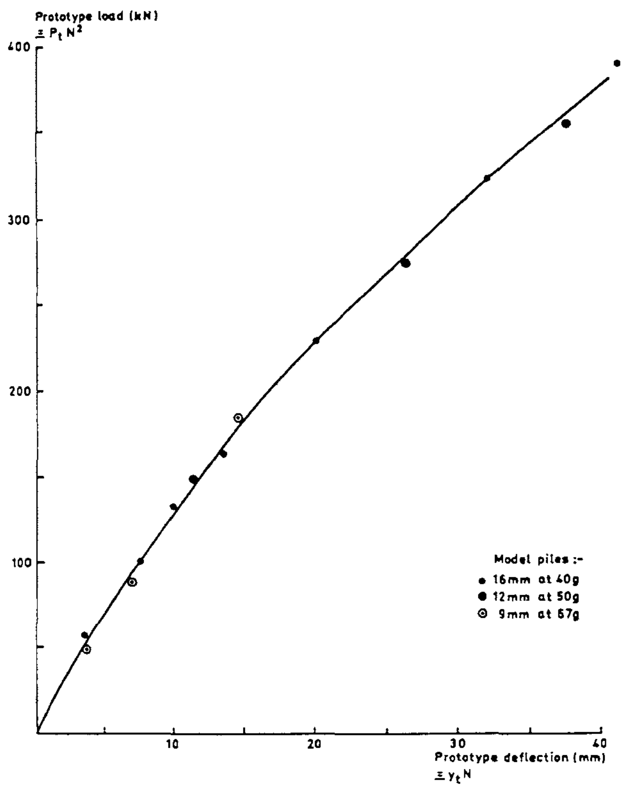

FIG. 3 Prototype Scale Load-Deflection Relationships for 10th Load Cycle.

The model chosen to represent the Mustang Island field test was that of a 15.875 mm diameter pile at a centrifugal acceleration of 38 g. Table II shows the pile properties for the field tests, the model and the equivalent prototype for the model. Note that the model pile diameter scales to within 5% of the full scale pile, but the equivalent pile stiffness is 9% too high. The scaled model tests results have been adjusted slightly to take account of this discrepancy.

Comparing results from cyclic loading tests,

TABLE II

QUANTITY	FULL SCALE (Cox et al, 1974)	MODEL at 38g	PROTOTYPE
Outside Diameter (mm)	609	15.875	605
Inside Diameter (mm)	590	13.92	529
Young's Modulus (GN/m^2)	210	69	69
EI (N/m^2)	$170x10^6$	88.9	$185x10^6$
Eccentricity of Load (mm)	325	15	570

Fig. 4 shows prototype load versus prototype displacement for the 10th cycle of loading.

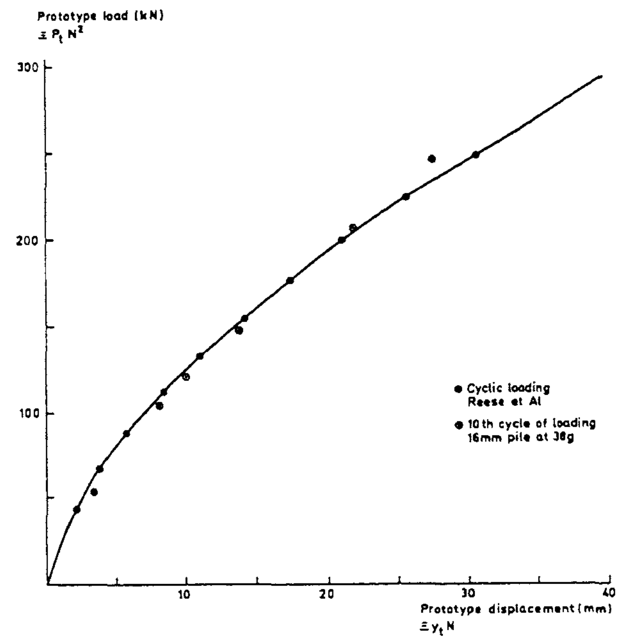

FIG. 4 Comparison between Load-Deflection Relationships for Field Tests and Centrifugal Model.

The correlation between field test and centrifugal model prototype is quite close. Prototype load and pile head rotation values can also be seen to agree well in Fig. 5. The measured bending moment distributions for cyclic loading are shown in Fig. 6, for a prototype lateral load of 244 kN. Note that at prototype scale, the lateral load in the model tests was applied at prototype eccentricity which was 250 mm higher than in the field tests. If the model test bending moment distribution is displaced by this amount, the curve coincides with that of the field data.

This comparison between centrifugal model and full-scale test demonstrates that scale modelling can give a reasonable representation of events in the field. The validation of the experimental method also implies that parametric studies on model piles in the centrifuge can be a viable alternative to performing

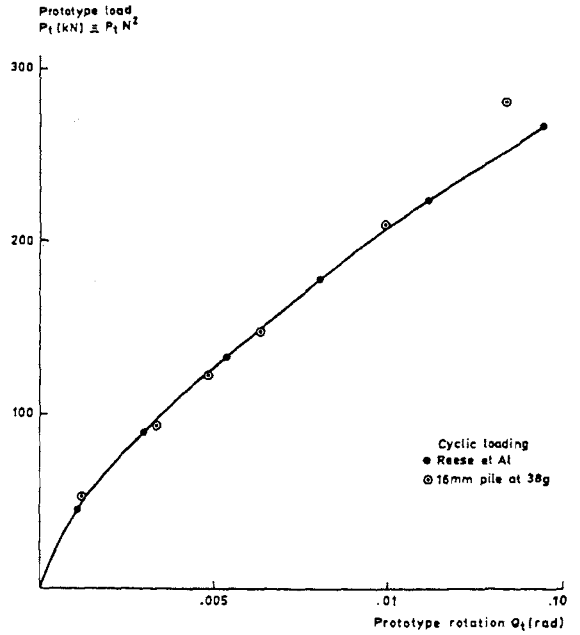

FIG. 5 Comparison between Load-Rotation Relationships for Field Test and Centrifugal Model.

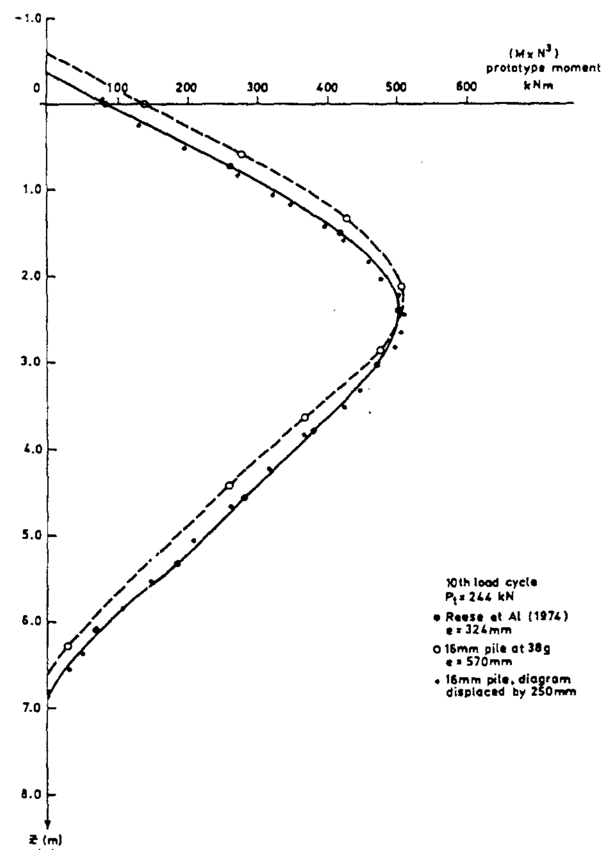

FIG. 6 Comparison between Measured Bending Moment Distribution for Field Test and Centrifugal Model.

extensive large-scale tests in the field.

Further comparisons between model and prototype can be drawn from qualitative observations of soil-pile interaction. A descripti of the fundamental mechanisms governing lateral pile response to cyclic loading has bee obtained from observations of model piles in the centrifuge. During the initial half cyc of load it was noted that soil in front of t pile undergoes shearing and rupture. Behind the pile, soil cannot sustain tensile stress and so the pile breaks away leaving a cavity On reversal of the loading direction, if fre water is present, ruptured material now becomes liquefied and flows into the cavity around the pile, later becoming compacted wh the pile reverses direction. With subsequen load cycles, more material flows down around the pile and becomes densified at some depth below the surface. The result is that the lateral response becomes stiffer with number of cycles until a steady state is reached, typically within 5 to 10 cycles for collinea load cycles. Individual cycles of load-displacement response are not symmetrical an on removal of load there are residual bendin moments and displacements induced in the pil

In the absence of free water, the cyclic response is slightly different. The same features of rupture and tensile yielding are observed in initial loading, but with cyclic loading the soil can sustain a more nearly vertical face when the pile moves away and nc material flows in behind it. This results ir an increase in lateral deflections during the first few cycles, but the response then reaches a steady state after approximately 2C cycles. The cyclic response is more symmetri than when free water is present.

Figure 7 shows characteristic patterns of soi erosion around cyclically loaded model piles,

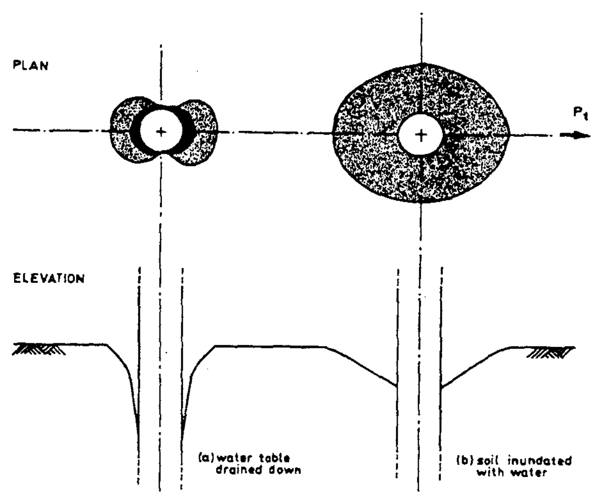

FIG. 7 Erosion Zones around Single Piles after Cyclic Loading.

with and without free water present. Reese (1979) has described the observation of a gap developing around the heads of piles during the Mustang Island field tests. With cyclic loading, water was seen to be pumped up and down in this gap carrying with it eroded soil particles.

PILE GROUPS

A further illustration of the use of centrifuge tests for providing case history equivalent prototype data is the study of interactions between piles within a group. Barton (1982) carried out centrifuge tests on two, three and six pile groups using a modified form of the centrifugal apparatus described above for single piles. Interaction factors derived from these tests for two pile groups will serve here as an illustration. Furthermore, the deduced interactions between pile pairs will be applied to the prediction and analysis of larger groups.

The degree to which piles in a group interact is generally quantified by an interaction factor, a concept introduced by Poulos (1971). It is well known that under lateral loading, a group of piles tends to deform more, for a given average load per pile, than a single pile. This is because a pile displaces not only under its own loading but also due to that caused by soil displacements resulting from the movements of other piles in the vicinity.

The interaction factor is defined as the fractional increase in deflection of a pile due to the presence of a similarly loaded pile in the proximity. For two piles, if a single pile deflects y_o under a given load, then the deflection of each pile under the same average load per pile is:

$$y = (1 + \alpha)y_o \tag{1}$$

The factor α will depend on the pile and soil properties, the loading configuration and the relative orientation of the piles. As shown in Fig. 8, the centre-to-centre spacing between the piles is defined as s and the angle between the line joining the piles' centres and the direction of loading is known as the departure angle, β. Analytic solutions for the interaction between two piles in a linear elastic homogeneous continuum have been presented by Poulos (1971) and by Randolph (1977).

The lateral response of a multi-piled group can be approximated simply if the principle of superposition can be assumed to hold, that is, if the increase in displacement of a pile due to all the surrounding piles can be calculated by summing the increase in displacement due to each pile in turn, using the interaction factors for two piles. Thus, for an n-pile group, the interaction factor obtained by superposition is given by:

$$\alpha = \alpha_2 + \alpha_3 + \alpha_4 \ldots + \alpha_n \tag{2}$$

in which α_2, α_3 ... α_n are the values of the appropriate interaction factor for pile 1 due to piles 2, 3 ... n, for the respective spacings and values of β between each of these

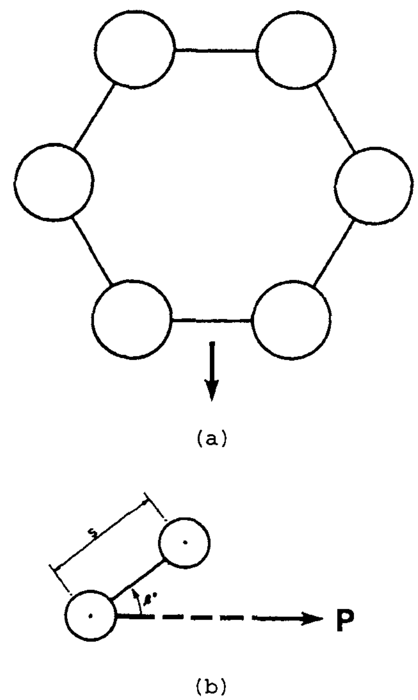

FIG. 8 (a) Configuration of six-pile group, (b) Configuration of two-pile interaction.

piles and pile 1. If the load is shared equally between piles, then the ratio of the displacement of the group, y_G, to the displacement, y_1, of a single pile carrying the same load as a pile in the group is then:

$$y_G = (1 + \alpha)y_1 \tag{3}$$

But for equal displacements of each pile in the group, if the total load on the group is P_G then:

$$P_G = \sum_{j=1}^{n} P_j \tag{4}$$

and the displacement of any pile i in the group is, by superposition:

$$y_i = \bar{y}\, P_i + \sum_{\substack{j=1 \\ j\neq i}}^{n} P_j \alpha_{ij} \tag{5}$$

in which $\bar{y}$ is the unit reference displacement of a single pile under a unit lateral load and α_{ij} is the appropriate interaction factor between two piles, i and j. Similar expressions can be written for pile head rotation and moment loading.

Tests on pile groups were designed to investigate the interaction between pairs of piles within a six-pile group of the configuration shown in Fig. 8. The six piles are arranged at two-pile diameter centres in a hexagonal pattern; a format typically used to support the legs of an offshore structure. The programme of tests on pairs of piles investigated the cyclic loading response of piles at 2, 4 and 8 diameter centres and oriented at 0^o, 45^o and 90^o to the direction of loading. The same pile sizes and centrifugal accelerations were used as for the single pile tests so that proper comparisons could be made. One further test was carried out on a three-pile group. All tests consisted of six cycles of the two-way loading at an amplitude of approximately 80% of the full lateral load capacity.

The lateral response of large, closely spaced pile groups offshore can be critical for the offshore engineer. The performance of steel jackets founded on piles can be strongly affected by pile group stiffness, not only during storm conditions but also during long-duration fatigue wave loading. Current offshore design methods rely on elastic methods to predict pile group interactions, even when a non-linear approach is used to determine the behaviour of single piles. It is important to be able to check the validity of elastic analysis for determining interaction factors between pile pairs and to verify that the principle of superposition holds for combining pile pairs to predict behaviour of larger groups.

Typical test results for pairs of piles under lateral load are shown in Figs. 9 and 10, together with a comparison with the response of a single pile under similar test conditions. It should be noted, firstly, that

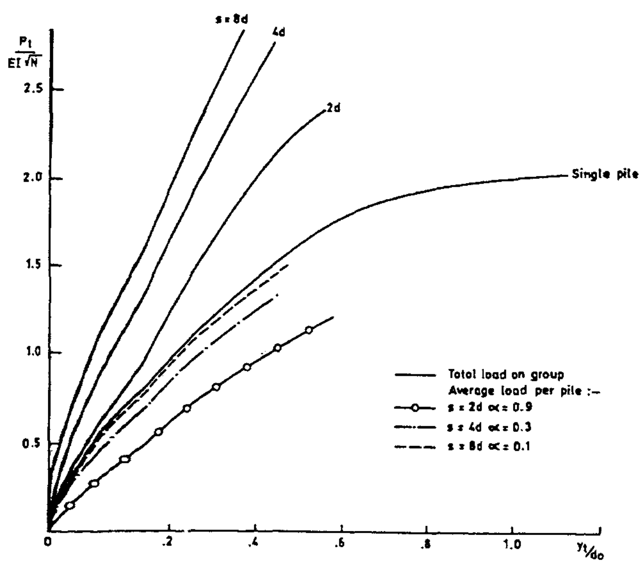

FIG. 9 Scaled Load-Deflection Relationships for Pile Pairs at Different Spacings; 12.7 mm Piles at 40 g, $\beta = 0^o$.

lateral group response is distinctly non-linear, and exhibits similar characteristics to single pile behaviour. However, there are obvious trends in group response; group interaction can be seen in Fig. 9 to decrease with increasing pile spacing, and in Fig. 10 to decrease as the orientation angle, β, increases from 0^o through to 90^o. Moreover, the group interaction factor, calculated for the average load per pile and compared with the single pile, tends to be consistent over a

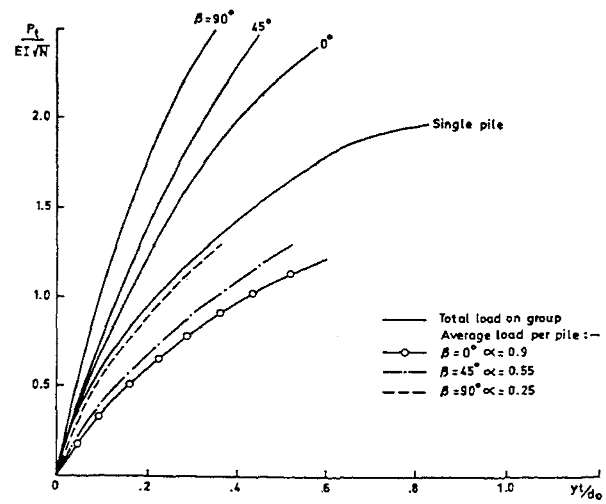

FIG. 10 Scaled Load-Deflection Relationships for Pile Pairs at Different Orientations; 12.7 mm Piles at 40 g, s = 2d.

large portion of the response.

Qualitatively, all tests showed trends in group interaction which were consistent with predictions from elastic theory. However, quantitative values of interaction factors were seen to be influenced by non-linear soil response. One important effect was seen in the distribution of load within a group; for example, elastic theory predicts that within a pile pair, both piles should carry identical loads, whereas experimental data would indicate that the portion of total load on the piles varies with both orientation angle and spacing. A study of yielding patterns and erosion zones around pile heads gives strong indications of the mechanisms of non-linear soil behaviour which affect group interaction.

At close spacing, erosion zones around each pile head overlap (Fig. 11), each zone having a similar form to that of single piles, described previously. The degree of interaction is broadly related to the extent to which those zones intersect. Two piles at 2d spacing and orientated with $\beta = 0^o$ were found to share the group load with 60% to the leading pile and 40% to the trailing pile. This is caused by the creation of a tensile yielding zone behind the leading pile, into which the trailing pile moves forward; the reduction in soil resistance afforded to the trailing pile results in a lower load being carried by it. Thus, a more compliant group response is observed than would be predicted when the effect of overlapping yielding zones is neglected.

Conversely, at wider spacings, where the erosion zones do not overlap, it is found that interaction is over-predicted by elastic analysis. Whilst elastic theory describes the displacement field around the pile head decreasing inversely with distance from the pile, the effect of non-linear soil behaviour results in the displacements decaying more rapidly with distance. Thus, experimental

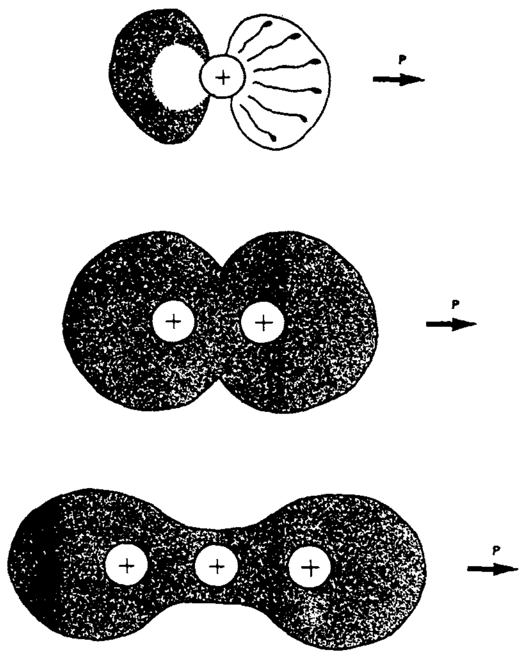

FIG. 11 Erosion Zones around 1,2 and 3 Piles at 40 g, $\beta = 0^{o}$.

pile pairs at spacings of 8d showed negligible interaction, whereas elastic analysis predicts significant interaction at spacings of up to 20 diameters.

A comparison between experimentally measured group interaction factors and those computed by elastic analysis is given in Table III, for 12.7 mm diameter model piles tested at 40 g centrifugal acceleration.

TABLE III

NO. OF PILES	s/d_o	β^o	Experimental α_G	Elastic Analysis α_G
2	2	0	0.654	0.331
2	4	0	0.187	0.173
2	8	0	0.012	0.103
3	2	0	0.854	0.512
6	2	Various	0.638	0.825

Note that elastic parameters have been chosen which give a close correlation with the measured response of a single 12.7 mm diameter model pile at 40 g.

Data gathered from the parameter study on pile pairs can now be applied to the prediction of interactions within larger groups. If the principle of superposition holds for interaction factors when predicting group response, then the experimentally measured values for interactions for pile pairs at 2d and 4d spacings and $\beta = 0^{o}$ should sum to give the measured group interaction factor for the three-pile group. From Table III, the calculated value of α_G is:

$$\alpha_G = 0.654 + 0.187 = 0.841 \qquad (6)$$

and the calculated group displacement:

$$y_G = 3.06 \text{ mm} \qquad (7)$$

The measured value for three piles were $\alpha_G = 0.854$ and $y_G = 3.08$ mm. This would imply that a reasonable estimate of group response can be calculated by superposing experimental interaction factors for the constituent pile pairs.

The prediction of the six-pile group response is more complex because of the variety of configurations of the constituent pile pairs. However, Table IV lists approximate values for interactions for each pile pair based on experimental results.

TABLE IV

PILE NO.	s/d_o	β^o	Experimental α_G	Elastic Analysis α_G
1	–	–	1	1
2	2	30	0.385	0.263
3	3.5	0	0.187	0.172
4	4	30	0	0.131
5	3.5	60	0	0.108
6	2	90	0.063	0.150
Predicted y_G (mm)			2.655	2.92
Predicted α_G			0.635	0.825

The sum of α values predicts a group interaction factor of 0.635 and displacement of 2.655 mm, as compared with measured interaction factor of 0.638 and displacement of 2.66 mm. This correlation gives encouraging support to the method of superposing interactions between experimental pile pairs. Also note that even within a group at these close spacing, piles at opposite sides of the group interact very little.

Also shown in Table IV are corresponding α factors computed by elastic analysis, and this demonstrates that the elastic solution overpredicts the group response. This result is largely due to the prediction of significant interactions between piles on opposite sides of the group. Examination of the erosion pattern around the six-pile group confirmed that erosion did not extend across the centre of the group, but was restricted to local overlapping zones around the perimeter of the group. This would tend to support the deduction that elastic theory does not adequately describe the strain field around a laterally loaded pile.

CONCLUSION

The close comparison between lateral pile response data measured in the Mustang Island field tests and the data from appropriately scaled centrifuged model tests demonstrates the viability of centrifuge testing to explore pile response to lateral loading. The tests clearly show also the great advantage centrifuge testing enjoys in readily allowing the exploration of the effects of pile size, soil conditions and loading regimes while maintaining the stress conditions of the prototype situation. Exploring the effects of these parameters in field tests over any significant range of parameters would be prohibitively expensive.

The test data on pile group response to lateral loading indicate that pile interaction factors obtained from elastic theory do not adequately model pile response in the non-linear range. It appears that, at the close spacing typical of offshore piles, elastic interaction factors underestimate pile response while they overestimate it at larger spacings. The pattern of erosion zones around piles is clearly evident in the centrifuge tests and is an important factor in understanding the deviation of the interaction factors from those predicted by elastic theory.

Case histories play a very important part in the evolution of methods of design and analysis by providing data for checking theories and illustrating phenomena from which design concepts may evolve. The centrifuge test has a major role to play in providing case history data under carefully controlled, fully monitored conditions.

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