CYCLIC TENSILE TESTING OF A PILE IN GLACIAL TILL

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RESUME

For offshore structures in deep water, piles are being designed to withstand cyclic tension, due to uplift and wave loading, throughout their design life. However there is a scarcity of data concerning the load levels which can be safely applied in this manner. This paper reports tensile tests on a heavily instrumented 10 metre long pile jacked into glacial till. Previous work had shown satisfactory pile behaviour under cyclic tensile loads peaking at up to 48% of the ultimate tensile capacity, and so this work was aimed at investigating pile response to more severe load levels, approaching failure. Cyclic tests were performed with varying peak loads up to 80% of the initial static capacity, and up to 13,500 cycles were applied depending on pile response. The pile sustained encouragingly high loads without serious deformation, but failure did occur during the most severe test, when the peak load was nominally 80% of the ultimate tensile capacity. Pile response analysis provided insight into compression pile design methods when applied to tension piles. Alpha and Lambda methods, but not the Beta method, estimated ultimate tensile capacity well, whilst stiffness was greater than implied by published T-Z curves.
1.0 INTRODUCTION

The oil and alternative energy industries, by their increasing interest in large scale floating structures, have stimulated research into the behaviour of conventional steel pipe piles as anchors.

A major development programme has recently been completed for one such floating system, required to provide a working platform in the 200-400 metre range of water depths (Smith and Taylor 1980). The structure comprises a buoyant column, tethered to a foundation anchorage by a series of tendons. Due to the positive buoyancy of the column, the fluctuating loads imposed on the foundation are always tensile.

The choice for the foundation design was between a piled or gravity base. Most of the foundation development programme was directed towards a piled base.

Available design guidance was considered inadequate for both the ultimate carrying capacity of piles in tension and pile response to working loads. Further, the effect on pile capacity of the vertical and lateral cyclic loads transmitted during severe storms could only be tentatively quantified. With this level of uncertainty the preliminary design method had to ensure that relatively low stress levels were applied to the piles during the 100 year storm. To check the adequacy of the design an experimental programme of laboratory and onshore large scale pile tests was undertaken. These tests substantiated the design adopted, examined the stability of a soil during representative storm loadings and determined the relative efficiency of jacking force to tensile capacity. In these tests the basic pile parameters and loading regimes were applicable to a jacked pile arrangement in a preliminary foundation design.
On completion of this section of the field test programme it was felt that it would be of considerable value to determine the behaviour of a single pile under more severe loadings. Thus a single, heavily instrumented, open ended 10.4m long steel pipe pile was tested under a series of high level tensile load packages. The results of these tests are presented in this paper. Further details of the laboratory and field test programme have been reported by Garas (1979) and Garas and McAnoy (1980).

2.0 DESCRIPTION OF FIELD TESTS

2.1 TEST SITE

Taking into account likely North Sea locations for a buoyant column structure, the most suitable ground condition for an onshore test site was considered to be a clayey glacial till. The UK Building Research Establishment (BRE) had already established a test site on the Holderness coast at Cowden, 23 km north east of Hull, Humberside, to investigate the properties of glacial till. An area of this site was made available for these pile tests together with site investigation data, including large scale plate tests and pressuremeter results.

The ground consists of an exposed thick deposit of uniform overconsolidated, sandy silty clay till, with interposed layers of sand and gravel. Boreholes and cone penetrometer results adjacent to the test position indicate zonal weathering of the till down to 4.5 metres and a dense grey black sand and gravel layer at 10.5 metres. Marsland and Powell (1980) gave a more detailed description of the till. A comparison of the residual strength of this Cowden till with various other clay types has been presented by Lupini et al (1981).

Table 1 shows typical soils data as assessed from information provided by the BRE, and from the above references. Figure 1 (a) shows a typical borehole log for the site, adjacent to the test position and 1 (b) typical cone penetrometer readings.
2.2 TEST PILE AND INSTRUMENTATION

The test pile was one of four steel pipe piles (193mm diameter, 9mm wall thickness by 10.4m long) which were each jacked 9.9m into the ground to form a square group at 580mm centres. They were installed open ended with a cutting shoe to reduce plugging. External ducts were welded to the piles to protect the instruments and cables. The location of the instruments on the most heavily instrumented pile, used for these single pile tests, is shown on Figure 2.

The axial load along the pile was monitored by $\frac{1}{4}$ bridge weldable electrical resistance strain gauges, positioned as pairs, diametrically opposite each other. The imposed head loads were monitored by a load cell built into the loading system. Pore water and total lateral soil pressure measurements at the pile/soil interface were taken by miniature electrical transducers mounted in the ducts, flush with the outer face.

Pile head vertical and horizontal displacements were measured relative to an independent reference beam by linear displacement transducers. Pore pressures in the surrounding soil at 2, 6 and 12 radii and 3.5m depth were measured by hydraulic piezometers open to the atmosphere.

All instruments were calibrated and checked in the laboratory, to ensure as far as possible that they would perform satisfactorily during testing. Whenever practical, the same calibration procedure was repeated on site before pile installation.

2.3 TEST PROGRAMME

The four piles were jacked into the ground over an eight day period in April 1980. A pile was pitched and the instrumentation checked during one day, and the pile installed the next. On average installation took 6 hours to reach the full embedded length of 9.9 metres.
Over a 41 day period from the end of jacking, the measured excess pore pressure at the pile-soil interface reduced to within 5% of the hydrostatic pressure. During this period the test facility was rearranged to accommodate the load actuating system, and a rigid steel pile cap was welded to the head of the piles, leaving a clear gap between the pile cap and the ground.

A series of cyclic load tests, summarised in Figure 3, were then carried out on the pile group. This test work, which is to be discussed by McAnoy et al (1982), included a load package equivalent to the spectrum of a severe storm, with applied loads of up to 48% of the vertical or 6% of the horizontal estimated pull out capacity ($U_t$). The estimate of $U_t$ was based upon the capacity of a single pile of identical dimensions installed and tested on the same site adjacent to the test position during the summer of 1979.

At the conclusion of the group tests it was decided that it would be of considerable value to determine individual pile performance under even more demanding loading conditions. The pile cap was therefore dismantled, and the loading system positioned over one of the four piles. A constant rate of extraction test to failure was carried out as soon as possible after the group tests to determine the ultimate capacity of this pile. The value obtained was then used as the initial capacity of the single pile prior to high level cyclic loading. Results from this initial extraction test and subsequent high level cyclic loading of this pile have been used for the analysis presented. A summary of the test programme considered is shown in Figure 3(b).

Each of the three cyclic tests shown was immediately followed by a constant rate of extraction test to determine the ultimate pullout capacity. A rate of displacement of 0.75 mm/min was used, as recommended by Whitaker (1963) for piles in cohesive soils. The tensile loads and numbers of cycles applied were nominally:

<table>
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<th>Mean</th>
<th>Cyclic Component</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>40% $U_A$</td>
<td>$\pm 20% U_A$</td>
<td>11,775</td>
</tr>
<tr>
<td>40% $U_B$</td>
<td>$\pm 40% U_B$</td>
<td>564</td>
</tr>
<tr>
<td>30% $U_C$</td>
<td>$\pm 30% U_C$</td>
<td>13,597</td>
</tr>
</tbody>
</table>
for tests A, B and C respectively, where $U_A$ is the ultimate capacity determined immediately before test A, etc.

Sinusoidal loading was performed at two frequencies ($0.1H_z$ and $1.0H_z$) to maintain similitude of strain and drainage respectively with a prototype pile. Problems of similitude for scaled pile tests are discussed by Gallagher and St. John (1980). To ensure that viscous damping did not affect cyclic displacements when drainage was being modelled, testing was performed under displacement control for the higher frequency.

3.0 ANALYSIS OF FIELD TEST DATA

3.1 ULTIMATE TENSILE CAPACITY

3.1.1 Results

Figure 4 shows the six constant rate of extraction tests to failure performed on the pile. The first two tests E1, E2, were carried out before the pile had been subjected to high level cycling and were used to deduce the ultimate tensile capacity. The change in the capacity after applying the cyclic load was assessed by using the capacity recorded on extraction tests E3 to E6.

The maximum load from test E1 was 495kN, which was comparable to 481kN measured during test E2. The first test, E1, was carried out a month after the completion of the group tests, and the second test, E2, was performed a month later, one day before the start of the cyclic tests. The slopes of the load displacement curves are however considerably different. The first curve was non-linear over the majority of its length and peak capacity was reached at 5.2mm displacement. The second curve was linear up to over 80% of the ultimate capacity, which was reached at 3.2mm displacement.

Extraction test E3, performed immediately after cyclic test A, indicated a decrease in ultimate load capacity, to 457kN. Extraction test E4, after cyclic test B, indicated a further decrease to 400kN. The
last two extraction tests, E5 and E6, showed a slight increase of ultimate capacity. The shape of the load-displacement curves are linear over 80% of their range. The corresponding displacement required to mobilise these values were reasonably consistent at about 2.5mm.

3.1.2 Predictions of Ultimate Tensile Capacity

Three conventional methods of predicting ultimate pile capacity have been assessed: the Alpha, Beta, and Lambda methods. None of these methods distinguish between tensile and compressive loading.

In using them a number of assumptions have been made based upon site investigation data and earlier results of pile load distribution. Firstly it is assumed that the uppermost 1.0m of the ground contributed nothing to the side resistance, since strain gauge readings indicated that resistance was very low in this region. Secondly the Cu vs depth profile can be idealised to 170 kN/m² for depths 1.0m to 4.5m and 120 kN/m² from 4.5m to 10.0m depth, as suggested by pressuremeter test results (Table I).

Alpha Method

API RP2A (1980) recommends that this total stress method is to be used for soils of medium to low plasticity and can be generally applied to overconsolidated clays. It is assumed that the ultimate skin friction on the pile wall is proportional to the undrained shear strength, according to the equation.

\[ \tau_s = \alpha C_u \]

where 

- \( \tau_s \) = ultimate shear stress on pile wall 
- \( \alpha \) = 0.5 for the values of \( C_u \) assumed above.

The alpha method predicted a static tensile capacity of 410 kN.
Beta Method

A number of methods have recently been developed using the effective stress parameters $c', \phi'$. These methods generally require an estimate of the radial stress on the pile shaft after set up. In heavily overconsolidated clays this is difficult to obtain.

The basic effective stress equation involving $\beta$ as suggested by Burland (1973) is:

$$\tau_s = \beta \sigma'_{vo}$$

where

- $\beta = K_s \tan \phi'$
- $\phi' = \text{softened drained angle of shear resistance}$
- $K_s = \text{coefficient of lateral earth pressure}$
- $\sigma'_{vo} = \text{insitu vertical effective stress}$

It is assumed throughout that $c' = 0$ due to disturbance of the soil during installation.

Meyerhof (1976) has analysed a large number of pile tests on bored and driven piles to derive $\beta$ values. He defined $\beta$ as above but from his work states that $K_s$ varies from roughly $K_o$ to $2K_o$ for driven piles, and suggests that on average $K_s$ may be taken as $1.5K_o$ in stiff clays. An empirical relationship between the overconsolidation ratio, $\phi'$, and $K_o$, is also suggested:

$$K_o = (1 - \sin \phi')\sqrt{OCR}$$

where

- $K_o = \text{coefficient of earth pressure at rest}$
- $OCR = \text{overconsolidation ratio}$

Hence

$$K_s = 1.5 (1 - \sin \phi')\sqrt{OCR}$$

Meyerhof's formula predicted an ultimate pile capacity of 270kN.

Lambda Method

The Lambda method (Vijayvergiya & Focht, 1972) has been developed from the same pile test data as the alpha method, but incorporates the
effect of penetration length on ultimate skin friction, which is calculated from the equation

\[ U_T = \lambda (\sigma'_{vo} + 2 C_u) A_s \]

where
- \( \lambda \) = dimensionless constant, which is a function of pile penetration
- \( A_s \) = surface area of pile
- \( \bar{C} \) = signifies average over pile length

The Lambda method predicted an ultimate pile capacity of 430 kN.

3.1.3 Discussion on Ultimate Capacity

A summary of measured and predicted ultimate capacities is given in Table 2. Two of the extraction tests have been used to highlight the measured capacities before and after failure under cyclic loading (Tests E1 & E4 respectively). It is possible that the 19% change in measured capacity during this series of tests was influenced by the formation of continuous slip surfaces, as 18mm of cumulative displacement was recorded. The reduction in capacity was close to that which could be expected if the effective angle of friction of the soil reduced to the residual.

The alpha and lambda methods provided reasonable estimates of the ultimate tensile capacity, and compared particularly well with the measured capacity after cyclic failure. In order to have predicted the measured capacity before cycling the alpha values used would have had to be approximately 0.6. Previous case studies indicate that alpha is often higher than the recommended value of 0.5. Further it is generally expected that jacked piles have a greater capacity than driven piles. The greater measured capacity may also reflect the influence of the installation of the adjacent three piles due to a change of insitu stresses acting on the pile shaft.

The beta method provided poor correlation between measured and predicted results. However, Meyerhof (1976) does show a wide scatter of \( \beta \) values from case records of piles in stiff clays. It has been
suggested that part of the reason for this lies in the values of the effective stresses assumed (Parry & Swain, 1977), as it is difficult to assess the correct values. This uncertainty is reflected in the poor correlation noted above. However, given a better determination of the effective stresses to be used and the manner in which they change due to installation and loading, the method could in principle allow for the effects of adjacent piles, method of installation and direction of loading. Better correlation has been achieved in soft clays, in which effective stress can be more accurately assessed.

Difficulties were experienced in assessing the undrained shear strength values to be used in the calculations as the various insitu and laboratory test methods indicated very different values. Most methods indicated that for 0-10 metre depth the top 4.5 metres of soil had a higher Cu value than the soil below. However, the magnitude of the difference varied considerably.

3.2 RESPONSE DURING CYCLIC TESTS

Displacements during tests A, B and C are shown in Figures 5 and 6. Tests A and C produced surprisingly small permanent displacements - 0.12 and 0.14 mm respectively - after over 11,000 cycles despite the high load levels applied. A linear relationship between mean displacement and log number of cycles is evident, with the gradient of the line being slightly greater for A than for C, reflecting the difference in peak loads. During the first 40 cycles or so, test B exhibited a similar linear relationship, although the gradient of the line was considerably greater than for the other tests. However, from 40 cycles onwards an entirely different response occurred. Mean displacements increased drastically, and continued to do so up to the point where the change of mean displacement per cycle was increasing with every cycle, indicating pile failure. Once this point had been clearly passed test B was terminated - only 564 cycles had been necessary.
No change in cyclic displacement occurred within any of the three tests, indicating that there was no degradation of cyclic soil modulus, contrary to the suggestion of Poulos (1980) that cyclic shear degradation was a function of stress level and number of cycles. It is particularly surprising that no degradation in cyclic stiffness occurred during test B, especially when the pile was failing as is clearly shown in Figure 6(a).

3.2.1 Axial Load Distribution

Figure 7 shows the axial load down the pile under cyclic and static loading at the beginning and end of tests A, B and C. There is no significant change with number of cycles, indicating that no load shedding occurred during cycling. Furthermore there is no change in the relative proportion of load taken by each element during these cyclic tests and the ultimate values given by extraction test E3. It is clear that the average shear stress, represented by the rate of decrease of axial load with depth, varies considerably from the 1-3m depth to the 3-8m depth, which is broadly consistent with the change in soil strength at about 4-5m below ground, as discussed in Section 3.1.2.

3.2.2 Pore Pressures

Readings of excess pore pressure at the pile/soil interface, taken at three depths as shown in Figure 2 during tests A, B and C showed that initial loading gave rise to a small excess pore pressure. A maximum increase of 15 kN/m² was measured at 3.15m level. For tests A and C, cyclic loading did not lead to any build-up in excess pore pressure and the initial pressure reduced. There was some evidence to suggest a slight increase in excess pore pressure during the first 40 cycles of test B to a maximum of 20 kN/m² at the 3.15m level. As pull-out increased this pressure dissipated, and at the end of all three tests excess pore pressures were continuing to decrease. None had quite returned to zero, but the maximum value at the end of cycling was down to approximately 5 kN/m².
3.2.3 Pile Head Stiffness and Apparent Soil Shear Modulus

Values of pile and soil stiffness deduced from the cyclic tests are shown in Figure 8. The initial static stiffness was reasonably consistent between test A, B and C, whilst the final stiffness was similar for tests A and C but dropped by 12% in B. The effective stiffness reduced as the pile gradually pulled-out, the drop being drastic during test B.

The cyclic stiffnesses remain constant throughout each test, but vary from one test to another. Figure 9 shows a plot of pile head stiffness versus load range (i.e. maximum minus minimum head load) for the three cyclic tests. It appears that load range and hence stress range, has been the dominant factor in terms of cyclic stiffness, reflecting the non-linearity of the soil stress-strain curve, particularly at high load levels. Mean load level and rate of test would also be expected to influence cyclic stiffness, but are not thought to be significant in this case. For example, the highest loading rate, which occurred during test B, would be expected to produce the highest cyclic stiffness, when in fact test B exhibits the most flexible response.

The apparent shear modulus of the soil at the pile/soil interface has been derived for an element of soil at 4.5m depth and exhibits the same trend as pile head stiffness (Figure 8). The apparent shear moduli derived, which range from 50-76 MN/m², are 3 to 7 times greater than values determined from plate and pressuremeter tests, Table 1. Similar behaviour was observed by Gallagher & St. John from pile tests on the same site. This is to be expected as the shear strain during the tests was considerably lower than those produced by most insitu testing.

3.3 NUMERICAL ANALYSIS OF LOAD-DISPLACEMENT BEHAVIOUR

One method of estimating the load-displacement behaviour of a pile is to use a discrete element technique. The soil is modelled as a series of non-linear ground springs, characterised by 'T-Z curves' relating pile displacement to mobilised shear stress. The pile is divided into elastic elements acted on by the relevant ground spring. A beam-column computer program can then be used to determine the pile head
load-displacement curve. Results from the extraction tests have been analysed to derive experimental T-Z curves. Results from extraction test E4, which produced the lowest ultimate capacity, were considered to be of greatest interest, and are presented here.

3.3.1 Derivation of T-Z Curves from Experimental Data

The pile was divided into seven elements as shown in Figure 10. Using the measured drop in axial load across a given element, the average shear stress on that element was computed. Displacement, \( z \), of an element was found from the measured pile head displacement and the axial load down the pile;

\[
  z = Z_t - x \int_0^x \frac{Q(x)}{AE} \, dx
\]

where
- \( Q(x) \) = axial load in the pile at any depth \( x \)
- \( A \) = sectional area of pile
- \( E \) = elastic modulus of pile material
- \( Z_t \) = pile head displacement.

3.3.2 Results and Predictions

From extraction test E4, the development of shear stress on each element for four load levels (a) - (d) are shown on Figure 11. Load level (d) represents the approximate ultimate capacity. The corresponding displacements of each element are also shown. Using information of this type the T-Z curves for elements 3-6 have been plotted on Figure 12. Also shown are 'predicted' curves, which have been produced from the work by Coyle & Reese (1966), who suggested a series of T-Z curves for piles installed in cohesive soils. Three standard curves were proposed for varying soil depths, each being presented in terms of undrained shear strength, after a correction to take into account the effect of pile installation. The predicted curves of Figure 12 have been produced using the in-situ undrained shear strength profile assumed in Section 3.1.2.
Vijayvergiya (1977) developed a different approach from results of load tests on compression piles. It was suggested that the equation:

\[ T = T_{\text{max}} \left( 2 \sqrt{Z/Z_c} - Z/Z_c \right) \]

where \( T \) = average shear stress on an element
\( T_{\text{max}} \) = ultimate average shear stress on an element
\( Z \) = vertical displacement of an element
\( Z_c \) = critical displacement of an element required to mobilise \( T_{\text{max}} \).

could be used to define \( T-Z \) curves for cohesive soils up to failure. Furthermore Vijayvergiya suggested that \( T_{\text{max}} \) should be derived using conventional methods for axial capacity, as discussed in Section 3.1.3, and that 5-8mm is a typical range of \( Z_c \) for piles of 300mm diameter and greater. Figure 13, produced from Figure 12 shows that measured values of \( Z_c \) decrease with depth from a maximum of approximately 1.5mm (0.8% of pile diameter) to 0.4mm. Figure 14 shows the difference in shape between normalised plots of Vijayvergiya's equation, measured values, and Coyle and Reese's curve for depths greater than 6 metres.

### 3.3.4 Discussion

It is clear from Figures 12 and 14 that the \( T-Z \) curves exhibit a bilinear behaviour, and that the curve suggested by Vijayvergiya is inappropriate in this case. The curve for element 3 indicates a slightly softer response than the remainder, but this is considered to be due to the existence of a low radial stress over the upper part of this element. Each curve can be described by two parameters, \( T_{\text{max}} \) and \( Z_c \).

The values of \( T_{\text{max}} \) predicted by Coyle & Reese for element 6 agrees with measurements, but curves for elements 3, 4 and 5 yield gross underestimates. It would appear that a method of predicting \( T_{\text{max}} \) from cone penetrometer results is desirable, since some correlation can be observed between the shape of the ultimate shear stress distribution of Figure 11(a), and the sleeve friction measurements of Figure 1(b).
With respect to critical displacement, the recommendations of Coyle and Reese, and Vijayvergiya, again significantly overestimate measured values. Neither of these methods incorporate the effect of diameter on \( Z_c \), although the measured results appear to conform to the suggestion of 1\% pile diameter as an upper value for \( Z_c \) (Whitaker and Cooke, 1976) for the piles tested. The reduction of \( Z_c \) with depth which was observed is implied by the curves of Coyle and Reese, and is possibly due to an increase in confining pressure with depth.

In any assessment of existing methods of analysis, it is important to note the conditions for which the methods were designed. The curves of Coyle and Reese and Vijayvergiya were suggested for piles under sustained compression, and may be affected by direction and rate of loading. On first impressions, therefore, it might seem inappropriate to compare the curves with results from a constant rate of extraction test, which might be expected to produce a stiffer response, particularly at high load levels. However, to put this into perspective the cumulative displacements recorded after the completion of cycling in Test A have been plotted on Figure 12. The pile was still under the mean load and this displacement is considered as an upper bound to the displacements which would have been recorded during sustained static loading over the same period of 6 ½ hours. The point still indicates that the element response is stiffer than that predicted by Coyle and Reese. However at higher load levels displacements due to creep are more significant and thus may approach or even exceed predicted displacements.

4. CONCLUSIONS

Effect of Cycling on Ultimate Tensile Capacity

1. The ultimate tensile capacity \( (U_t) \) of the pile reduced by 19\% during the test programme. This reduction was close to that which could be expected if the effective angle of friction of the soil reduced from the remoulded to the residual value.
2. The lowest $U_t$ value occurred immediately after the test B, when the pile failed under cyclic loading, and the reduction was probably due to a combination of cyclic loading and cumulative displacements.

Accuracy of Compression Pile Methods for Determining $U_t$

3. Good correlation between measured and predicted $U_t$ values was achieved using the alpha and lambda methods, which erred by 10-20% on the conservative side before cycling, and were within 7% of the measured value after cyclic failure.

4. The beta method provided a gross underestimate before and after cycling.

Response During Cycling

5. The pile sustained encouragingly high levels of cyclic loading, peaking at up to 60% $U_t$, with permanent displacements of only 0.14mm after 11,000 cycles.

6. At a higher load level, peaking at nominally 80% $U_t$, a dramatic change in behaviour occurred after a small number of cycles, and failure followed.

7. The cyclic stiffness of the pile did not vary with number of cycles in any of the tests, even during failure.

8. No significant build-up of pore pressure due to cycling was observed.

9. No load shedding down the pile occurred during cycling, even at failure
T-Z Curves

10. The relationship between mobilised shear stress (T) and displacement (z) on a pile element was linear for \( Z \leq Z_c \), the 'critical' displacement. For \( Z > Z_c \), T remained approximately constant.

11. Ultimate shear stress, \( T_{\text{max}} \) predicted by the T-Z method of Coyle and Reese underestimated measured values in the upper levels of the pile, apart from the top metre, which took no significant load.

12. The distribution of \( T_{\text{max}} \) with depth showed some correlation with core penetrometer sleeve friction results.

13. The critical displacement reduced with depth, possibly due to an increase in confining pressure.

14. Critical displacements were considerably less than the values suggested by Coyle and Reese. This may be partly due to the test pile being of a smaller diameter than those from which the design curves were derived.

Future Work
This investigation was limited to the tensile behaviour of jacked piles in one soil type. Considerably more research is necessary before efficient design methods can be relied on for dynamically loaded tension piles. This research should concentrate on the load transfer mechanism, creep and sensitivity of soils to dynamic loads.

Taylor Woodrow and the U.K. Building Research Establishment have proposed an extensive development programme to measure the performance of different types of piles under representative tensile axial and lateral loading conditions and to assess appropriate design methods. This will incorporate and extend both organisations' present work in this area. (Negotiations with potential sponsors for this project are well advanced, although further sponsorship is still being sought).
Acknowledgements

The Authors would like to thank the Directors of Taylor Woodrow for their permission to publish these results, the U.K. Building Research Establishment for the use of their test site and site investigation results, and all Research Laboratory personnel involved in the project.

LIST OF REFERENCES


McAnoy, Cashman and Williams. To be Published


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* Calculated from the equation \( C_u = (P_L - P_{HO})/6.18 \), where
  
  \( P_L \) = limiting pressure & \( P_{HO} \) = original horizontal stress in the ground.

+ Average G for 1% strain

° Secant modulus over range \( P_{vo} \) to \( P_{vo} + \frac{1}{3} (q_u - P_{vo}) \), where
  
  \( q_u \) = ultimate base pressure & \( P_{vo} \) = original vertical stress in the ground.
Table 2. Summary of Ultimate Tensile Capacity

<table>
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<tr>
<th>Test No.</th>
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<th>Predicted Values (kN) and Predicted/Measured Values (%)</th>
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<td></td>
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</table>
FIGURE 1 (a) Borehole Log (b) Cone Penetrometer Log.
FIGURE 2 Pile Instrumentation
FIGURE 3. Load History of the Piles (a) Previous Loading During Group Test (b) Single Pile Test.
FIGURE 4. Constant Rate of Extraction Tests To Failure, E1–E6
Cyclic Test A
(194 ± 97 kN)

Cyclic Test B
(194 ± 193 kN)

Cyclic Test C
(123 ± 122 kN)

FIGURE 5. Pile Head Displacements During Cyclic Tests A, B, & C
FIGURE 6 Pile Head Displacements

(a) Cyclic Amplitude

(b) Mean

Cyclic Test B
(194 ± 193 kN)

Cyclic Test C
(123 ± 122 kN)

Cyclic Test A
(194 ± 97 kN)
<table>
<thead>
<tr>
<th>Pile Head Load</th>
<th>Static, Before Cycling</th>
<th>Peak, During Cycling</th>
<th>Ultimate, During Extraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>A, C</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>E3</td>
</tr>
</tbody>
</table>

FIGURE 7 Axial Load Distributions During Testing.
<table>
<thead>
<tr>
<th>Description of Modulus</th>
<th>Test A</th>
<th>Test B</th>
<th>Test C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K</td>
<td>G</td>
<td>K</td>
</tr>
<tr>
<td>Initial Static Loading</td>
<td>262 (252)</td>
<td>76</td>
<td>262 (252)</td>
</tr>
<tr>
<td>Cyclic</td>
<td>255 (215)</td>
<td></td>
<td>226 (198)</td>
</tr>
<tr>
<td>Effective, 10\textsuperscript{th} Cycle</td>
<td>249</td>
<td>211</td>
<td>233</td>
</tr>
<tr>
<td>100\textsuperscript{th} &quot;</td>
<td>231</td>
<td>198</td>
<td>228</td>
</tr>
<tr>
<td>250\textsuperscript{th} &quot;</td>
<td></td>
<td>139</td>
<td></td>
</tr>
<tr>
<td>500\textsuperscript{th} &quot;</td>
<td></td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>1,000\textsuperscript{th} &quot;</td>
<td>220</td>
<td></td>
<td>217</td>
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<tr>
<td>10,000\textsuperscript{th} &quot;</td>
<td>217</td>
<td></td>
<td>196</td>
</tr>
<tr>
<td>Final Static Unloading</td>
<td>255</td>
<td>70</td>
<td>228</td>
</tr>
</tbody>
</table>

K - Pile Head Stiffness (kN/mm)
(K) - Stiffness, Derived From CRE Tests Over Same Load Range
G - Apparent Shear Modulus at Mid-Depth of Pile (MN/m²)

FIGURE 8 Variations in Pile Head Stiffness and Apparent Soil Shear Modulus
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Critical Vertical Displacement, $Z_C$, (mm)

Depth (m)

FIGURE 13 Critical Vertical Displacement Versus Depth
FIGURE 14 Normalised T-Z Curves


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**PROBLEMS IN DESIGN AND INSTALLATION OF OFFSHORE PILES**

By Bramlette McClelland,1 John A. Focht, Jr.,2 Fellows ASCE and William J. Emrich,3 M. ASCE

(Reviewed by the Technical Council on Ocean Engineering)

**INTRODUCTION**

During the past 20 yr (prior to 1969), several hundred ocean platforms have been built in waters up to 385 ft deep. Most of these have utilized template or “jacket” type construction. As illustrated in Fig. 1, construction by this procedure begins with placement on the ocean floor of a tubular steel substructure previously fabricated onshore. Open-ended pipe piles are then inserted and driven through the substructure columns, which may be either vertical or on a moderate batter. After the piles are fastened to the substructure, a prefabricated deck unit is placed on the piles at an elevation above the crests of anticipated storm waves.

Under the combined effects of vertical and horizontal loads on the structure, as illustrated in Fig. 2, the pipe piles are subject to high loads, requiring ultimate compressive capacities of up to 3,500 tons. Because of high overturning moments, particularly in deep water, the piles also require ultimate tension capacities up to 2000 tons. Horizontal design loads up to 150 tons per pile develop high bending stresses in the piles near the mudline.

Piles presently used in offshore construction are typically 30 in. to 48 in. in diameter. Penetrations to produce the required ultimate capacities vary widely, but are frequently in the order of 250 ft to 350 ft. As the piles generally extend some distance above the water level, total pile lengths of 400 ft to 600 ft are common.

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Some typical features of pile design and fabrication are given by the example in Fig. 3. The pile illustrated here has a diameter of 4 ft and a length of 310 ft. It was designed for a site where water depth is 280 ft and where the subsurface materials consist of soft-to-stiff clays. The indicated variation in wall thickness, from 5/8-in. to 1-1/8-in. in this case, is typical. Of particular interest is the heavy-wall section in the zone of high bending stress, which extends from a short distance above the mudline to a substantial depth below the mudline. The lower part of the embedded portion of the pile has a reduced thickness except for a heavy-wall end section which serves as a driving shoe. Field splices, shown in the left-hand side of Fig. 3, are located well away from the points of change in wall thickness, and the distances between field splices are chosen to facilitate field assembly and pile driving. The pile described in Fig. 3 will be referred to again in later illustrations, and for brevity it will be designated simply as Pipe A.

It is pertinent to compare offshore piles to those used onshore, as most pile design and construction experience has accumulated in connection with land operations. Fig. 4 emphasizes the contrast that exists between this onshore experience and the offshore situation to which it must be extended. In land construction, pipe piles are typically designed to produce ultimate capacities up to 200 tons, only a small fraction of the capacity required offshore. Offshore piles, both in dimension and in weight, dwarf the piles...
normally used onshore. The design and installation of piles for ocean platforms, therefore, constitute a giant step beyond land experience.

It is the intent of this paper to direct attention to pile design and installation problems now facing the marine construction industry as a consequence of this giant step. In this state-of-the-art review, which will utilize four case histories for purposes of illustration, three broad problems will be seen to emerge:

1. For some subsurface conditions, great uncertainties exist in establishing the minimum pile penetration required to support an offshore design load.
2. Offshore design capacities now required for deep-water structures cannot always be achieved by pile driving alone, using presently available hammers.
3. When supplemental means such as drilling or jetting are used to aid pile penetration, uncertainties with respect to pile capacity are increased.

PREDICTION OF PILE CAPACITY

Traditionally, three methods have been used for determining pile capacity, namely: (1) Load tests; (2) dynamic formulas or "driving tests"; and (3) computations using the static method. A load test program is uneconomical for the design of a specific ocean platform because of the marine environment and because of the high loads required. Dynamic formulas have long been discredited as a means for predicting pile capacity, and the method has such limited applicability in offshore construction that it offers little design assistance. The third method, which utilizes borings combined with laboratory or in-situ tests, is the most useful design approach for long, heavily-loaded piles despite its inherent limitations.

The static method of predicting the ultimate capacity, Q, of a pile driven to penetration, D, relies on empirical data derived from model studies and full-scale load tests, interpreted in the light of accepted soil mechanics theories. Fig. 5 illustrates that the major components of ultimate compressive capacity are \( Q_s \), the shaft or skin friction load, and \( Q_p \), the end bearing or point load. Thus

\[
Q = Q_s + Q_p = fA_s + qA_p
\]

in which \( A_s \) and \( A_p \) = respectively, the embedded surface area and the pile end area; and \( f \) and \( q \) = respectively, the unit skin friction and unit end bearing. Mathematically, the appropriate value of \( f \) in Eq. 1 would be the average unit skin friction over the full depth of pile penetration. In practice, \( Q_s \) is usually computed as the summation of skin friction available in individual soil strata or within incremental portions of the pile penetration, and the values of \( f \) used for this purpose represent the unit skin friction at specific depths below the soil floor. The capacity of a pile to resist tensile loads is normally considered to be only \( Q_s \), the total skin friction. Successful application of the static method depends upon selection of appropriate values of \( f \) and \( q \) that take into account the combined effects of soil conditions, pile type and dimensions, pile installation procedure, and manner of loading.

Piles in Clay.—For piles in clay, the unit end bearing capacity, \( q \), is the simple product of a bearing capacity factor, called \( N_c \), and the cohesive shearing strength of the clay. For a round, deeply embedded pile, \( N_c \) has a value of about nine according to both theoretical and empirical studies. For most cases of piles in clay, the resulting value of end bearing, \( Q_p \), will be quite small in comparison to the total friction on the pile, \( Q_s \).

The unit skin friction, \( f \), at any point along a pile in clay may be less than, but not more than, the shear strength of the clay, \( c \). The magnitude of this soil

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**FIG. 5.—BASIC CONCEPTS AND DESIGN FACTORS IN STATIC METHOD FOR COMPUTING PILE CAPACITY**

**FIG. 6.—TWO MAJOR SOURCES OF EMPirical DATA FOR APPLYING STATIC METHOD TO PILES IN CLAY**

resistance factor, also called "soil-pile adhesion," cannot be determined theoretically but must be established empirically from load tests. Two important groups of data giving information on the relationship between \( f \) and \( c \) are illustrated in Fig. 6. Data assembled by Peck (9)* and reproduced in Fig. 6(a) indicate that for soft-to-firm clays, having a shear strength of less than 0.5

*Numeral in parentheses refer to corresponding items in the Appendix.—References.
ton per sq. ft, the maximum adhesion mobilized between pile and soil in a load test is approximately equal to the shear strength of the soil. Data assembled by Tomlinson (14) and reproduced in Fig. 6(b), on the other hand, indicate that \( f \) is substantially less than \( c \) in stiff-to-hard clays in which \( c \) exceeds 0.5 ton per sq. ft. Similar conclusions are drawn by Woodward, Lundgren, and Boitano (17) from their analysis of another group of pile load tests. Each of these reports concerning adhesion in stiff clay deals with piles less than 100 ft in length, and it is evident that their data relate entirely to overconsolidated clays. No published data are available, so far as the writers know, to indicate the relationship between \( f \) and \( c \) in stiff, normally-consolidated clays, which is frequently encountered by offshore piles driven below 100-ft penetration. Limited laboratory tests by the writers suggest that adhesion will be approximately equal to cohesion in such situations.

![FIG. 7.—BEARING CAPACITY FACTOR \( N_q' \) FOR DEEP CIRCULAR FOUNDATIONS](image)

![FIG. 8.—VARIATION OF END BEARING CAPACITY WITH PENETRATION, FROM EXPERIMENTS WITH 4-IN. DIAM MODEL PILES](image)

**Piles in Sand.**—The design factors and equations frequently used in current practice for computing the capacity of piles in sand are noted on Fig. 5. The unit skin friction, \( f_s \), at a particular depth is expressed as a coefficient \( K \) times the effective overburden pressure, \( p_o \), and the tangent of the skin friction angle, \( \phi \), at that depth. The unit end bearing, \( q_e \), is expressed as a bearing capacity factor \( N_q' \) times overburden pressure at the pile tip. The terms in the equations for both soil resistance factors, \( f_s \) and \( q_e \), are readily determinable within narrow limits except for \( K \) in the friction equation and \( N_q' \) in the end bearing equation.

The bearing capacity factor, \( N_q' \), for a deep circular bearing area is widely held to be a function of the angle of internal friction, \( \phi \). The specific nature of the relationship, however, is subject to widely varying interpretations. Those by Terzaghi (13), Hansen (3), and Meyerhof (8), are shown together on Fig. 7. For a sand with an angle of internal friction \( \phi \) equal to 35° the indicated range for \( N_q' \) is from 41 to more than 300, a variation of about 700%. The relationship shown are theoretical, the differences deriving from assumptions concerning the path of shear failure and the compressibility of the sand above the pile tip. Unfortunately, \( N_q' \) does not yield readily to experimental investigation because of difficulty in isolating it from other factors.

Based on the equations in Fig. 5, average skin friction and unit end bearing for a pile in sand both increase directly with effective overburden pressure and, therefore, almost directly with depth of penetration. On the other hand, Kerisel (4) and Vesic (15,16) found in small-scale pile tests that \( q \) is not proportional to pile penetration but tends to reach a maximum value and then to remain more or less constant with increasing depth as shown in Fig. 8. Their tests indicate further that average skin friction also reaches a limiting maximum value, leading Vesic (15) to conclude that "indiscriminate use

![FIG. 8.—VARIATIONS IN COEFFICIENT OF EARTH PRESSURE, K, COMMONLY RECOGNIZED IN DESIGN PRACTICE](image)

of overburden stress, \( p_o \), constitutes a major fallacy in analyses of deep foundations." While results of such studies cannot be interpreted quantitatively for application to large-diameter piles driven to great depths, the writers conclude that the use of equations in Fig. 5 should be combined with reasonable maximum values of \( f_s \) and \( q_e \). It is of interest to note that this conclusion agrees generally with longstanding design practice in Europe, as given for example by Schenck (11).

In the equation for unit skin friction for piles in sand, as given in Fig. 5, the coefficient of earth pressure, \( K \), is the most sensitive and also the most elusive factor. This coefficient, when multiplied by \( p_o \), is supposed to give the
intensity of earth pressure pushing against the side of the pile. This pressure intensity, and therefore the magnitude of \( K \), is known to be influenced by at least the following six factors: (1) Initial state of stress \( (K_o) \) in the sand deposit; (2) initial density of the sand; and (3) displacement volume of the driven pile; (4) pile shape, including taper; (5) installation procedures other than driving; and (6) loading direction (compression or tension). The first two factors describe the initial condition of the sand deposit, the next three relate to volume changes that occur during pile installation, and the last factor concerns the direction of loading and its effect on the state of stress in the sand body.

Typical values of \( K \) that are used in prediction of skin friction capacity and that have been found to be reasonably consistent with results of onshore pile load tests, are illustrated in Fig. 9. Prior to any construction activity, the ratio between the horizontal soil pressure, \( p_h \), and overburden pressure, \( p_o \), is called the coefficient of earth pressure at rest, \( K_o \). The at rest value of \( K \) depends upon the geologic and stress history of the deposit but is typically in the order of 0.5. When a displacement pile is driven into sand, the earth pressure against the side of the pile builds up in intensity, and experience indicates that \( K \) may then range from 0.7 to over 2. For a low-displacement pile, it is probable that \( K \) will not alter greatly from the value of \( K_o \). When sand caves around a pile following jetting or after placement of a pile into an oversized hole, arcing of the sand will limit the lateral pressure against the side of the pile, and the resulting value of \( K \) will likely be in the range of 0.1 to 0.4. The selection of \( K \), therefore, must take into account not only the condition of the deposit prior to pile installation but also the type of pile and the method of its installation.

APPLICATION OF STATIC METHOD TO OFFSHORE PILES

The foregoing presentation has illustrated the wide range of choices available to a designer when he seeks to establish several of the necessary factors in computing the capacity of long, heavily-loaded piles. The consequences of

![Graph](image)

FIG. 10.—RANGE OF PILE PENETRATIONS COMPUTED USING DIFFERENT ASSUMPTIONS FROM CURRENT DESIGN PRACTICE

the resulting design uncertainty are emphasized by the following illustrative examples, one for a pile in deep clay and another for a pile in sand.

Shown in Fig. 10(a) are two computed curves relating pile capacity to pile penetration for a 36-in-diam pipe pile driven into a normally-consolidated clay deposit. The two curves differ because of different design assumptions in each case taken from current practice. For the upper curve, soil-pile adhesion was assumed equal to the shear strength of the clay throughout the pile penetration. For the lower curve, a similar assumption was made except that soil-pile adhesion was limited to a maximum of 0.5 ton per sq ft. Both sets of assumptions have support in available empirical evidence. The differences arise from interpretations placed on that evidence, and the differences clearly

![Graph](image)

FIG. 11.—EXAMPLE OF LOAD RANGE REPRESENTED BY A MAJOR SOURCE OF DOCUMENTED PILE LOAD TESTS, COMPARED TO TYPICAL OFFSHORE PILE LOAD

are substantial. As indicated in Fig. 10(a), the required penetrations to achieve an ultimate capacity of 2,000 tons vary from 275 ft to 475 ft according to these different assumptions.

In Fig. 10(b) a range in computed penetrations is again shown for a 36-in.-diam pile, in this case assuming the pile to be driven into a medium-dense clean sand. For both curves, the angle of internal friction was taken as 35° and the skin friction angle as 30°. For the upper curve, \( K \) was assumed to be 1.0, and a value of \( N_f \) of 94 was taken from Hansen (3). Both soil resistance factors, \( f \) and \( q \), were assumed to be directly proportional to overburden pressure, with no limit placed on either factor. For the lower curve, \( K \) was assumed to be 0.7, and the value \( N_f \) of 41 was taken from Terzaghi (13). For this set of computations, maximum limits of 1.0 ton per sq ft for unit skin
friction and 100 tons per sq ft for unit end bearing were applied. It will be noted that the range of values represented by these two sets of assumptions is less than the possible ranges indicated by Figs. 7 and 9. Still, there is a 100-ft difference in the computed pile penetrations required to achieve an ultimate capacity of 2,000 tons.

One explanation for the difficulty in making precise computations for high-capacity piles and for the absence of a design consensus can be found by evaluating the stockpile of empirical data on which the static method relies. Shown at the top of Fig. 11 is a histogram representing one of the major sources of pile test data (Peck, 9) and showing the magnitude of failure loads observed in the 178 reported tests. It is significant that most of the piles failed at loads ranging from 40 tons to 200 tons. In the lower graph in Fig. 11, the same histogram is proportionately reduced so that this representative source of empirical evidence can be compared on the same load scale with the offshore design problem to which it must be applied. A similar comparison could be made with other important groups of pile test data, such as Mansur and Focht (5), Mansur and Kaufman (7), Mansur, Hunter, and Davidson (6), Tomlinson (14), Woodward, Lundgren and Beittano (17). The design uncertainties illustrated by the two sets of curves in Fig. 10 will persist until significant numbers of documented pile load tests are performed at loads more nearly approaching the design conditions.

In spite of the lack of design consensus and the paucity of empirical data on which to base confident design, construction of deep-water platforms continues and design decisions must be made. In the writers’ design practice, the following design criteria for computing pile capacity are presently utilized in the design of heavily-loaded, driven pipe piles.

Clays.
1. Normally consolidated: $f = c$
2. Overconsolidated, with $c = 0.5 - 2.0$ tons per sq ft, $f = 0.5$ ton per sq ft, or $f = c_{o}$, whichever is greater, where $c_{o}$ = strength of normally consolidated clay $\approx 0.25 \rho_{o}$
3. All clays: $q = 9c$

Sand (medium-dense, clean).
1. $\phi = 35^\circ$, $\delta = 30^\circ$
2. $K = 0.7$ (compressive loads), or 0.5 (tensile loads), $f_{(max)} = 1.0$ ton per sq ft
3. $N'_{q} = 41$, $q_{(max)} = 100$ tons per sq ft

It should be clear from the foregoing analysis that the design factors are of necessity based in part on judgement, intuition, and fragments of experience. The authors fully expect, therefore, to modify this set of design guides from time-to-time as load test data for long, high-capacity piles becomes available and begin to provide a more definitive basis for design.

The limiting values previously given for unit friction and unit end bearing in sand will be encountered usually at a depth of about 80 ft. At bearing pressures equal to or greater than 100 tons per sq ft, the limiting value assigned in Table 1 to unit end bearing, crushing of the sand grains may occur [De Beer, (2)]. For cohesionless materials other than medium-dense clean sand, smaller friction angles and lesser limiting values of friction and end bearing are used in the writers’ practice.

All of the criteria previously outlined are intended to establish a “most probable” capacity. Because of the severity of the pile installation problem, to be presented next, they are not in every case the most conservative choice. With respect to the examples presented in Fig. 10, application of the design criteria results in the least conservative or shortest penetration for the pile in clay and the most conservative or deepest penetration for the pile in sand.

CHARACTERISTICS OF AVAILABLE PILE HAMMERS

Another area of major concern in offshore construction is the adequacy of existing hammers to produce pile penetrations sufficient to support the required heavy loads. The dimensions of the problem may be easily recognized by again referring to Fig. 4, where a pile-hammer combination typical of offshore construction is shown on the right and one typical of land construction is shown on the left. For the two piles selected for this illustration, design ultimate capacities would typically be about 2,000 tons for the ocean pile and 200 tons for the land pile, a 10-to-1 ratio. The indicated weight of the offshore pile is 1.10 tons, which is more than 35 times the weight of the smaller pile. The hammer for driving the offshore pile, however, has a rated energy only three times that of the smaller pile hammer. While hammers larger than the one referred to in Fig. 4 have been introduced in limited numbers since 1967, it is clear that the size and capacity of hardware for offshore pile installation has lagged behind the size and capacity of the piles themselves. The question arises, therefore, “under what circumstances are available hammers likely to achieve required pile penetration, and when is it probable that other means of installation will be required?”

Wave Theory Considerations.—The most satisfactory basis for studying the relative performance of pile hammers, especially when driving long piles, is a computer analysis utilizing wave theory [Smith, (15)]. This type of analysis takes into account the fact that each hammer blow produces a stress wave that moves along the length of the pile at the speed of sound, and that the entire length of the pile is not stressed simultaneously as assumed in conventional dynamic formulas.

Each computer analysis is made for a given pile-and-hammer combination. Using the modification of Smith’s procedure reported by Samson, Hirsch and Lowery (10), a study was made of the performance of the O-20 (or S-20) hammer in driving Pile A, Fig. 3, under various conditions. The pile-hammer combination is the same that is represented in Fig. 4(b). Assumptions included in the computer input and held constant throughout each analysis are illustrated in Fig. 12(a). These include the hammer efficiency, the coefficient of restitution for the cushion block, damping factors for the equivalent springs which represent soil resistance along the side and at the end of the pile, a quake factor for the pile-soil system, and the shape of the stress resistance diagram along the side of the pile. Variables considered were pile length above the mudline, $L_{u}$, pile penetration, $L_{p}$, and the ratio of soil resistance at the pile point, $R_{p}$, to total soil resistance, $R_{u}$.

For each set of conditions analyzed, results of the computer solution can be expressed in the form of a curve of hammer blows per inch of penetration.
versus total soil resistance overcome during driving. The results given by curves in Fig. 12(b) reveal that a variation of pile length above mudline from 100 ft to 400 ft, with consequent variation in pile weight from 88 tons to 154 tons, produces almost no change in the amount of soil resistance that can be overcome at a given driving rate. Similarly, results given in Fig. 12(c) and 12(d) indicate that there is little change in hammer performance for variations in pile penetration from 100 ft to 200 ft, and for variations in the soil resistance ratio $R_p/R_u$ from 6% to 75%. In all cases, total soil resistance during driving is shown to increase almost directly with blows per inch up to about 20 blows per in.; but with harder driving the resistance overcome increases only slightly.

Results of the wave theory evaluation of the O-20 hammer in combination with Pile A are summarized in Fig. 13. In Fig. 13(a), the two curves encompass the full range of driving responses for all considered variations in $L_1$, $L_2$, and $R_p/R_u$. If a driving rate of 40 blows per in. is selected to define the practical limit of pile driving, as harder driving obviously yields very little increase in soil resistance, the two curves indicate that maximum soil resistance overcome during driving will be on the order of 1,100 tons to 1,250 tons. This range of resistance is emphasized on the graph by means of the shaded band which is labeled $R_u$ (max), the pile driving limit for this pile-hammer combination. It is of interest to compare $R_u$ (max) to design pile capacity, as in Fig. 13(b), and to consider whether the pile in a given situation can be driven to the penetration required to develop that capacity.

**Pile Driving Limit Versus Design Capacity.**—In the generalized illustration of soil resistance versus pile penetration in Fig. 14, the shaded band represents the pile driving limit, $R_u$ (max), for a given pile-hammer combination. Required compressive capacity is assumed to be substantially greater than the pile driving limit, as shown by the relative position of the vertical line on the right. This relationship is typical of most deep-water construction today. The solid-line curve labeled $Q$ represents pile capacity versus pile penetration, and its intersection with the design capacity line establishes the required pile penetration. Soil resistance during driving, $R_u$, has the same units as $Q$, but it cannot at present be predetermined quantitatively as a function of penetration. Nevertheless, it is well known that when a pile is driven into soft-to-firm clay, $R_u$ will be substantially less than $Q$ because soils of this type soften appreciably under the remolding effect of the pile movement. Under this circumstance, as illustrated in Fig. 14, it is possible for the pile to reach the required penetration without first encountering the pile driving limit. Reconsolidation of the soil around the pile after completion of driving results in a gain in soil strength and supplies the additional resistance needed to attain design capacity. In this special situation, therefore, a pile hammer with a
driving limit less than design capacity may successfully drive a pile to design penetration.

In Eq. 15, a similar relationship between driving limit and design compressive capacity is assumed to exist. The pile in this case, however, is driven into a hard clay or into sand. In either of these materials, soil resistance during driving may be greater than \( Q \), the static capacity of the pile. Under this circumstance, it is clear that the driven pile will encounter the driving limit of the hammer before it reaches design penetration. The result will be a pile deficient in capacity, with a low or nonexistent factor of safety.

Design capacity for tensile loads has not been considered in the preceding paragraphs or in Fig. 14 and Fig. 15, but it is a controlling factor for many structures. The design tensile capacity will be less than the design compressive capacity; however, the static tensile capacity of a pile may be substantially less than both the driving resistance, \( R_b \), and the static compressive capacity, \( Q \), for a pile with high end-bearing capacity. In such cases, the driving limit for some pile-hammer-soil combinations may be an obstacle primarily with respect to achievement of adequate tensile capacity.

**Adequacy of Available Hammers.**—Present technology is not adequate to state positively for specific pile-hammer-soil combinations whether the pile can be driven deep enough to develop design capacity. It is possible, however, to draw some broad conclusions on the basis of results such as those summarized in Fig. 13, especially when supported by construction experience. For example, it now seems apparent that 60,000 ft-lb hammers, until recently the largest pile hammers in general use, cannot produce pile penetrations to develop ultimate capacities of 2,000 tons except when driving through soils that soften during driving and set up after driving. Although hammers with rated energies of 120,000 ft-lb to 180,000 ft-lb are now in use in limited numbers, it continues to be uncertain in many specific cases whether the largest available hammer will prove fully adequate to produce the required ultimate capacity. In situations where the rated hammer energy is only marginally adequate, it is evident that any impairment of hammer efficiency will result in an unfavorable construction situation, sometimes without the true cause being apparent.

**CASE HISTORIES**

The relationships of hammer capability, soil conditions, and design loads as described in the foregoing presentation are illustrated further by four case histories. These case histories include a wide range of pile-hammer-soil combinations and were selected from different geographic areas where there is current activity in offshore construction. For each case, construction records are presented with brief descriptions of foundation conditions and comments on both design and construction adequacy.

**Case 1.**—Installation records for two typical piles installed for a structure in the German North Sea as shown in Fig. 16. Subsurface materials consist entirely of relatively dense silty sand and sand. The 21.5-in.-diam open-end pipe piles were to be driven to 92-ft penetration to accommodate a tensile load of 200 tons. With a 140-C hammer, all piles for this structure initially refused at about 30 ft to 35 ft of penetration. After jetting was performed through the piles to an unrecorded extent, the piles were re-driven, but again firm-to-stiff clay to a level about 170 ft below the seafloor, with dense sand lying below that depth. To achieve design capacity, it was necessary that the 30-in. piles be driven to firm bearing in the deep sand. The curves labeled "blows per foot" give the driving record for two of several piles driven at this location. The discontinuities at penetrations of 75 ft and 130 ft are the result of interruption of driving to splice on additional pile sections. Following the delay in each case, a marked increase in driving resistance was encountered. For seven of the eight piles at this structure, the splice at 130-ft penetration required 3.5 hr to 4.5 hr, and all of these seven piles were successfully driven to design penetration at about 175 ft. One of the piles, shown by the dashed line, encountered an 8.5-hr delay at 130 ft. As a result of the greater delay,
this pile failed to move under 800 blows of the hammer.

For the type of soil profile illustrated here, gain in soil resistance after driving makes up for hammer deficiency. It is clear, however, that this advantage may be lost if allowed to occur prematurely, especially at a pile penetration where the hammer capacity is almost fully taxed. As a matter of interest, reinforcement of this deficient pile was achieved by driving a 24-in. insert pile to sand, and then welding the two piles together at the top.

Case 3.—In the English sector of the North Sea, a typical subbottom profile consists of hard clay with gravel and some layers of sand. Shown in Fig. 18 is the driving record for a 36-in. pipe pile driven into such a formation with the S-20 hammer. Wave equation analysis indicates that this pile and hammer combination should be capable of overcoming a soil resistance during driving of about 1,100 tons. This is an unusual situation in that the pile driving limit is greater than the design capacity of the pile, which was 1,000 tons. In spite of this relationship, the piles reached refusal at penetrations of 60 ft to 65 ft as compared to a computed design penetration of about 50 ft, resulting in an apparent deficiency in capacity of about 200 tons. Extensive and costly efforts to advance the piles further were unsuccessful and may even have reduced the capacity of these piles. This case history illustrates not only the need for larger hammers to drive piles into hard formations but also the need to avoid overconservatism in design by improvement of empirical design criteria.

FIG. 18.—INSTALLATION RECORD FOR TYPICAL PILE, STRUCTURE IN 77 FT OF WATER IN ENGLISH NORTH SEA

Case 4.—Probably more and longer piles have been driven off the Louisiana coast in the near vicinity of the Mississippi delta than in any other offshore area. The pile installation records in Fig. 19 are for a structure in that area in 280 ft of water. Soils there consist almost entirely of clay with shear strengths that are negligible near the mudline and increase almost linearly to about 1.3 tons per sq ft at 300-ft penetration. The piles for this structure were 48-in.-diam piles corresponding to Pile A illustrated in Fig. 3. Driving records for eight piles are summarized in Fig. 19(a) by the shaded band, which clearly shows the effect of splices at penetrations of 150 ft and 230 ft. Static capacity computations indicated a required pile penetration of almost 300 ft for the design capacity of 2,000 tons; at penetrations of 270 ft to 290 ft, however, driving resistance with the O-20 hammer had reached 800 blows per ft, and the full design penetration was achieved for only a few piles after sustained driving at an extremely high rate. A few piles at this site, as illus-

FIG. 19.—INSTALLATION RECORDS FOR 48-IN. DIAM PILE A, STRUCTURE IN 280-FT OF WATER ON LOUISIANA CONTINENTAL SHELF: (A) Driving resistance range for 8 piles driven with O-20 hammer; (B) Driving resistance for single pile driven alternately with O-20 and O-40 hammers
when pile driving alone is insufficient, are less than positive and are in need of significant development to permit their confident use.

PILE INSTALLATION PROCEDURES

Installation by Pile Hammers Alone.—Numerous pile installations of offshore foundations continue to be completed using pile driving alone. Some of these installations are fully successful because soil resistance during driving is low in comparison to hammer capability. Other installations of driven piles are successful because the geometry of the structure, combined with moderate deck and storm load requirements, results in required pile capacities that are well within the driving capability of available hammers. Still other structures, however, are supported by piles which fall to reach design penetration and which therefore must be expected to have less than a desirable factor of safety with respect to foundation failure.

![Diagram](image)

**FIG. 26.—RESULTS OF WAVE EQUATION ANALYSIS OF THE EFFECTS OF: (A) PILE HAMMER SIZE; (B) PILE WALL THICKNESS, ON SOIL RESISTANCE OVERCOME DURING DRIVING**

Hammers of 60,000 ft-lb rated energy, including the O-20 and S-20, are widely used. Hammers of twice that rated energy, including the air or steam operated O-40, S-40, and MRB-1500 hammers, are available in increasing numbers, and there are a few hammers of 180,000 ft-lb rated energy. Thus, in a few years, maximum available hammer energies have tripled, and the generally effective performance of these new large hammers is a noteworthy achievement. Serious consideration of the pile installation problem suggests, however, that this progress toward heavier hammers may not be sufficient to keep pace with the rapid increase in pile design loads.

Wave theory computations were made to evaluate the effect that doubling hammer energy would have on overcoming soil resistance when driving Pile A. Results are given in Fig. 20(a). The lower curve, repeated from Fig. 12, gives the driving response for a 60,000 ft-lb hammer; the indicated value of $R_a$ at 40 blows per in. is about 1,250 tons. The upper curve gives the computed response of the same pile driven by a 120,000 ft-lb hammer, all other variables remaining the same. In the latter case, the soil resistance over-

come at 40 blows per in. is 1,600 tons, an increase of only 28%. This limited effect of increased hammer size, theoretically determined, is borne out qualitatively by the experience reported in connection with Case 4. Approximate computations based on wave theory concepts indicate that hammer energies in excess of 500,000 ft-lb will be required to effectively and reliably drive piles with ultimate capacities of more than 3,000 tons in all types of soil. It is apparent that such hammers, even if feasible from a design and manufacturing standpoint, will not be available for use in the near future.

An interesting fact demonstrated by wave theory studies is that increased pile cross-sectional area is useful in achieving more effective use of a hammer's energy, even though pile weight is increased. In Fig. 20(b) the response of Pile A to driving with an O-20 hammer is compared with two other, heavier piles, one with 1-in. uniform wall thickness and another with 2-in. uniform wall thickness. Other variables were held constant for purposes of this comparison. The upper curve reveals a striking increase in soil resistance overcome during driving of the pile with 2-in. wall as compared to Pile A; at 40 blows per in., the increase in $R_a$ was 1,250 tons to 2,300 tons, or 84%. This increase is far more than that achieved by doubling hammer energy as illustrated in Fig. 20(a). Similar results were obtained by Bender, Lyons and Lowery (1) in their evaluation of wave-equation analyses applied to offshore pile foundations.

Installing Undrivable Piles.—In the absence of hammers of sufficient size to drive piles to full capacity, methods supplemental to driving must be employed to install satisfactory piles. The supplemental methods chosen must be effective in achieving greater pile capacity as compared to that of a driven pile, and the increment of pile penetration must be evidence of itself of effectively increasing pile capacity. Broadly speaking, supplemental installation methods may be divided into four categories: (1) Driving an insert pile through an initially-installed, larger pile; (2) grouting a pile into an oversized hole; (3) driving a pile concentrically with an undersized pilot hole; and (4) driving a pile with the aid of uncontrolled drilling or jetting. The following paragraphs include a discussion of the relative merits of each of these supplemental methods and a description of some of the problems associated with predicting pile capacity with each of them.

Insert Pile.—An insert pile is driven through and penetrates below the tip of a previously installed, larger diameter pile as shown in Fig. 21(a). Usually the upper end of the insert pile is welded to the initial pile near the top of the jacket leg. Insert piles provide a useful backup procedure when the larger pile cannot be driven to grade, as occurred in Case 2, Fig. 17, or when a pile fails to encounter an expected bearing stratum. Also, when modifications of an existing structure result in increased pile loads, insert piles may be used to provide the needed extra capacity if the superstructure can be temporarily removed. If a modified structure is designed to resist greater lateral loads, grouting insert piles into the existing piles can aid in obtaining extra capacity to withstand bending moments.

Partial or complete removal of the soil from within the initial pile may be necessary prior to driving an insert pile. Even then, the problem remains as to whether an insert pile can be driven to a predetermined level.

Grounded Piles.—The installation of a grounded pile includes the drilling of an oversized hole to the proposed pile tip level, placing the pile in the center of the drilled hole, and then grouting the annular space between the outside
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is an unfortunate fact that uncontrolled drilling as employed in Case 1, Fig. 16, may be a self-defeating procedure, and it is entirely possible that a pile lengthened in this manner may have less capacity than one driven to refusal at a substantially higher elevation but without jetting.

COMMENTS ON STATE-OF-THE-ART

Reflection on the factors and circumstances described in preceding sections of this paper lead the writers to the conviction that the marine construction industry suffers today from serious deficiencies in foundation design and construction technology. Little improvement in this situation appears forthcoming in the near future, and, in fact, the situation will probably deteriorate before it improves. The predicted increase in maximum water depth for ocean platforms utilized by the oil industry, from about 385 ft at present to as much as 600 ft, will bring a further increase in foundation loads and, with this increase, greater hazards of foundation instability.

The origins of this situation are of interest. One major factor is simply the rapid extension of offshore construction into deeper water and the consequent increase in pile loads. Another is the false sense of security that comes from the ring of a massive hammer when it drives a pile to refusal. A third factor is that the oil industry, which has been responsible for most marine platform construction to date, has traditionally accepted high risks in some aspects of its operations.

Offshore structure design under a high-risk policy is in noticeable contrast to bridge design practice, for example, and to most other forms of structural design. In the Gulf of Mexico, where offshore construction began, this policy has been fostered by the fact that maximum loads are always associated with hurricanes, and it is standard procedure to remove personnel before the storm strikes. This risk, therefore, is financial, and acceptance of a high degree of uncertainty or a low factor of safety is the owner's prerogative.

There is a danger, in the authors' opinion, that design and construction practices associated with acceptance of high risks will insinuate themselves into marine construction projects where high risks are not acceptable. This would include all manmade structures susceptible to maximum loading while personnel are aboard. Examples would include loaded structures in Cook Inlet, Alaska, and structures located where maximum gales can develop quickly without sufficient warning to remove personnel—as in the northern Pacific and the North Sea.

Load tests on deeply embedded piles are essential to the alleviation of this design problem. Unfortunately, even a single test of this type is extremely costly, and a significant number of tests will be required to achieve real progress. This conclusion is applicable not only to driven piles but also to piles installed by any other procedure or combination of procedures.

CONCLUSIONS

Data, analyses, and opinions have been combined here to describe the state-of-the-art of design and construction of heavily loaded pipe piles, especially as these piles are used in offshore construction. The writers' conclusions may be summarized briefly as follows:

1. The static method of computing pile capacity is the most reliable means for establishing design penetration for high-capacity offshore piles. Full-scale load tests on deeply embedded piles are needed, however, to expand the empirical base of information being used as a guide for such computations.

2. Progress in development and manufacture of larger pile hammers has been significant but has not kept pace with increasing pile loads in offshore platform construction. Much larger pile hammers, with rated energies up to three times the largest hammers now in use, are needed for some soil conditions and for maximum pile loads in current designs.

3. Wave theory computations provide a useful means of obtaining a broad evaluation of available pile hammers and of investigating the influence of many variables on pile driving capability.

4. Development of controlled drilling techniques and pile grouting procedures will assist in installation of high-capacity piles when pile driving alone is insufficient. Full-scale field load tests are required as part of this development. For the present time, design of piles installed by such measures should be conservatively executed and closely coordinated with construction procedures.

ACKNOWLEDGMENTS

The writers express their appreciation to T. J. Hirsch, who performed the wave equation analyses of the pile-hammer problem presented here; also to M. T. Davisson, for his helpful suggestions in connection with some of the concepts presented. In addition the writers are especially indebted to the many engineers and construction specialists, representing various oil companies and marine contractors, whose discussions, comments, and suggestions helped to formulate the opinions and conclusions expressed herein.

APPENDIX—REFERENCES

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DISCUSSION

Note.—This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 95, No. SM6, November, 1969.
of soil mechanics and foundation engineering, as well as the professional aspects of the practice.

The invitation to Mr. McClelland to deliver the Ninth Terzaghi Lecture is indeed a mark of the high respect with which he is regarded by his peers and, if I might speak for his colleagues who know him personally, of the affection we hold for him. His combination of an extraordinary range of knowledge and practical ability, with the total absence of pretentiousness, is appreciated by all those who work with him in his daily practice, in his business, and in his professional pursuits.

It gives me great pleasure to ask President John Rinne to please come forward with Mr. McClelland and present to him the honorarium as a token of our esteem and recognition given to a Terzaghi Lecturer.

DESIGN OF DEEP PENETRATION PILES FOR OCEAN STRUCTURES

By Bramlette McClelland, 1 F. ASCE

INTRODUCTION

In this lecture, I will discuss the design of deep penetration piles for ocean structures. Let me elaborate my subject first by stating what structures it will refer to, where they are, and why they require deeply penetrating piles.

There are many types of ocean structures. Among those that rise above the sea are: (1) Deep-water ports; (2) light stations; (3) airdromes; and eventually (4) nuclear power plants. However, the most common type of structure in the ocean is a platform used for drilling, producing, or storing oil.

The history of petroleum construction activity in the ocean is relatively short, having its significant beginning only about 25 yr ago. As recently as 15 yr ago, there was production activity adjacent to only three or four countries in the world. Today, it extends to more than 60 countries, with oil or gas having been discovered near 28 of these. It has been estimated that by 1980 over one-third of total world-wide oil production will come from the sea.

Construction activity in the oceans, for whatever purpose, is limited to the continental shelves—those gently sloping margins of continents that extend out to about 600 ft (180 m) of water. As indicated by the shaded area in Fig. 1, the continental shelves represent a relatively small part of the earth surface. The shelf area is still large, however, consisting of about 11,000,000 sq miles (28,000,000 km²), and almost 40% of this area is underlain by deep sediments. These sedimentary basins present geologic conditions favorable for oil exploration; as a corollary, foundation materials in areas of offshore petroleum activity consist primarily of clays, sands, and some gravel and boulders.

A record of previous construction on the continental shelf in terms of water depths (Fig. 2) is of interest because this is also a record of the intensification of design loads and other design requirements. Significant platform construction began in 1947 in 20 ft (6 m) of water off the coast of Louisiana. It extended steadily into deeper water until, in 1970, a platform was built in 373 ft (114 m) of water, again off Louisiana (11). In 1974 for the first time the water depth record will leave the Gulf of Mexico when a platform will be built in

Note.—Discussion open until December 1, 1974. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 100, No. G7. July, 1974. Manuscript was submitted for review for possible publication on August 6, 1973.

420 ft (128 m) of water in the North Sea. Fig. 2 suggests the probability of construction in 700 ft (210 m) of water within the next 2 yr or 3 yr. As a matter of fact, regional USGS approval has already been given for a platform in 840 ft (256 m) of water off the West Coast, and the structure has been completely designed in readiness for final approval. Independent studies by several organizations have demonstrated the current feasibility of platform construction in water at least 1,000 ft (310 m) deep.

Now returning to our subject. What characterizes piles for offshore structures? The characteristic that comes most quickly to mind is size—not only the size of the piles themselves but also the magnitude of pile loads. There is another identifying characteristic, however. This is the high degree of interaction between foundation piles and the structure they support. This is in marked contrast with the common use of piles for supporting mats and footings.

**Characterization of Pile Design Problem**

To illustrate the interdependence of foundation and structure—also to support my later discussion of pile design—I will present four examples of significant offshore structures, describing in each case the environmental loads and the foundation system employed. Our interest in the foundation system is twofold: (1) the construction process by which the piles are installed; and (2) the structural system by which the piles receive and transmit their loads.

**Gulf of Mexico Structure**—The first example is a drilling platform located about 30 miles (48 km) off the coast of Louisiana in 250 ft (75.4 m) of water (Fig. 3). Maximum design loads in this region are associated with hurricanes, and a 100-yr storm is expected to produce 57-ft (17.3-m) waves (22).

The substructure (Fig. 4) is a contemporary example of the highly successful structural system first used in 1947 off the coast of Louisiana. Commonly referred to as a tetrapod or jacket, it was prefabricated onshore, launched at sea, and upended. A length of open-end pipe piles was then inserted through each structure column and driven into the ocean floor. Additional pipe segments were added and field welded, and after the pile reached design penetration, it was welded to the tetrapod at its top. The eight main piles for this structure are 48 in. (1,220 mm) in diameter and were driven to about a 300 ft (91-m) penetration to provide the required 2,000-ton (18,000-kN) ultimate compressive capacity for each pile. As of 1972, 1,893 drilling and production platforms had been installed on the Outer Continental Shelf in the Gulf of Mexico (27), most of these being smaller structures but otherwise bearing strong similarity to the example given. Many other structures using this system have been built and are still being built in various parts of the world.

An interesting evolutionary feature shown by the structure in Fig. 4 is its use of so-called skirt piles, the tops of which are driven down to the first level of horizontal braces above the mudline. These piles are driven with a follower, then connected to the jacket by injecting cement grout in the annulus between the pile and the short jacket column. Skirt piles are used in this manner to distribute both lateral and axial loads primarily where bottom conditions are soft.

Several significant features of the tetrapod foundation system, with emphasis on its use in the Gulf of Mexico, should be noted:

1. **Piles are widely separated and react independently of each other.** The main piles in the structure are 40 ft to 60 ft (12 m to 18 m) apart, and the skirt piles are separated from other piles by about 30 ft (9 m).
2. **Because the foundation loads are highly concentrated on a relatively few**
FIG. 3—Location of Drilling Platform, Offshore Louisiana

12-well structure

**PILE LOADS**
- ULT. AXIAL CAPACITY: 4000K
- DESIGN LAT. LOAD: 250K

- 1.7 batter
- 8 main piles - 48-in. diameter - welded at top - 300-ft penet.
- 4 skirt piles - grouted in sleeves

Templet weight 2200T

FIG. 4.—Templet-type Substructure for Drilling Platform, Offshore Louisiana

FIG. 5.—Location of Drilling Platform, Cook Inlet, Alaska

30-well structure

**PILE LOADS**
- ULT. AXIAL CAPACITY: 4000K
- DESIGN LAT. LOAD: 200K

- 15-ft dia. leg, with 8 piles - 30-in. dia. - 90-ft penet. - grouted in leg

FIG. 6.—Substructure for Drilling Platform, Cook Inlet, Alaska
individual piles. Significant structure movements are expected.
3. Cyclic wave loads represent about two-thirds of maximum design loads.
4. Piles are installed in segments in order to reach the required total length. Piles for the illustrated structure were about 600 ft (180 m) in length and were assembled from six sections, requiring five field splices.
5. Significant bending moments and slope changes are induced in piles at the seafloor as a result of interaction between the piles and the templet substructure.

Later discussion will bear on each of these five considerations.

Cook Inlet Structure.—The first major variation in jacket design arose in response to new and severe environmental loads—in this case, loads produced by thick sheets of ice moving with the tide in Cook Inlet, Alaska (Fig. 5). Ice up to 42 in. (1,070 mm) thick with a crushing strength of 300 psi (2,100 kN/m²) can apply its load anywhere within the full tidal range—which may be as great as 30 ft (9 m) or more.

The structural solution for this area, as shown in Fig. 6, was to eliminate diagonal and horizontal braces where they would be exposed to ice, and to support the deck on four large-diameter columns, each in turn supported by several piles in a circular group. A further innovation, to avoid exposure to ice, was to place well-conductor pipes inside the tower legs—in fact, to use the foundation piles themselves as surface casings for the oil wells. The space between the piles and the two concentric shells forming each tower leg was grouted, as shown in Fig. 7, making the entire leg section act as a unit in bending (4).

The 12 structures of this type that were built between 1964 and 1969 have successfully resisted severe ice forces for several seasons. Similar use of the massed support of several piles beneath large structural columns will undoubtedly have other applications where loading conditions are severe.

In review, the important foundation feature of the Cook Inlet structure that is significantly different from the previous example is the use of piles in groups in contrast to widely spaced individual piles. Later discussion will consider the influence of this difference on both pile capacity and pile movement.

North Sea Structure.—The third example is another variation of the templet structure which has been designed—but not yet built—for use in deep waters. The pioneer structure of this type will be a platform due to be erected in 1974 in the Forties Field located off Scotland (Fig. 8). The water there is 420 ft (128 m) deep, and the design storm is expected to produce 94 ft (28.7 m) waves.

As shown in the substructure sketch (Fig. 9), circular pile groups as used in Cook Inlet will again be used to provide high capacity support for four main columns. In this variation, however, all piles terminate near the first level of horizontal braces above the seafloor. The portion of each pile extending above the seafloor—approximately 80 ft (24 m) in length—will be grouted into templet sleeves in a manner similar to the skirt piles previously described.

The large-diameter cylinder forming the lower section of each leg will be supported by 11 piles with a diameter of 54 in. (1,370 mm). To resist severe loads indicated, the piles will have a general wall thickness of 2 in. (51 mm), increasing to 2.5 in. (64 mm) near the mudline. Each pile will be installed in one piece, thereby avoiding all field splices, and they will be driven to an expected penetration of about 240 ft (73 m).

Foundation considerations of interest here are: (1) The continued escalation in pile loads—the total horizontal load to be carried by one leg of this structure being about twice that of a Cook Inlet structure; and (2) the total commitment to transfer of the axial load through a grouted connection between the pile and substructure. A further item of interest to the foundation engineer is the cost of a single pile in place. Based on its design weight and the commonly quoted tonnage cost of North Sea construction, each pile will cost about $500,000. Published estimates of the total structure cost, exclusive of equipment, range from $110,000,000 to $120,000,000.
Persian Gulf Structure.—The fourth structure is a radically different type called a Khazzan—and was designed for underwater oil storage. It was erected in 1969 in the southern part of the Persian Gulf (Fig. 10) where the water depth is 154 ft (47.0 m). The structure has an inverted funnel shape and no
bottom (Fig. 1), and its base is about 270 ft (82 m) in diameter. Its interior space is alternately filled with oil and then seawater.

The foundation consists of a circular ring of 30 piles installed through a flange extending out from the base of the vessel. Pipe piles having a diameter of 36 in. (914 mm) and a length of 90 ft (28 m) were grouted into drilled holes having a diameter of 42 in. (1,070 mm). Required ultimate capacities, allowing a factor of safety of two during storm conditions, range from 1,100-kip (4,900-kN) tension when the tank is full of oil to 1,220-kip (5,430-kN) compression when it is full of water (33).

Significant new foundation features here are: (1) The controlling tension load, which is sustained rather than intermittent; and (2) the use of piles grouted into drilled holes. Also, pile load is transferred to the structure through a relatively short grouted connection to the base flange. Adequate load transfer in this section was provided by use of welded beads in the top 8 ft (2.4 m) of each pile, and by use of expansive grout.

Three Khazzans are now in place and interconnected at the deck level, demonstrating the success of this highly unusual design concept.

With the foregoing description of four selected structures as a background, I would now like to consider in some detail the design of the foundations themselves. The subject can be divided into five topic areas: (1) Selection of pile size; (2) determination of pile length; (3) design of pile wall thickness; (4) prediction of pile movements; and (5) planning pile installation procedures. Each of these topic areas will be considered in turn.

**Pile Size**

Among the pile design topics I have listed for discussion, the subject of choosing proper pile size will receive only brief mention. Because of the high interaction of pile and structure, the choice of pile size for offshore structures usually is preempted by the structural designer during his initial schematic design. Some of the factors affecting his choice are worth mentioning, however. The first of these is inventory, which varies widely in different parts of the world. Next, he will want to minimize pile size in order to hold down drag forces, minimize weight, and reduce cost. On the other hand, it may in some cases be economical to increase pile size if by so doing the number of piles can be reduced to a practical minimum. The overall geometry of the structure is, therefore, a strongly controlling consideration.

The pile diameter of course must be large enough to allow steel stress and pile wall thickness to be held within proper limits. The question of driveability of a possible pile size is a soil-related factor that will frequently give a structural designer difficulty. He may have a structurally adequate pile but may also have a design load so great that the pile cannot be driven to grade with the largest available hammer. Further, if the largest available hammer has the capability, he may suffer the embarrassment of having chosen a pile size that is not large enough to support so heavy a hammer without buckling or becoming overstressed during driving.

Common pile sizes in the Gulf of Mexico today (1972) are 30 in. to 42 in. (760 mm to 1,070 mm), with a maximum of 48 in. (1,220). In the North Sea, as mentioned previously, piles up to 54 in. (1,370 mm) in diameter are being utilized.
Once pile size and type have been selected, consideration may be given to determining its required length. This is primarily a matter of designing the pile to develop adequate tension and compression capacities, giving due consideration to pile installation procedures and their influence on capacity. In my discussion of this design phase, I will be concerned primarily with driven piles, since this method of installation is the most common and is usually the most economical procedure.

Techniques for predetermining pile capacity have received a great deal of attention in the past, and with some success. Numerous approximate methods have proved quite satisfactory with piles of moderate length. These same techniques may lead to difficulty, however, when extrapolated to piles requiring deep penetration. A closer look at this problem is in order.

For piles driven into clay, most design attention for many years has focused on establishing the ratio of unit side friction $f$ to undrained shear strength $c_u$. Because this friction ratio $\alpha$ has been observed to decrease with increasing shear strength, there have been many attempts to identify this trend by correlating $\alpha$ with $c_u$.

One of the first published accounts of such efforts was by Tomlinson in 1957 (25), and his suggested correlation is shown in Fig. 12. The other correlations shown in the same figure were published subsequently by Peck (16), Woodward et al. (32), and Kerisel (10). In 1967, McClelland, Focht, and Emrich (13) pointed out that virtually all of the reported tests showing $\alpha$ factors less than one were in overconsolidated soils. In fact, if observed values of $\alpha$ were plotted against overconsolidation ratio rather than $c_u$, it is probable—as independently observed by Wroth (31)—that curves very similar to those shown in Fig. 13 would result. Then, a value of $\alpha$ of 1.0 would be correlated with an overconsolidation ratio of 1. This concept led us to conclude that $\alpha$ would have a value of 1.0 in a deep, normally consolidated clay which was stiff by virtue of large overburden pressures rather than from some source of overconsolidation. Application of this concept in many cases results in much higher estimates of offshore pile capacity than would be predicted by any of the correlations shown in Fig. 12.

Because of persistent uncertainties in this important design matter, a concerted effort was made in our office during the past 2 yr to assemble and analyze all published data for load tests on steel pipe piles in clay. Included in the data assembly were only those tests for which there was reasonably complete information on soil properties and site conditions. In addition, a survey was made of 60 engineering authorities and institutions on a worldwide basis to seek additional unpublished results of a similar character. This combined effort resulted in a total of 47 tests, involving pipe piles ranging from 8 ft to 333 ft (2.4 m to 101.6 m) in length, 6 in. to 30 in. (150 mm to 760 mm) in diameter, and with total capacities ranging from 3 tons to 900 tons (27 kN to 8,000 kN). Results of these tests were then used to reexamine the adequacy of the several proposed criteria for load transfer in clay.

In Fig. 13, the ratio of predicted capacity to observed capacity is plotted against depth for the 47 tests. Predicted capacities for this plot were computed using Tomlinson's criteria as modified in 1970 (26). Indicated values greater than one represent overestimates of capacity; those less than one indicate underestimates. Predictions accurate to within ±10% plot within the shaded zone. The scatter suggests that the criteria are less than satisfactory.

Similar comparisons were made using the other correlations shown in Fig. 12, and the quality of the comparison was found to be similar. The evaluation was extended to the 1969 criteria of McClelland, Focht, and Emrich (Fig. 14) and the results again were disappointing.

**FIG. 15.—Shear Strength and Water Content Variations Between Two Driven Piles**

**FIG. 16.—Frictional Capacity Coefficient, $\lambda$, for Estimating Pile Capacity in Clay** (30)

It is not surprising that there is a growing dissatisfaction with attempts to solve this problem through correlations of $\alpha$ with $c_u$. This is accompanied by a growing conviction that pile support in clay is frictional in character—that load transfer is dependent on the effective lateral pressure acting against the side of the pile after it is driven.

When a pile is driven into clay, pile displacement produces large shearing strains in the surrounding soil with consequent remolding and loss of strength. This is accompanied by development of excess pore pressures which then dissipate with time. This process results in a decrease in water content and an increase...
in strength in the soil near the wall of the pile. Observations made by Flaae (6) of soil properties between two driven piles 5-1/2 yr after their installation demonstrate vividly the correctness of this concept. His results obtained at a 6.7-m (22-ft) depth are shown in Fig. 15 and are quite similar to those obtained at both greater and lesser depths.

Clearly, the amount of strength increase adjacent to the pile at any depth, and therefore, the load transfer capacity at that depth, is dependent on the intensity of soil pressure producing these changes. While this encourages use of an effective pressure approach when predicting load transfer capacity in clay, there remains the problem of how to estimate the magnitude of the lateral pressure.

My colleagues, Vijayvergiya and Focht (30), suggested that soil displacement during pile driving would mobilize passive pressure in soil surrounding the pile, and that subsequent lateral pressure acting on the side of the pile would be influenced by this passive resistance. Accordingly, they suggested that a correla-

![Diagram of pile penetration and friction capacity coefficient](image)

**FIG. 17.**—Correlation of Friction Capacity Coefficient with Pile Penetration

![Graph showing predicted vs observed capacities](image)

**FIG. 18.**—Comparison of Predicted and Observed Capacities of Piles in Clay, Criteria of Vijayvergiya and Focht

...tion might be found between pile friction $f$ and Rankine passive pressure $\sigma_m + 2c_m$, in which $\sigma_m$ is the effective vertical pressure. Unfortunately, available test pile data are insufficient for investigating this possible correlation. The authors, therefore, proposed a modified relationship—one which could be tested. According to this concept, as shown in Fig. 16, total friction capacity $Q_f$ can be obtained from

$$Q_f = \lambda (\sigma_m + 2c_m) A_s$$  \hspace{1cm} (1)

in which $\lambda =$ friction capacity coefficient; $\sigma_m =$ mean effective vertical pressure for depth of pile embedment; $c_m =$ mean undrained shear strength for depth of pile embedment; and $A_s =$ surface area of the pile. A single value of $\lambda$ can be computed from the results of any pile load test where only the load at the head of the pile is observed and where the indicated soil properties are available. It is significant that determination of the average effective vertical pressure requires knowledge of the position of the ground-water level at the time of the test.
In Fig. 19 are plotted curves for the load tests previously referred to are plotted against pile penetration (30). The correlation appears to be quite good, especially when one considers the wide range of pile lengths, pile sizes, and soil types represented in the sample. Capacities of the test piles were computed using the correlation line shown in Fig. 17, and corresponding ratios of predicted to observed capacities for the assembly of test piles were again plotted against depth as shown in Fig. 18. Because of the evidently greater reliability of this method for predicting total pile capacity, our office has now adopted its use as a standard design procedure.

Although the method appears to be successful empirically, it does leave something to be desired. In the first place, it is inconvenient to use, particularly in situations where clay layers are interbedded with sand layers. Second, we still have a less-than-adequate understanding of the soil mechanism that controls load transfer. Continued study of pile friction in clay is needed.

For computing the capacity of piles in sand, load transfer on the sides of the pile has traditionally been evaluated as a frictional resistance using

\[ f = K \delta \tan \delta \]  

in which \( K \) = coefficient of lateral pressure and \( \delta \) = friction angle of soil on steel. The magnitude of \( K \) depends on the state of stress in the sand deposit prior to pile installation, as well as changes in the condition of the deposit that result from pile installation.

Values of \( K \) for design are chosen on the basis of empirical observations drawn from many sources. For example, information on the effects of installation procedures was obtained as part of a study performed by our staff on the pull-out capacity of pipe piles. Included in the program were tests on four piles 20 in. (510 mm) in diameter, each installed by a different procedure to

<table>
<thead>
<tr>
<th>Test pile</th>
<th>Gross, in tons</th>
<th>Net, in tons</th>
<th>Percentage of driven pile capacity</th>
<th>Coefficient, ( K ), of lateral pressure, ( K ) for ( \delta = 30^\circ )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>51.8</td>
<td>47.0</td>
<td>100</td>
<td>0.4</td>
</tr>
<tr>
<td>B</td>
<td>41.7</td>
<td>38.1</td>
<td>81.1</td>
<td>0.33</td>
</tr>
<tr>
<td>C</td>
<td>27.0</td>
<td>24.3</td>
<td>51.7</td>
<td>0.20</td>
</tr>
<tr>
<td>D</td>
<td>5.0</td>
<td>5.0</td>
<td>11.3</td>
<td>0.05</td>
</tr>
</tbody>
</table>

*Adjusted for submerged weight of pile and soil plug within the pile.

Direct shear tests for sand-on-steel indicated \( \delta = 30^\circ \) to 33° for a relative density ranging from 70% to 100%.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( \delta ) in degrees</th>
<th>( f_{\text{max}} ) in tons per square foot</th>
<th>( N_d )</th>
<th>( q_{\text{max}} ) in tons per square foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean sand</td>
<td>30</td>
<td>1.0</td>
<td>40</td>
<td>100</td>
</tr>
<tr>
<td>Silty sand</td>
<td>25</td>
<td>0.85</td>
<td>20</td>
<td>50</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>20</td>
<td>0.7</td>
<td>12</td>
<td>30</td>
</tr>
<tr>
<td>Silt</td>
<td>15</td>
<td>0.5</td>
<td>8</td>
<td>20</td>
</tr>
</tbody>
</table>

a penetration of 48 ft (14.6 m) into a uniform beach sand deposit. Located on Padre Island, a barrier reef off the Texas coast, this deposit consisted of fine, poorly graded, medium dense sand.

Load-deformation curves for the load tests are given in Fig. 19, accompanied by sketches identifying the installation procedures for each pile. As indicated, Pile A was installed by driving only and yielded under a tension load of 51.8 tons (461 kN). Pile B was initially driven 34.5 ft (10.5 m) with the simultaneous aid of a rather unique jet located a few inches above the tip. All of the return flow of sand and water remained inside the pile and was discharged from ports located above ground level. This pile was then driven without jetting to final
grade at when loaded, yielded at 41.7 tons (362 kN) tension. Pile C was installed to 41.5 ft (12.7 m) by jetting with return flow outside the pile, followed by driving, and Pile D was installed entirely by jetting in the same manner. Piles C and D failed at loads of 27.0 tons and 7.0 tons (240 kN and 62.3 kN), respectively. All loads were applied within 2 to 4 days after pile installation.

Table 1 is a summary of the significant results of the tests. The effects of jetting are seen to drastically reduce the frictional capacity of the piles. Jetting without redriving is indicated to be quite unacceptable in terms of resulting pile capacity. Although the computed $K$-values are directly applicable only to the specific conditions which were obtained in these tests, it is of interest that they indicate an average $K$ of 0.4 for the driven pile under tension loading, and 0.2 for the redriven jetted pile.

The results of such previous investigations and the experiences and opinions of many engineers have been combined in brief criteria published by the American Petroleum Institute (2) for design of pipe piles in sand, when used to support offshore structures. These criteria are based on Eq. 2 for pile friction and the following equation for $q$, the unit end bearing resistance:

$$ q = \sigma_N q $$

in which $N_q$ is a bearing capacity factor. Table 2 gives values of $\delta$ and $N_q$ recommended by the criteria for different gradations of sandy soils. Eq. 2 and Eq. 3 carry the suggestion that both of these components of capacity increase directly with depth or effective vertical pressure. Important studies by Vesic (28) and Kerisel (9), both published in 1964, and the results of other investigators since that time show that these components of capacity increase linearly to only a limited depth, below which they increase only very slowly or not at all. The API criteria give recognition to those studies by assigning limiting values to both friction and unit end bearing as indicated in Table 2. These limiting values are usually reached at a depth of about 80 ft (24 m) below the seafloor.

An important exception should be noted. These criteria, and the bulk of experience on which they are based, pertain to piles installed in silica sand. There are a number of places on the continental shelves where there are sands of greatly different composition. These are the carbonate sediments, examples of which are illustrated in Fig. 20. Such sands may have a familiar grain-size distribution and an angle of internal friction of 35$^\circ$ or more—not unlike silica sand, but experience shows that they will provide vastly inferior support for driven piles.

Studies have shown that the conditions favoring carbonate deposition in the world oceans exist between 30$^\circ$ N lat and 30$^\circ$ S lat (Fig. 21) (19). Within that belt, warm temperatures prevail and the water is saturated or in some cases supersaturated with calcium and other carbonates. Enormous quantities of carbonate ooze are collecting at the bottom of the deep oceans within that zone, but additionally there are significant deposits in shallow waters on the continental shelves. Some of these, for example, occur in North America off the west coast of Florida, and in Central America off the Yucatan peninsula. Other large deposits, in each case coinciding with some oil and gas exploration or production activity, include the Bass Straits at the southern tip of Australia, the continental shelf off western Australia, and the southern part of the Persian Gulf.

The sand in Fig. 20(a) consists of ooliths, rounded and highly polished particles of calcium carbonate in the fine-to-medium size range. Oomets are quite solid and are formed by chemical precipitation in highly agitated waters. This sample was obtained in shallow water at the south end of the Persian Gulf. The specimen in Fig. 20(c)
This maximum skin friction is one-fifth of that given in Table 1 for clean sand, and the maximum end bearing is one-half of the corresponding value in Table 2. On the basis of the very limited available data to date, it is our practice to estimate the skin friction of a grouted pile in the same manner as given in Table 2 for a driven pile in silica sand. Extensive full-scale load tests, with careful identification of formation properties, will be required to establish more comprehensive and reliable criteria. Because of the highly variable nature of carbonate deposits, any early attempts to generalize the results of limited full-scale tests must be considered highly speculative.

As a closure to this section on design of pile lengths to achieve required pile capacity, Fig. 22 shows the typical end product of this phase of pile design analysis. Results are typically expressed in terms of ultimate capacity and are given for a range of pile penetrations. This allows application of the results to a variety of pile loads that may be considered during final design. It is also useful in considering the effect of some necessary changes in pile length that may arise as a result of construction problems. Factors of safety applicable to results expressed in this manner vary usually from 1.5 for the condition of maximum environmental loading to 2.0 for normal operating conditions (2).

**Pile Wall**

The phase of pile design directed toward establishing pile wall thickness depends primarily on analysis of bending stresses developed in the pile as a result of wave and other lateral loads. Lateral loads on most offshore structures are...
of such magnitude that complete solutions are needed. Approximate
simplifying concepts, such as the assumption of a point of equivalent fixity,
are totally inadequate. (1) The solution must be fully responsive to boundary
conditions imposed on the pile by the structure itself; (2) it must be capable
of dealing with piles of varying moment of inertia; and (3) it must recognize
the nonlinear response of the soil to the applied load.

To illustrate this phase of the design, I would like to present the first end
result of a successful design analysis, and then describe the techniques used
to solve this and other similar problems.

From the introductory part of this paper, you will recall my description
of a Gulf of Mexico platform in 280 ft (85.4 m) of water, supported by piles
48 in. (1,220 mm) in diameter penetrating 300 ft (91 m). Fig. 23 depicts how
a pile design might look for that structure, in order to support storm loads
of 2,670 kips (11,900 kN) compression and 250 kips (1,120 kN) shear at the
mudline. You will note several variations in wall thickness: first, an upper section
that is 1-in. (25-mm) thick; then a section near the mudline that is 1-1/8 in.
(28 mm) thick; and then steps down to 5/8-in. (16-mm) thickness in the lower
portion of the pile.

Computations were made assuming the soil to consist of normally consolidated
clay with mudline shear strength of about 100 psf (4.8 kN/m²), increasing
almost linearly with depth below the mudline. Analysis determined that the
bending moment in the pile under a 250-kip (1,120-kN) lateral load would be
as shown by the curve in Fig. 24. One further condition of the solution should
be noted. This is consideration of additional section modulus and stiffness
produced by a 33-in. (835-mm) dia jacket leg penetrating 15 ft (4.6 m) below
the mudline, acting integrally with the pile as a result of cement grout placed
in the annulus between the two pipes.

The shaded area in the graph in Fig. 24 represents the available moment
capacity of the pile at various points along its length. This diagram was developed
after reserving part of the allowable stress for the axial load.

You will note that the applied moment at all depths is equal to or less than
the available moment, although it gets rather close at two points. Before concluding
that this is a satisfactory design, consideration must be given to the possible
adverse effects of a pile failing to reach its design penetration or being driven
deepener than planned for some reason. It is common practice to provide sufficient
wall thickness to allow for 20 ft to 30 ft (6.1 m to 9.2 m) of variation each
way in pile penetration without failing to meet the moment requirement.

The difference-equation method used for this analysis has been developed
through the work of several authors but it became a powerful and practical
analytical tool when it was generalized for computer solution by Matlock and
Reese (15). In the solution, as shown in Fig. 25, the pile is assumed to be
divided into segments, each of which experiences lateral deflection y and soil
resistance p in units of force per unit pile length. Part of the input for the
solution includes restraint at the head of the pile consistent with the supported
structure, and elastic properties of the pile itself. Additional input must describe
the relationship of soil resistance to pile deflection in a realistic way, which
means taking into account the nonlinear behavior of most soils.

Beginning some 15 yr ago and extending over a period of many years, extensive
lateral load tests were carried out on large-scale model pipe piles in clay under

FIG. 24.—Example of Lateral Load Analysis for Pile in Fig. 23 and Structure in Fig. 4

FIG. 25.—Elements in Analysis of Laterally Loaded Piles by Difference Equation
Method
the sponsorship of five oil companies who recognized the importance of this design problem. One of the important results of the program was the development of criteria for the p-y curves applicable to laterally loaded piles in normally consolidated or moderately overconsolidated clays. Some of the important features of these criteria, as presented by Matlock (14), are shown in Fig. 26. The diagram gives dimensionless load-deformation relationships (or p-y curves) for both static and cyclic loading. Ultimate soil resistance \( p_u \) is used to normalize the vertical axis, and deflection at 50% of ultimate stress, \( \gamma_{0.5} \), is used to normalize the horizontal axis. These values are determined using the equations within Fig. 26, in which \( N_p \) is a nondimensional ultimate resistance coefficient with a value of about nine; \( d \) is the pile diameter; and \( \epsilon_i \) is strain at 50% of maximum stress in the laboratory test used to determine \( \epsilon_{u,i} \), the undrained shear strength.

An important result of the tests reported by Matlock is that repetitive or cyclic loads produced a softening of the soil resistance, the peak resistance after cycling being only about 0.7 of the resistance offered a single static load. Furthermore, the resistance at large strains decreased significantly in the soil near the mudline. According to the criteria based on those results, the p-y curve for a depth \( x = 0 \) follows the lower boundary of the shaded area in Fig. 26, with \( p \) becoming zero at large deflections. Below critical depth \( x_i \), which the author defines as a function of pile diameter, soil density, and soil strength, the applicable p-y curve follows upper boundary of the shaded area.

Less complete information is available for establishing p-y curves for laterally loaded piles in sand. Simply expressed criteria are recommended for this purpose, however, in the API reference previously quoted (2). This recommendation (Fig. 27), based in part on Terzaghi's analysis of soil modulus problems (24), is to treat the soil as an elastic-plastic medium. For the elastic phase, the ratio of \( p \) to \( y \) is given as a density coefficient \( A \) times the effective vertical pressure, divided by 1.35 \( d \). Values of \( A \) range from 200 for loose sand to 2,000 for dense sand. The plastic part of the p-y curve is established by a limiting soil resistance \( p_u \), which is three times the Rankine passive earth pressure times the pile diameter. In the opinion of most designers familiar with this problem, actual sand response will be stiffer than indicated by these criteria, and, as a result, deformations and moments determined using them will probably be somewhat on the high side.

**Pile Movement**

In addition to sizing a pile for diameter, length, and wall thickness, it is frequently of critical importance to predict the character and magnitude of its movement under load.

One of the principal needs for describing pile response is to provide the structural designer with suitable boundary conditions for his analysis of stresses and deflections of the various structural members of the temple, such as those shown in Fig. 4 and Fig. 9. In performing a stress analysis of the frame, the structural engineer will usually substitute a structural model for the real pile as shown on the right-hand side of Fig. 28. This will consist of a short length of dummy pile, connected to three independent springs that characterize the pile reactions. One spring simulates the axial load response of the pile-soil system, another reproduces the lateral resistance, and the third gives the moment-rotation characteristic of the embedded pile.
Individus.—Separate analyses of a pile model as indicated in the lower sketch of Fig. 28 can provide the components of pile response needed as boundary conditions for frame analysis and for other purposes. Using a lateral load analysis of the type previously described, curves of lateral load versus deflection at the pile head can be developed to define the lateral spring, and moment-rotation values can be obtained from the same source.

Less fully developed methods are available for numerical definition of an equivalent axial spring, i.e., a curve of axial load $P$ versus deformation of pile head $\delta_A$. Finite element analyses may eventually provide useful answers. In the meantime, there are two general approaches. These have been called

![Diagram of pile analysis](image)

**Fig. 29.—Transfer Function Approach to Movement Analysis of Axially Loaded Pile**

the elastic solid approach and the transfer function approach (29). The elastic solid approach is based on solutions of Mindlin’s equations for stresses and displacements in a semi-infinite solid under the action of interior point loads. This requires assigning a constant modulus of deformation to the soil and does not appear well-suited to deeply penetrating piles. In the transfer function approach, as shown in Fig. 29, the pile is assumed to be divided into short segments. Each segment is treated as a short compressible column and is assumed to transfer a part of its axial load to a nonlinear spring that simulates the surrounding soil. With suitable load-transfer characteristics assigned to these springs, a computer solution can produce a load-deformation curve for the pile head similar to the results one would obtain with a load test. Coyle and Reese (3) developed this type of solution fully and recommended a family of load

![Graphical representation of load transfer](image)
transfer curves, now commonly referred to as "t-z curves," for use with piles in clay. However, they limited their recommendation to piles penetrating less than 100 ft (31 m).

At an earlier ASCE convention, McClelland and Lipscomb (12) presented results of a load test conducted in the Gulf of Mexico on a steel pipe pile having a penetration of 333 ft (101.6 m), and the load deformation curve for that pile is reproduced in Fig. 30 as a solid line. Also shown is a dashed curve that was computed using the transfer function approach just described. The form of t-z curve used for the solution is shown in an inset. Assumed peak load transfer value $t_0$ is $\sigma_0/\gamma$ plus $c_0/2$, and the postpeak stress is 80% of $t_0$. The fundamental correctness of this relationship is not claimed. Its usefulness is being investigated, among others, and is shown here for purposes of illustration. In this situation, it does produce results that compare favorably with the test pile results.

The same technique was used to compute the load deformation for the 48-in. (1,220-mm) diam, 600-ft (180-m) long pile previously described in connection with the first structure illustration (Fig. 4). Similar t-z curves were assumed to describe soil resistance. In Fig. 31, the upper curve gives settlement of the pile at the mudline, and indicates the performance of a pile grouted into its jacket leg. Deformations given by the lower curve include the effect of elastic shortening of the pile between the mudline and top of pile, and therefore, are descriptive of piles connected to the jacket by welding at the top. An applied storm load of 2,670 kips (11,900 kN) is seen to produce 1.5-in. (38-mm) deformation for the grouted pile and 3.7 in. (94 mm) for the pile welded at the top. Clearly, either movement could have significant influence on jacket stresses, and the difference between the two movements suggests that the method of connection to the jacket merits close study in preliminary design analysis.

Pile movements have added significance when structures are interconnected as in many production complexes. As an illustration, Fig. 32 shows the computed lateral movement of a 3-leg jacket in 230 ft (70.1 m) of water, used for a bridge support between structures in the North Sea. The predicted 14 in. (356 mm) of lateral movement is made up of three major components: one part is horizontal translation of the structure, equivalent to lateral deflection of the piles at the mudline; another part comes from rotation of the structure, the result of axial shortening of some piles and lengthening of others; and the third component results from flexure or distortion of the tripod itself.

In the illustrated case, the distortion component proved to be the largest. With a wide-based production platform, the translation movement could be the greatest. Having adequate tools to investigate and describe pile response and performance is obviously of importance in design treatment of such connecting structures.

Pile Groups.—The problem of movement prediction becomes more complicated and, at present, more uncertain when dealing with pile groups, as with the Cook Inlet and North Sea structures shown in Figs. 6 and 9.

Considering vertical movement first, the axial shortening of piles in a group may not be greatly different than that of a single pile under the same pile load. However, the soil movement beneath and around the group is certain to be greater because of overlapping effects of the different pile loads, and therefore, the total movement of the pile group at its top will be greater. Thus, the pile group will be represented by a "softer" spring compared to an individual pile carrying the same pile load.

Model tests on piles in clay confirm that the vertical movement of pile groups is greater than that of single piles under equal pile loads. Model tests by Sowers on pile groups in clay found that group settlement could be as much as six times that of individual pile settlement (21).

![FIG. 32.—Computed Lateral Movements of Bridge Support, North Sea](image)

![FIG. 33.—Increase in Lateral Deflection due to Group Effect](image)

In the absence of definitive data from axial load tests on full-scale groups of long, compressible piles, estimates for offshore pile groups have been made by approximate rational methods, guided by the results of model tests and of full-scale tests on short piles. Results of several such studies have indicated ratios of group deflection to single-pile deflection in the range of 1.5 to 3.0.
Research is required in this area to permit more accurate predictions of group effect.

Similar design consideration must be given to the performance of pile groups under lateral loads. As shown in Fig. 33, the total deflection of the group, \( y_d \), will be greater than that of a single pile, \( y_p \), under the same pile load, and the difference between the two, \( y_y \), is the increment of deflection attributable to group action. A numerical method for computing group response at the ground surface has been provided by Poulos (17) for an idealized elastic soil, and could be used for estimating group deflections resulting from stresses of low intensity typical of onshore pile foundations. Also applicable to modest loads and small deflections are studies by Prakash (18) and Tamaki et al. (23) that link group behavior to the geometric relationship between the group and a single pile, without regard to soil type or properties. However, as we have already seen when considering single pile behavior, large loads on offshore piles usually produce large strains, and the single pile deflection, \( y_p \), results partly from plastic yielding in a zone adjacent to the pile and partly from elastic strain where stresses are less intense. While there is an empirical method for determining \( y_p \) that combines both of these responses, there is no empirical method for predicting group behavior which recognizes the disproportionate effect of plastic deformation when large lateral deflections occur.

As an interim procedure, Focht and Koch (7) have proposed a rational method for predicting lateral deflections of offshore pile groups. Assuming one pile to be missing from the group, consideration is given first to the effect all other piles will have on the soil at the position of the missing pile. Except for a closely spaced group, this movement is likely to be within the elastic range, and therefore, can be determined by computer solution using the Poulos method. This computed displacement is assumed to provide an estimate of the incremental group deflection, \( y_y \), previously defined. This deflection can then be combined with single pile deflection, \( y_p \), computed by the empirical \( p-y \) procedure that includes the effect of local yielding. The principle of superposition is somewhat strained by this heterogeneous solution, but the technique has some rational basis and is the most satisfactory tool now available. The Focht and Koch method includes a technique for estimating a moment diagram for the most heavily loaded pile in the group, which is essential for the selection of pile wall thickness.

In one analyzed case involving a circular group of 11 54-in. (1,370-mm) piles on a 40-ft. (12-m) circle with an average lateral load of 160 tons (1,400 kN) per pile, lateral deflection of the group was estimated to be greater than 8 in. (200 mm), which is more than twice the equivalent individual pile deflection. In several cases, analyzed all generally circular, the group movement ranged from 2.2 to 3.4 times individual pile movement, and computed maximum bending moments were 40% to 70% greater than would be anticipated for single piles under comparable loads.

**Pile Installation**

After the requirements of pile and structure design have been dealt with fully, there remains the task of successful pile installation. Here again, size is a dominant factor—both the physical size of the pile member and the large load capacities that must be achieved. Installation by driving remains the preferred method if it can be carried to completion without supplemental procedures. In its infinite offshore
illusions has since been dispelled. In today’s environment of continually increasing peak loads, concern is shared by design and construction people alike as to how well a given design can be executed.

For any given combination of pile and pile hammer, the maximum soil resistance that can be overcome during driving has a rather narrow range. This has been demonstrated using computer analyses based on one-dimensional wave theory. Results of one case study (13) involving the 48-in. (1,220-mm) pile shown in Fig. 23, driven by an O-20 hammer, are shown in Fig. 34. The two curves give upper and lower limits of soil resistance for various driving rates, expressed in blows per foot, and for wide variations in pile lengths above and below the mudline. The practical driving limit in this case was 75 ft (23 m). The required static capacity of 2,000 tons (18,000 kN) identified for this same pile in Fig. 4. When required pile capacity is greater than the practical driving limit for a given pile and hammer combination, then it must be achieved either through soil set-up that occurs after driving, or by use of a larger hammer, or by some other means of installation.

Fig. 35 presents driving records for eight piles installed in the Gulf of Mexico for a platform similar to the first structure illustration (Fig. 4). The 600 ft (180-m) long, 48-in. (1,220-mm) piles were driven with an O-20 hammer and almost

Fig. 36.—Record of Increase in Maximum Rated Energy of Available Offshore Pile Hammers

The practical driving limit in this case was 75 ft (23 m). The required static capacity of 2,000 tons (18,000 kN) identified for this same pile in Fig. 4. When required pile capacity is greater than the practical driving limit for a given pile and hammer combination, then it must be achieved either through soil set-up that occurs after driving, or by use of a larger hammer, or by some other means of installation.

Welding of splices is a significant factor in this connection. Splicing of large piles is usually accomplished by four or five welders working simultaneously. About 4 hr are required to weld a 48-in. (1,220-mm) pile of 1-in. (25-mm) thickness, and the time increases to about 7 hr for a thickness of 1-1/2 in. (38 mm). This is one reason that thicknesses greater than 1-1/2 in. (38 mm) are used with reluctance by designers.

Fig. 37.—Drilling and Underreaming Phase of Grouted Pile Installation, Oil Storage Structure in Persian Gulf (33)

To be less dependent on set-up after driving, and to achieve capacity in those soils that do not provide set-up after driving, there has been a steady increase in the capacity of hammers designed especially for this purpose. An
History of this development is given in Fig. 36. In the infancy of offshore construction, hammers such as the 200C and the S-14 were the largest available, with rated energies in the range of 40,000 ft-lb to 50,000 ft-lb (54 kN-m to 68 kN-m). The first truly offshore hammers came into use around 1955. These were the S-20 and O-20 with rated energies of 60,000 ft-lb (81 kN-m). Several larger hammers in the range of 120,000 ft-lb to 180,000 ft-lb (160 kN-m to 240 kN-m) were put into use beginning around 1965. Other still larger hammers are now available as shown, and beginning next year use is to be made of the M 7000 hammer that has a rated energy of 615,000 ft-lb (834.6 kN-m). At least four of those monster hammers are expected to be working in the North Sea.

Despite the relatively rapid increase in the size of offshore hammers, other means of pile installation are required and as a matter of fact are in use. Two brief but significant examples will be given.

As Woes and Chamberlin (33) described, in connection with the Persian Gulf structure for underwater storage of oil (Fig. 11), the piles there are 36-in. (910-mm) diam pipes grouted into 42-in. (1,070-mm) diam holes. As shown in Fig. 37, each pile was first inserted through an opening in the floor of the structure and extended into the bottom to a very limited penetration. A hole was then made beneath the pile by rotary drilling using an underreamer and seawater. Subsequently, the pile was lowered to its design penetration and neat cement grout was pumped to displace the drilling water. A separate grouting operation was used to interconnect the pile to the structure flange. In this case, drilled and grouted piles were used not because of difficulty in driving piles but rather to develop a high capacity pile in the carbonate sediments of the area.

At a structure site off Norway in 230 ft (70.2 m) of water, high compression and tension capacities were required, and the existence of a very dense sand underlain by a very hard clay made pile driving a questionable solution. As described by Ehlers and Bowles (5), the novel device used in this case was to drive a 42-in. (1,070-mm) diam pipe pile to a penetration of 85 ft to 100 ft (25.9 m to 31 m), then to drill and underream a footing of 9-ft to 14-ft (2.7 m to 4.3 m) diam in the hard clay beneath the pipe. A cross section of the foundation element is shown in Fig. 38. A controlled drilling mud was used in conjunction with this drilling operation. Steel reinforcing was set in the bottom 50 ft (15 m) of the pile, and sand-cement grout was used to displace the drilling mud within that same section of the pile. As a result of careful planning the operation was carried out within the time originally anticipated, and further use of this technique will probably be made in special cases where it appears to be warranted.

**Closure**

It is hoped that this discussion has provided some insight into offshore pile design from the viewpoint of a practicing foundation engineer. In closing, I would like to make some general observations. Rather than attempt a summary of my rather wide-ranging talk, I would like to comment on some of the offshore industry's accomplishments and some of its needs within today's subject area.

In this context, I include our profession within the industry.

With respect to needs, the most stringent of these can be attributed to the rapidly advancing development of ocean structures—an advance that is being met by larger derrick barges, heavier pile hammers, better welding techniques, etc.—but which also requires sharper answers to questions of foundation feasibility and closer predictions of foundation performance.

For example, let me quote my own remarks from the text to earlier portion of this lecture, concerning:

1. **Axial Load Capacity**—“We still have a less than adequate understanding of the soil mechanism that controls load transfer,” and “The fabric of in-place carbonate sediments warrants further study.”
2. **Laterally Loaded Piles**—“Less complete information is available for establishing p-y curves for laterally loaded piles in sand.”
3. **Axial Deformations**—“Less fully developed methods are available for a numerical definition of an equivalent axial spring.”
4. **Group Behavior**—“The problem of movement prediction becomes mor
The offshore industry, including my colleagues in engineering, and, at present, more uncertain when dealing with pile groups," and "In the absence of definitive data from axial load tests on groups of long, compressible piles, estimates for offshore pile groups have been made by approximate rational methods." Also, "There is no empirical method for predicting group behavior (under lateral loads) which recognizes the disproportionate effect of plastic deformation when lateral deflections occur."

These are real and pressing needs, and of course there are others. The impressive characteristic of these needs, in my opinion, is that they all require major testing programs, preferably field testing at full scale in many cases or, at the very least, model tests of large scale; and, in every instance, with thorough instrumentation.

Because of the very high cost of such programs, the problem of funding and sponsoring them has been and continues to be a problem. Although the first beneficiary of improved design will be the industry owners—that is the oil companies—the main thrust of their research budgets is towards methods of finding, producing, and processing petroleum.

Let me hasten to say, however, that the needs have not gone entirely unattended by the industry. Major engineering research and development efforts addressed specifically to offshore design needs have in fact been undertaken repeatedly in the past 15 yr, and these efforts have produced major results. These studies have had limited sponsorship by a handful of companies responsive to the need, and this has led to a secondary problem—the traditional proprietary, and therefore, confidential treatment of costly information that was hard to obtain. Substantial studies have been made of the response of laterally loaded, single piles in soft clay, in hard clay, and in sand. Research on grouted connections, on behavior of pile groups, on axially loaded piles in clay and sand, and many related topics and subtopics have been investigated. Portions of the results have reached the literature; others have not. In the literature, for example, is the previously quoted reference to Matlock (14) that included P-y crit.ria for the design of laterally loaded piles in clay. Those results were from a study undertaken by a single oil company that was subsequently joined by four others as sponsors.

Broader financial sponsorship of needed work not only will bring attention to the unsolved problems but will avoid the natural reluctance of the lone sponsor, which is to say the lone spender, to give away the fruits of his efforts to others less willing to face the need.

There is room for cautious optimism with regard to this problem. A study of group behavior of piles in clay, undertaken 2 yr ago by one oil company, has now been extended and broadened with 16 additional sponsors supporting the effort. This should assure public dissemination of the results in the reasonably near future. It is my hope that this program will serve as a model for voluntary cooperative effort among responsible members of the industry, to balance the industry's remarkable progress in other areas with significant improvement in foundation design techniques.

**Acknowledgments**

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APPENDIX II. Notation

The following symbols are used in this paper:

- \( N_f \) = bearing capacity coefficient;
- \( P \) = applied load at head of pile, in tons;
- \( p \) = total static axial load capacity, in tons;
- \( \rho \) = ultimate lateral soil resistance, in pounds per square foot;
- \( Q \) = total end bearing capacity, in tons;
- \( Q_f \) = total frictional capacity, in tons;
- \( q \) = unit end bearing, in tons per square foot;
- \( R_f \) = total soil resistance during pile driving, in tons;
- \( R_s \) = soil resistance at pile tip during pile driving, in tons;
- \( t \) = frictional stress on pile, in tons per square foot;
- \( \sigma_{eff} \) = maximum frictional stress on pile, in tons per square foot;
- \( x \) = vertical distance below ground surface, in inches;
- \( y \) = critical depth for surface effects, laterally loaded pile, in inches;
- \( y_v \) = lateral pile deflection, in inches;
- \( y_t \) = lateral pile deflection at \( p_u \), in inches;
- \( y_{def} \) = lateral deflection due to group effect, in inches;
- \( y_s \) = lateral deflection of single pile, in inches; and
- \( y_{ax} \) = axial pile movement at depth \( x \), in inches.

- \( \alpha \) = friction ratio;
- \( \beta \) = friction angle of soil on steel;
- \( \delta \) = axial deformation of pile head, in inches;
- \( \epsilon \) = strain at 50% maximum deviator stress in laboratory undrained strength test;
- \( \lambda \) = frictional capacity coefficient;
- \( \sigma_{eff} \) = mean effective vertical stress, in tons per square foot; and
- \( \sigma_{eff} \) = effective vertical stress, in tons per square foot.