Geotechnical planning of piled foundations for offshore platforms

F. E. TOOLAN & D. A. FOX

Mr Toolan

Since the Forties Field FB Platform, my company has been involved in the design and installation of the following jackets in the British sector of the North Sea; Forties FD; Brent A; Claymore; Heather and Ninian. The Forties Field was underlain by normally consolidated clays, sands and at depth by overconsolidated clay. The other four fields are further north and are underlain by very hard clays and dense sands. No major pile installation problems were encountered at these northern sites. It was reassuring to find that the design methods used for planning pile installations work just as well for pre-consolidated sands and clays as they do for normally consolidated soils.

Professor T. J. Poskitt, Queen Mary College, London

My comments are on the use of the wave equation in driveability analyses and on the problem of plugging.

71. By any standards the piles used offshore are extremely large. Load testing is clearly out of the question and it is therefore natural for engineers to seek ways in which they can relate the soil resistance at the time of driving (SRD) to the static capacity. This is proving to be an extremely difficult job because the SRD is essentially a dynamic phenomenon which has to be studied by the wave equation. In addition pore pressure changes and thixotropy in the remoulded soil will produce time dependent changes which are difficult to predict analytically. These are areas of piling behaviour which are not fully understood and research is necessary in order to gain a better understanding of the mechanisms involved.

72. The Authors give a comprehensive description of the use of the wave equation in drivability studies. In principle the wave equation is a powerful tool which will enable the designer to take into account the many different factors known to affect piles. However, in my experience considerable skill is needed if it is to give meaningful answers. Parameters such as quake, damping factors etc., are as important as shear strength. This is not always sufficiently well appreciated by engineers and must have led to many disillusioned users of the wave equation. It is my opinion that where the wave equation is being used intensively as the Authors suggest in § 7(e)–(g), the determination of the constants for use in the wave equation should have the same priority as is attached to other soil tests.

73. In Fig. 11 the Authors give basic data on SRD/blow count obtained from wave equation analyses. When this data is plotted in the form $1/\text{SRD}$ against $1/\text{blow count}$

Fig. 17

Table 1

k	a , kip ft	$\bar{c}$, ft	slope, $\text{kip}^{-1}\text{ft}^{-1}$
65%	321	0.047	0.0048
75%	310	0.050	0.0043

Fig. 18

Fig. 19. Construction sequence of foundation for platform in Gulf of Mexico

aspects of pile design, not least that of static axial capacity. The Authors have summarized current practice, which leaves a lot to be desired.

82. The Authors mentioned three methods of calculating static axial capacity: the API method, the λ method and the CPT method. The first two methods are empirical, based entirely on tests performed on piles normally of smaller scale than those used offshore, on soils of great variability and in piles of different types. The results are all summed up in one line. Both methods depend upon undrained shear strength values and the empirical correlations with shaft friction obtained from the tests are derived from laboratory tests of samples of superior quality to those obtained offshore. But assuming good in situ testing can be carried out giving different c_u values from those currently obtained from offshore site investigations, what is then the position?

83. The CPT method is again empirical and again relies to a certain extent on the ability to extrapolate in terms of pile size. There is often a danger with empirical methods that they will be applied in conditions to which they are not suited. I would ask whether the CPT method they describe holds for both unplugged and plugged piles. I believe the Begemann method was derived mainly for solid piles and the region over which the averages of the q_c values are taken is given in terms of pile diameters. Surely if a pile is unplugged, then the bearing capacity is dependent on a region related more to the wall thickness than to the pile diameter.

84. The effective stress method was not used in the Paper. It often seems to be dismissed by practising engineers as something of an academic dream in spite of its simplicity. It does, however, allow a rational approach to pile design and allows those factors mentioned in § 44, such as the increase in lateral load of the pile or the effect of driving adjacent piles or the installation of nearby conductors, to be put on a rational basis as it relies purely on the evaluation of the in situ stress conditions in the soil.

Ir J. W. Jansz, Hollandsche Beton Groep, Netherlands

A recent article in *Offshore Engineer*¹⁹ has given some information on underwater piledriving in 320 m deep water in the Gulf of Mexico.

86. The piles were 190 m long and had to be driven more than 150 m into the sea bed. In all 24 such piles were driven successfully, and the actual driving of each pile took no more than 55 min on average.

87. The piles were for a three-part platform, or more correctly for the foundation section of a two part platform. The sequence of construction is shown in Fig. 19. The

Fig. 20. Driving heavy piles in deep water

base section is first lowered to the sea bed using derrick barges, acoustic transponders and television (Fig. 19(a)); piles are then lowered (Fig. 19(b)) and located in pile guides, again using television and transponders. The piles were then stabbed, and the hammer was run down guide-wires and the piles were driven (Figs 19(c) and 20).

88. In the next stage the middle section of the platform was lowered onto the base, and the connecting rods in the legs were then cemented into place. Finally the top section was lowered and pre-installed pins in the legs were cemented in.

DISCUSSION

Mr G. Lewis, CJB Earl and Wright

As designers of a number of North Sea pile platforms my firm appreciate the need to be able to predict with confidence the depth to which piles may be driven. We were therefore interested in the methods for predicting soil resistance at time of driving, particularly on the assumption that the skin friction acting on the pile is the remoulded shear strength, and furthermore that this remoulded shear strength can be determined from the liquidity index.

90. If the predicted SRD on Fig. 12 is compared with the backfigured SRD on Fig. 15, quite good agreement is shown. In order to have more confidence in the use of this method in the more northerly parts of the North Sea where the clays are generally much stiffer, my company decided to carry out a similar comparison on one of the structures which we have designed.

91. The results (Fig. 21) are for all eight piles of one leg of the Chevron Ninian Southern platform. The nearest borehole was 40 m away from the centre of the leg and the predicted soil resistance at time of driving was estimated from remoulded shear strength derived from liquidity indices using, as suggested, the relationship as presented by Skempton and Northey.¹⁴ The shaded area is the range of calculated actual soil resistances at time of driving backfigured from the driving logs by carrying out wave equation analyses and assuming a hammer efficiency of 75%. The results presented are for the clay part of the boring only and for only the first hammer used. They show for this particular site that the methods suggested by the Authors agree with field experience quite well, with a practical amount of built-in conservatism, as in general the predicted SRD is slightly higher than that found in experience.

92. The other field comparison I should like to present very briefly is the predicted self-weight pile penetrations. The Authors propose that these can be estimated by equating the weight of the pile and followers with the calculated SRD when the piles are lowered from the mudline.

93. Table 2 shows for the Ninian Southern platform the actual self-weight penetrations when the pile and follower string and hammer had eventually been placed prior to pile driving. Columns 1 and 2 are the observed range and the mean. The final columns are the values subsequently calculated but without knowledge of what the actual results were from the borehole logs. There was at this site one borehole within 50 m of each leg and one CPT test within 50 m of each leg. They compared pretty well. We are grateful to Chevron Petroleum (UK) Ltd for permission to reproduce the field observations.

Dr R. H. G. Parry, University Engineering Dept, Cambridge

I would like to consider the very high undrained shear strengths measured on samples from some of the glacial-marine clays in the North Sea. It is extremely important to decide whether this is due to some measure of cementing or to some other cause. If it is due to cementing, a problem arises. In soft cemented calcareous sandstones, for example, open ended steel tube piles can be driven some considerable distance, but they develop very little shaft resistance because the sandstone does not close back around the pile. If on the other hand, the shear strength derives from some other source, then the soil probably will close back around the pile. It is my view that the very hard clays in the North Sea, with shear strengths sometimes four or five times that of very stiff London clay, do not derive their strength from cementing, but from high effective stresses in the ground; in particular high lateral effective stresses at shallow levels, and a high effective stress angle of shearing resistance.

95. Consider an undisturbed sample of soil taken from 10 m below seabed, at which level undrained shear strengths of 600 kN/m² and higher have been measured in triaxial tests. Corresponding measured values of unit weight and effective stress angle of shearing resistance ϕ' are 22 kN/m³ and 35° respectively. This high value of ϕ' arises from the fact that clay is a very compact material, well graded, with a significant sand content.

Fig. 21. SRD vs depth for 8 piles at Ninian Southern platform

Table 2. Self weight pile penetrations at Ninian Southern platform

Leg	Self weight penetrations, m		
	Observed		Calculated from equations (13) and (4)
	Range	Mean of all 8 piles	
B2	7.0-12.0	9.3	12.3
B4	7.8-10.0	8.7	7.8
F2	8.6-10.5	9.8	11.6
F4	8.8-13.8	10.0	10.0

- Notes:
1. Piles 1825 mm o.d. and generally 63 mm wall thickness with shoe 85 mm thick and 2250 mm long at tip.
 2. Effect of shoe assumed to reduce inside skin friction above shoe by 30%.
 3. Penetration measured after installation one 80 m long pile, two 70 m long followers and Menck 4600 hammer.
 4. Data from nearest boring and CPT test used in calculation at each leg.

Fig. 22. Stress paths for undrained triaxial compression tests

96. Laboratory consolidation tests have indicated over-consolidation ratios of about 12 at this depth, for which k_0 , the ratio of lateral effective stress to vertical effective stress, can be assumed to be at least 3.0 (and this value will be assumed here). It is quite likely that values of k_0 of 5.0 or higher may exist in these heavily overconsolidated deposits of the North Sea. With the above assumptions it is now possible to explain the high undrained shear strength measured in the triaxial compression test on an undisturbed specimen in the laboratory.

97. In the laboratory specimen the high effective stresses in the ground are replaced by a negative pore pressure, and it is assumed here that the soil voids are sufficiently fine to be able to retain high negative pore pressures. In fact a well-graded compact clayey soil should be able to retain a negative pore pressure of several atmospheres.

98. If it is assumed that the soil behaves as an isotropic elastic medium with respect to stress changes during sampling, the negative pore pressure Δu is given by the expression

$$\Delta u = -\frac{1}{3}(\sigma'_v + 2\sigma'_H) \quad (16)$$

where σ'_v , σ'_H are respectively the vertical and horizontal direct effective stresses in the ground. Any anisotropic behaviour of the soil or inelastic behaviour would be likely to lead to higher negative values of pore pressure than given by equation (16).

99. For a soil specimen from 10 m below seabed level, taking $\gamma = 22 \text{ kN/m}^3$, $\phi' = 35^\circ$ and $k_0 = 3.0$, equation (16) gives $\Delta u = -280 \text{ kN/m}^2$. Consequently, when the sample is placed in the triaxial cell it is experiencing an isotropic effective stress of 280 kN/m^2 , represented by point A' on the stress path diagram in Fig. 22.

100. If a cell pressure of σ_c is applied, the corresponding total stress point is A and the total stress path during axial compression is ABC. The effective stress path is separated from the total stress path by the pore pressure value at any time and two typical effective stress paths are shown, A'B' assuming the pore pressure parameter at failure A_f is zero and A'C' assuming A_f is 0.15, where (after Skempton²⁰) for a saturated soil:

$$\Delta u_f = \Delta \sigma_3 + A_f(\Delta \sigma_1 - \Delta \sigma_3) \quad (17)$$

A value of $A_f = 0.15$ is realistic for a heavily overconsolidated clay and, indeed, higher values are possible.

Fig. 23. Influence of pile length on α value

101. The undrained shear strength c_u is given by the value of $(\sigma'_1 - \sigma'_3)/2$, where the effective stress path meets the failure envelope OY. The values of c_u in this case are 380 kN/m² and 600 kN/m² for $A_r=0$ and -0.15 respectively. Thus, the high shear strengths can be readily explained by the high effective stresses in the ground. Some proportion at least of these effective stresses will act in developing pile shaft resistance.

102. It is likely that some decay of high negative pore pressures in the soil will occur after sampling, rapidly at first then more gradually, and ideally undrained strength tests should be performed on site, immediately after sampling. The recently developed Tri-con machine makes this possible.

103. I would like to make a further point about shaft resistance. Although I believe effective stress methods of determining shaft resistance will eventually supplant total stress methods based on some proportion α of the undrained shear strength, α is likely to continue in use for some time. However, the relevant value of α should be assessed on the basis of effective stress conditions around the pile as far as these can be determined. Unfortunately, α values have tended to be based on undrained shear strengths whereas, in fact, α is more fundamentally related to the ratio of lateral to vertical direct effective stresses in the soil²¹ and consequently to overconsolidation ratio.²² Values of α adopted for design are based on field loading tests on land based piles with an average length of perhaps 10 m. Offshore piles are longer, often 50 m and more, and the consequence of this is that the average values of α along the pile length, for the same average shear strength, will be higher, as shown in Fig. 23. Thus, the use of α values from land based observation may be unacceptably conservative when applied to offshore piles.

Mr P. George, Lloyd's Register of Shipping

I would like to take up the point made by Dr Parry concerning negative pore pressures in the very strong North Sea clays, and give some results of tests carried out on some of these clays.

105. Figure 24 shows results from tests carried out on clays with undrained shear strength in the region of 20–40 t/m². These tests were run as consolidated undrained tests. Since high suctions were observed in the samples, dry filter stones were placed on each end of the sample. A confining pressure equivalent to the overburden pressure was then applied to the cell before water was introduced into the filter stones. Water was then allowed to come into contact with the clay sample through the pore pressure connection, and any sign of swelling was controlled by increasing the cell pressure to keep the pore pressure equal to zero. The confining pressure (σ'_c) which was necessary

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Fig. 24

in each case was recorded. Fig. 24 shows σ'_c divided by effective overburden pressure (p'_o) versus depth.

106. This is evidence from laboratory testing that these negative pore pressures do exist. The retention of these negative pore pressures by clay samples is a very important phenomenon, since it should be possible to estimate values of k_0 indirectly from measurements of negative pore water pressure, which can be used in effective stress methods of pile design.

Ir E. P. Heerema, Heerema Engineering Service, The Netherlands

Equation (10) states that for more heavily overconsolidated clays the proportion between cone resistance and shear strength increases.

108. At a recent conference Professor Dr E. de Beer, who has done much testing in overconsolidated clay (mainly Boom clay) described a tendency in the opposite direction: the proportion decreases from 20 for very soft clay via 15 for soft clay to 9–10 for stiff overconsolidated tertiary clay. His experience is in a different type of clay from that of

the Authors, but I should like the Authors to explain what may be the reason for the different findings.

109. The hardest clay that has been found so far at shallow depths in the North Sea was at Heather Field; it is very highly overconsolidated. A correlation between the measured cone resistances and the results of triaxial tests would lead to a lower factor than 15–30 c_u . If the Authors proposition is to be accepted, it may lead to interpreting cone penetration test measurements to shear strength values which are on the low side, demanding deeper driven piles, while drivability might be underestimated.

Mr J. N. McFeeters, Harris & Partners

I would like to make a few remarks about Fig. 16. There is a remarkable correspondence between the static loading forecast of the left-hand curve, the CPT forecast of the right-hand curve and the estimates of capacity of the piles which fall between those curves. The samples from which the left-hand curve is formed have been lifted from dispersed holes, none of them nearer than 100 ft from the edge of the platform, and vertically at intervals as seemed appropriate and feasible to extract under the circumstances, but I do not believe they were continuous samples. The same is true of the probings. These also were taken at boreholes some little distance away.

111. Laboratory tests were performed and a theoretical abstraction, f_s , is decided upon for the average grip of the soil on the surface of the pile, resisting the movement of the pile under load in a particular horizon. Various values of this are summed, together with the theoretical computation of the end load also based on strata from the dispersed boreholes. Next taking the CPT curve on the right, a relationship is asserted between the resistance offered to a 35.6 mm dia. cone pushed into the soil at the same dispersed locations and the expectation of resistance to movement of the pile when installed. This yields a higher estimate.

112. Next the same soils data are used to form the basis of the wave equation for pile driving together with other inputs, such as hammer efficiency, anvil effect, pile make-up and damping effect, to set up the highly elaborate basis for the computer solutions of the wave equations.

113. The difference between the approaches, static and dynamic, needs to be emphasized, because it appears to be remarkable and encouraging that the calculated pile capacities fall as they do between these two capacities predicted by static methods. The calculations by the API theoretical approach give a figure for ultimate capacity of some 6000 kips. The cone penetration gives 7000 to 10 000 kips. The dynamic pile equations yield 7000 to 9000 kips. However one looks at it, the correspondence is astonishing. Nevertheless the Authors are careful not to suggest that the piles should be designed by the wave equation only; in fact the design remains based firmly on static methods.

114. However, it is necessary to turn again to the fact that the same soils inputs are used for providing the static and dynamic outputs, although different procedures are used thereafter. Thus, both calculations of capacity depend on the accuracy and relevance of the soil tests and, of course, this is fundamental. In these circumstances the information cannot be finally used to confirm the conclusions arrived at, as the same information is played in different ways through the three approaches.

115. The missing element is a factual loading on the piles. When one looks for confirmation of static methods one finds that most previous pile tests are irrelevant, either because they have not been carried beyond a test at working loads, and ultimate loads remain unknown, or because the piles hitherto tested have little relevance to the 54 in. open piles in North Sea soils. The nearest confirmation lies in the paper by Fox *et al.*²³ where two load tested 30 in. piles of the West Sole Field are compared and criteria of capacity are calculated. I am not certain that these criteria are identical with those used in the left-hand curve, and would be interested if the Authors would say how closely this positive information applies. Further information on other pile tests which tend to confirm the accuracy of the methods they have used would also be useful.

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Fig. 25. Undrained shear strengths determined from 98 mm dia. triaxial and 865 mm plate loading tests on glacial clays at Redcar

Mr B. Marriott, Vickers Offshore Ltd

Figure 16 denotes the assigned capacities of all the piles. The Authors state that these capacities were based on the SRD on redriving obtained either directly from a redrive test or estimated from the set up measured between drives.

117. Pile D6 is denoted in Fig. 16 by one of the plus signs. I do not follow from the Paper the mechanics of the correlation between pile D6 in Fig. 15 and a plus sign in Fig. 16. It would be interesting if the Authors could elucidate.

118. My second point is that the Paper makes no reference to the uplift requirements or capacities of the piles. A word on these might be useful.

Dr J. B. Burland, Building Research Establishment

I would like to make a few remarks about the shear strength of stiff clays and in particular the assessment of the point resistance of a pile. Firstly, it must be emphasized that there is no unique value of c_u for a given soil. The value of c_u depends on the volume of ground being sheared, the direction of loading and the degree of disturbance undergone by the ground. Thus the value of c_u to be used to estimate the shaft friction at the time of driving may be equal to the remoulded shear strength c_r . After driving the static shaft friction is often estimated as an empirical factor α of c_u . The value of α has been developed using the mean value of c_u from tests on $1\frac{1}{2}$ in. dia. specimens. If the value of c_u is determined in some other way it may not be correct to use the usual values of α . This reasoning applies to any other empirical or semi-empirical method.

Fig. 26. Cone resistance measured at penetration speeds of 1200 and 45 mm/min compared with ultimate bearing pressures measured on 865 mm dia. plate at Redcar

120. Turning now to the evaluation of point resistance, in recent years the Building Research Station has been studying the bearing capacity of stiff clays and tills using large diameter (865 mm) plate tests. The results of these tests provide a valuable means of assessing the accuracy of current methods of estimating point resistance.

121. Marsland²⁴ has carried out a detailed investigation of two glacial clays at Redcar and concluded that undrained triaxial tests on 98 mm dia., high quality samples overestimate the bearing capacity by as much as 2.0 in the upper boulder clay and by about 1.5 in the laminated clay (see Fig. 25). On this same site cone penetration tests (CPT) were carried out and the results are shown in Fig. 26. It can be seen that the bearing pressures q_c measured by the cone were about 2.5 and 2.25 times those measured with the large plate tests in the upper till and laminated clay respectively. Hence, in this case, the use of CPT as a model test, as suggested in § 39, would have seriously overestimated the point resistance.

122. Similar tests have recently been carried out at a site in Cowden in till extending to considerable depth. The work is at an early stage, but it is already clear that in this case the bearing pressures given by the CPT and large diameter plates are in reasonably good agreement.

123. It is evident that the evaluation of the strength and bearing capacity of stiff glacial tills is far from straightforward. The reason for the large variations in the relationship between the CPT, laboratory tests and plate tests are far from clear, but are thought to be related to the composition, fabric, degree of overconsolidation and pore pressure response under loading.

Mr Toolan

Professor Poskitt has provided three reasons why plugging may not always be observed when static analysis predicts that it should: inertia of the soil, softening of the soil, and stress-strain rate properties of the soil. I agree with the last two reasons but I am not convinced as to the first. Professor Poskitt mathematically models the soil plug as a rigid body to show the effect of inertia. If this model was correct, open ended pipe piles would never plug. However, there are occasions on which the plugging of such piles has been reported. These instances may be explained if the soil plug is modelled in the same way as the pile, as a series of masses and springs, and stress wave propagation through the soil is considered.

125. With reference to **Dr St John's** comments regarding the use of API, λ and CPT design methods, and advocating the use of effective stress design methods, I can only agree that each of the pile design methods used in the Paper is to a greater or lesser extent empirical. However, it must be remembered that they have all been proved in the field to give satisfactory foundations. There has not been, to my knowledge, a pile failure attributed to a deficiency in any one of these methods.

126. The problem of using the results of improved in situ testing techniques in conjunction with the API and λ methods was raised. With any design method the quality of the soil data used as input should be similar to that used to derive the method. For these two methods many of the basic data were obtained from tests on land. Thus problems should only arise when offshore soils data are of a higher quality than those obtained during a routine site investigation on land. A certain amount of engineering judgement will then be required. However, if all new in situ testing devices were calibrated against routine tests on land the problem might be to some extent alleviated.

127. The practice of extrapolating the results of static load tests on small diameter piles on land to much larger more heavily loaded piles offshore was questioned. A test on a very large pile similar to those used for the northern North Sea jackets would be extremely useful. However, the practical problems and costs are enormous and to my knowledge no-one has made such a test. In the meantime the Authors believe that the extrapolations are valid. It must be realized that the only parameters which are subject to extrapolation are pile diameter and pile load. The much more important parameters of stress and pile length are not subject to extrapolation.

128. The CPT method can be used for unplugged pipe piles in exactly the way suggested by **Dr St John**. In fact my company do this as a matter of routine.

129. With regard to effective stress methods for predicting skin friction in clay, my company does use them and does not dismiss them. In the past few years there have been a number of papers published on effective stress methods and as many as possible of these have been examined. In general two common points emerged; they tend to contain as much empiricism as total stress methods and they provide higher compressive capacities and hence shallower penetrations than total stress methods. One cannot be sure that a particular design method is going to supply the correct pile capacity at a given site and a given time after installation. Therefore it is advisable to design by a number of different methods, total and effective stress, to obtain a range of acceptable penetrations. This is the procedure described in the Paper although I must admit that an effective stress method was not used. This was partly because it was the assessment of end bearing in sand not skin friction in clay which was the critical variable. At sites underlain by clay effective stress methods have recently been used to define minimum penetrations.

130. **Ir Jansz** has provided an interesting description of underwater pile driving in very deep water in the Gulf of Mexico. Following the success of this project it appears inevitable that underwater driving will be used in the North Sea in the near future.

131. It was reassuring that **Mr Lewis's** experiences of using two of the design methods described in the Paper provided results which agreed reasonably well with observations.

132. At some sites in the northern North Sea very high undrained shear strengths have been measured on clay samples. **Dr Parry** provided a convincing explanation of

why this should be. **Mr George** presented test data which substantiate Dr Parry's theory and suggested a new laboratory procedure by which k_0 might be estimated indirectly. The contributions of both Dr Parry and Mr George are relevant to effective stress pile design methods.

133. **Mr McFeeters** discussed the interrelationship of static and dynamic methods of pile capacity assessment. The values of quake and damping that were selected for use in the wave equation for the Forties FB studies were developed by Professor Hirsch of Texas A and M University. They were determined on the basis of comparisons between driving tests and static load tests.

134. A question concerning the correlations between cone resistance (q_c) and undrained shear strength was raised by **Ir Heerema**. The relationships provided in equations (8) and (10) of the Paper were developed from my company's own experience and published records. For example for some overconsolidated clays on land the work of Marsland²⁴ indicates cone factors of approximately 25.

135. It is known that in certain clays these generalized relationships for cone factors are not valid. Professor Ir de Beer has shown that they are certainly not applicable to very soft or very hard strata of Boom clay. For this reason separate correlations must be developed for each clay.

136. It is standard practice in my company to plot for each site the cone resistance against the undrained shear strength measured in soil samples. With particular reference to overconsolidated North Sea clays it is consistently found that the cone factor is 15-20. This relationship is used rather than the more general one provided by equation (10). There does appear to be a slight tendency for the harder clays to plot towards 15 and the less stiff clays to plot closer to 20. It is not certain whether this is an intrinsic property of the soil or whether it is because the less stiff clays are more likely to suffer from sample disturbance. If the latter were the case then the shear strengths will be underestimated for softer clays. This would provide an apparent decrease in cone factor with increasing shear strength.

137. **Mr Marriott** requested some more information on the method the Authors used to assign pile capacities, and in particular how the capacity of pile D6/8 was obtained from the driving behaviour. Of the 32 piles driven for the FB jacket, 21 suffered a driving interruption when their tips were between 160 ft and 190 ft below mudline. It was found that the SRDs on redriving after these interruptions fell within a narrow envelope. For those piles for which there was no set-up data near final penetration the lower bound of this envelope was used to assign capacities.

138. For pile D6/8 there is no set-up data below 150 ft (Fig. 15). Above this level there is a possibility that the pile tip could be in sand and this can make set-up evaluations unreliable. Driving into a sand layer frequently causes a sudden increase in resistance and it is impossible to separate this from true set-up. At 180 ft all the boreholes showed that the pile tip would be in clay and this was confirmed by the driving behaviour. Data from driving other piles, i.e. the lower bound of the set-up envelope, indicated that if pile D6/8 were stopped at 180 ft its SRD on redriving would be at least 2800 kips. The backfigured SRD during continuous driving at 180 ft is 1800 kips. Thus a set-up due to shaft friction above 180 ft of at least 1000 kips can be expected. This figure was added to the SRD at final penetration, 6800 kips, to produce an assigned capacity of 7800 kips (Fig. 16).

139. Uplift resistance was not the governing design criterion for pile penetration. This is frequently the case for the larger North Sea jackets which have a considerable self-weight.

140. **Dr Burland** presented a number of interesting comparisons between the results of CPTs and large diameter plate tests. First I must emphasize that the Authors do not use the CPT to predict static end bearing resistances in clay. In § 39 we were discussing point resistance during driving. The CPT method as described in § 24 and on Fig. 7 is used only for piles end bearing in sand.

141. On Fig. 26 it is clear that the end bearing of the plate is more than twice the cone

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Fig. 27. West Sole: load test piles temporary compressions; theoretical and measured values

resistance. This is reasonably consistent with the end bearing of the plate being $9c_u$ and the cone resistance being $15c_u$ to $20c_u$, as mentioned in my reply to Ir Heerema. When comparing the results of plate tests to CPTs one must bear in mind the very much larger zone of influence of the plate to that of the cone. Also the stress regime around the cone is not identical to that below the plate.

Mr Fox

Professor Poskitt considers the pile driveability analyses derived from wave equation studies and their correlation with the Hiley formula.

143. I fully agree with his appreciation (§ 71) of the necessity for engineers to use this powerful tool in order to estimate the capacity of piles driven under offshore conditions, and with his comments (§ 72) on the difficulties involved. It appears to me that any disappointing results can usually be attributed to unrealistic modelling of the frictional and base or tip parameters as input data for the program, and also on whether redriving tests have been carried out to determine any time-dependent influences such as set-up or even final relaxation.

144. Referring to § 73, Fig. 17 and Table 1, these show a useful basis for adjusting the Hiley formula for very long piles by evaluation of an 'equivalent length'. This of course is the purpose of measuring the actual temporary elastic compressions when driving piles. BP encountered difficulties and dangers in trying to obtain these measurements from the bare steel jackets first installed in the West Sole Field in 1966. No wave equation program had been prepared at that time but in discussions with the late Mr Cornfield, as well as from ref. (18), it was understood that 'effective lengths' of very long piles would probably not exceed about 75–100 ft. My own studies of temporary elastic compressions from both field measurements and computer calculations generally confirm this order for 'equivalent lengths'.

145. I do not however agree with Professor Poskitt's analysis that equation (14) and Table 1 always result in the simple straight line of uniform slope shown on Fig. 17. I

Fig. 28. Blowcount/resistance correlations

find several cases where plotting as Fig. 17 results either in slight overall curvature or a line dividing into two or more distinct slopes. These variations from one single slope probably derive from variations between end bearing and shaft friction. The temporary elastic compression evaluated according to Hiley only takes into account the estimated

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total driving resistance, whereas computer calculations of temporary elastic compression reveal that changing end bearing and shaft friction proportions do influence this compression of the pile and therefore the energy absorbed.

146. According to Table 1, the total temporary elastic compression (c) is determined as 0.56 and 0.60 in. according to hammer efficiency. Hiley's value of c includes the ground quake, which for the case shown in Fig. 11 for the pile tip in sand was assumed as 0.10 in. The BP computer calculations in the early design stages, assuming then all-welded piles, indicated temporary elastic compressions of the pile and anvil-cum-cushion block of about $\frac{3}{4}$ – $1\frac{1}{4}$ in. in the higher ranges of SRD under the Menck 7000. But I am not aware of any field measurements indicating the values and 'effective length' corrections achieved.

147. However, the West Sole piles²³ show the possibility of reasonable correlation between temporary elastic compressions calculated by computer (pile and anvil-cushion only) and field measurements of total compression probably including quake (Fig. 27). The 30 in. piles were 240 ft long so that 'effective length' was about 100 ft.

148. Referring again to possible deviations of a Fig. 17 plot from the straight line and uniform slope, Fig. 28(a) shows Fig. 11 replotted to a different linear scale and (b) a 'handed' replot of Fig. 17 extrapolated to lower blowcounts. Fig. 28(c) shows the same data replotted semilog, all three figures limited to the 75% efficiency example. In the region of high resistances and small sets, a plot similar to Fig. 17 can lead to imprecise drawing unless the scales are enlarged, and many cases have been found where the blowcount/SRD relationship tends to deviate from a straight slope and curves asymptotic with the set or blowcount axis; this is difficult to show on the scale of Fig. 28(b).

149. At low resistances and large sets (low blowcounts) the normal linear plot (Fig. 11) is difficult to draw with precision and the further semilog plot can provide a useful correlation with Professor Poskitt's analysis. The point here is the influence of the pile dead weight on driving resistances since the basic Hiley formula states that static capacity = driving resistance + dead weight of pile. Different designers have shown different drawings of this relationship on their blowcount/SRD curves at low resistances, but from Fig. 28(b) and (c) it appears logical to combine the extrapolation from the Poskitt straight line with the semilog construction as shown. The resultant curves are found to intercept the value for weight of pile alone and in particular, the added weight when the followers and gravity connectors have been lowered to the pile head and the hammer and anvil placed on top, when further penetration often occurs before the hammer commences the first drive, at a range of blowcounts which can usually be equated with the observed range of blowcounts at the start of driving. Although it is more usual practice to start driving with the lightest of the hammers available in an offshore spread, a useful correlation has been found in this manner for the cases where a heavy hammer was put on for the first drive, both at Forties and elsewhere, and the low initial blowcounts observed.

150. I consider that experience of using the wave equation can be applied to the modification of the Hiley and similar energy formulae for ordinary 'manual' calculations, and that Professor Poskitt offers some very useful correlative studies.

151. In reply to Mr McFeeters, on Fig. 16 one thing that is not brought out is that with the experience already obtained in Forties, piles were being driven at FB much more quickly and therefore the soils would have been softened and would not have been allowed to set up as much as in the earlier piling operations. Only one of the 32 piles had a special period of 20 h for observation of set-up before redriving. One other pile was redriven after 5–6 h of set-up. The remaining piles all had less than 4–5 h before their various drives. That means that the assigned capacities are on the conservative side because one cannot take too many chances in estimating how much set-up will develop in 3–6 months' time or even longer. The general pattern of Forties Field pile set-up is illustrated in Fox *et al.*²³

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