

The production of North Sea Gas

G. B. MARRIOTT & V. J. R. SUTTON

Mr E. W. Flaxman, Binnie & Partners

The two submarine pipelines described by the Authors in § 75 represent a total capital investment of about £14 000 000 and they appear to be the largest examples of this class of work yet undertaken in British waters.

95. Increasing use is now being made of submarine pipeline techniques in other fields and it would be useful if the Authors could give some more information about the design of the natural gas pipelines in relation to sea bed conditions. In § 78 trenching depths of from 4 to 10 ft, according to location, were mentioned and it would be interesting to know what minimum depths of cover were specified. From § 81 it appears that the pipelines may remain exposed for several months before becoming buried by natural sand movements. Were the pipelines in operation during this period and was the risk of damage by ships' anchors considered to be negligible?

96. Finally, it appears from § 77 that the thickness of the weight coat has been increased where the pipelines are within territorial waters. The limit of territorial waters is a notional line, not connected directly with either depth or sea conditions, and it would be helpful if the Authors would explain the reasons for the adoption of a more expensive form of construction within this line.

Mr A. M. Muir Wood, Sir William Halcrow and Partners

An unsuspecting reader of §§ 45–49 might suppose that a fully sufficient theory was available for the calculation of wave force against a drilling platform. I suggest to the Authors that the following factors are among those that require considerably more study before there can be any complacency among engineers responsible for designing such structures.

98. While the higher order wave theories (or Cnoidal wave theory in shallow water) may appear better to represent the particle motions of steep regular symmetrical waves, they take no account of important modifying effects such as eddy viscosity. Moreover, in many marine applications, the irregular, short-crested and asymmetric wave of the open sea is found to have significantly different effects on structures than the long regular wave of the wave theories.

99. Considerable doubt is attached to the evaluation of the inertia and drag coefficients and it is highly improbable, from fundamental considerations, that these are each constant through a wave cycle as frequently assumed for marine structures.

100. Knowledge of the motions of the incipiently breaking wave relies more on experiment than on theory; this is often the most critical type of wave for an offshore platform but is such a rare event at the situation of a platform that few full-scale records are available.

101. Where, on account of steepness of the wave or the slope of the structural member, the motion of the water is inclined obliquely to the axis of the member, the inertial and drag coefficients will be varied in consequence.

102. The vortex street that forms in the wake of a cylinder in a steady current causes a cyclical variation in drag and in lift (i.e. the lateral force perpendicular to the direction of the current). In strong currents at sea considerable oscillations of long cylinders may result and this may be a factor in the design of a riser casing from a borehole (see § 27). Similar phenomena may be expected to be set up by wave motion but resonance is less likely to result.

DISCUSSION

103. The factors enumerated above relate to an individual member and the interaction between members of a structure will add further complications to any attempt at direct analysis, particularly for one member lying in the wake of another.

104. This is a most important field for increasing the available data and developing methods of analysis so that information acquired for one structure in one locality can be interpreted and applied with confidence to a given structure in a given situation. Until then, engineers will have to include a high allowance for 'ignorance factor' in their design, whatever order of wave approximation they may adopt. Model tests can only throw limited light on the problem, and I hope every opportunity is being taken to improve wave prediction and the assessment of hydrodynamic forces for the offshore platforms in the North Sea. There is no word in the Paper on instrumentation for such a purpose. This unprecedented opportunity must surely be exploited to the full to increase our knowledge of these phenomena so that, in a subsequent paper, the Authors or others may write a more convincing successor to §§ 45-49.

Mr F. G. West, Bataafse Internationale Petroleum Maatschappij NV, The Hague
On behalf of BIPM I would like to say that we found the Paper most interesting and as we ourselves apply many of the methods described in the Paper in our day-to-day work we would like to take this opportunity to pose some questions on certain design criteria.

106. In the design of casing strings (§§ 9-13) for exploitation wells, do the Authors make any allowance for subsidence which could arise as a consequence of a blowout? How are temperature differences between landing and operating conditions taken into account in the design of casings? Are possible wash-outs in the surrounding earth strata considered when evaluating the lateral support given to casings by the sides of pre-drilled deviated holes?

107. The lateral permissible off-hole displacement of 3% of the riser length quoted (§ 26) as being necessary to prevent overstressing of the conductor is very conservative. This factor is obviously a function of the many variables listed in § 27 but with the more modern pneumohydraulic tensioners capable of exerting a combined pull of 250 kips, a lateral displacement of 10% may be acceptable.

108. With respect to obtaining soil data for pile calculations (§ 36), are any attempts made to measure in situ shear strengths over the likely pile depth?

109. Why is the 100 year storm wave used as a design criterion (§ 45) in preference to, say, the 200 year storm wave?

110. In calculating wave forces on a structure (§ 48) what values of drag coefficient C_D and inertia coefficient C_m do the Authors use in Morison's equation?

111. What allowance is made in the design of an offshore structure for the likely occurrence of scour round legs and piles?

112. Computers seem to be applied to many of the problems. Are there any hand calculations made without a computer check or vice versa?

Mr T. D. Ruxton, Binnie & Partners

In § 50 Messrs Marriott and Sutton give the basic criteria for the 100 year storm used to design a fixed platform at a typical location in the North Sea. It would be interesting to know where the typical location is and what methods were used to establish the various criteria.

114. The maximum wavelength at the location is quoted as 800 ft. This corresponds to a wave period of about 15.2 s if the depth is taken as 104 ft (low tide) or 14.5 s at 123.5 ft (high tide). How is the quoted period of 13.5 s related to the maximum period? Is the wave height of 56 ft a 'significant' height adopted to represent a spectrum or is it indeed the greatest height which can be expected to occur at the location during the 100 year storm?

115. The sets of co-disturbance lines drawn by Rossiter¹⁵ show that a surge of about 7.5 ft occurred in the southern area of the North Sea in the storm of 31 January/

1 February, 1953. Is it reasonable to assume that the Authors consider the 1953 storm to have a return period of about 100 years?

Mr W. Kohring, Consulting Engineer

I would like to take this opportunity of posing a question on the subject of the driving of pile legs through the sleeve legs of a prefabricated braced template. It occurs to me that although piles can be stabbed to a great degree of precision, they do tend to wander off under driving to depth.

117. I wonder if thought has been given to the possible consequences of side thrust from the inner pile against the sleeve legs as it is possible that a high degree of stress could be built into the assembly as a result of the driving piles tending to drive off station.

Mr G. M. Cornfield, The British Steel Piling Co. Ltd

The search for and extraction of North Sea gas has given rise to some of the most difficult as well as interesting problems in piling. The Authors have perhaps under-emphasized the magnitude and complexity of the problems which have been occasioned by the site conditions of waves, wind and water depth, and by the high individual pile loads.

119. The high pile loadings are indeed an example of one trend towards increase in the size of foundation loads generally. The Authors have, I believe, correctly chosen driven piles in the given conditions, though there are other solutions which might be applicable in more sheltered waters. As an example, reverse-circulation drilling was used off the Californian coast for installing 36 in. dia. piles which were drilled 55 ft into the sea bed in 90 ft of water.

120. As mentioned by the Authors, test loading of piles is both expensive and difficult in the North Sea. Other methods of estimating ultimate pile loads are, therefore, necessary. The use of soil data can lead to one estimate, but the problem here is to decide on the value of adhesion which occurs in stiff or hard clays—the adhesion values sometimes used are in my opinion too low and can result in over-driving of piles to an unnecessary penetration. Another approach is to use the driving information, that is, hammer size, energy and final set per blow. A pile driving formula is an example of the use of this data. A further example is the application of wave equation analysis using a computer. Would the Authors care to indicate whether useful results were obtained from this last approach?

121. Another possible expedient for the estimation of ultimate resistance is to examine broadly the results of test loading to failure many piles at different sites and then to see whether any general correlation is possible. This was done some time ago when 65 cases of driven steel bearing piles were examined where test loading was definitely carried out to failure. The results are summarized in Fig. 5. Each symbol represents one pile test and for each test the quantity R_u/wH has been plotted against final set.

122. It must be emphasized that the information plotted is factual and no theory is involved. The particular method of plotting the results was arrived at after many attempts to discover some pattern or correlation, lengths of piles, weights of piles and other factors were considered but virtually no meaningful correlation resulted except in the case of the plotting method shown. Even here there is wide scatter but I suggest that the various points are broadly contained within the two lines drawn on the diagram.

123. From the investigation carried out and from the diagram it can reasonably be concluded that any pile formula is unlikely to be correct by more than $\pm \frac{1}{3}$. However, the data shown are nevertheless useful in setting some limits. Thus if wH and the set are known, approximate upper and lower limits are given for R_u . Conversely, if R_u is known and a final set has been selected, then upper and lower limits for hammer energy and size are obtained.

DISCUSSION

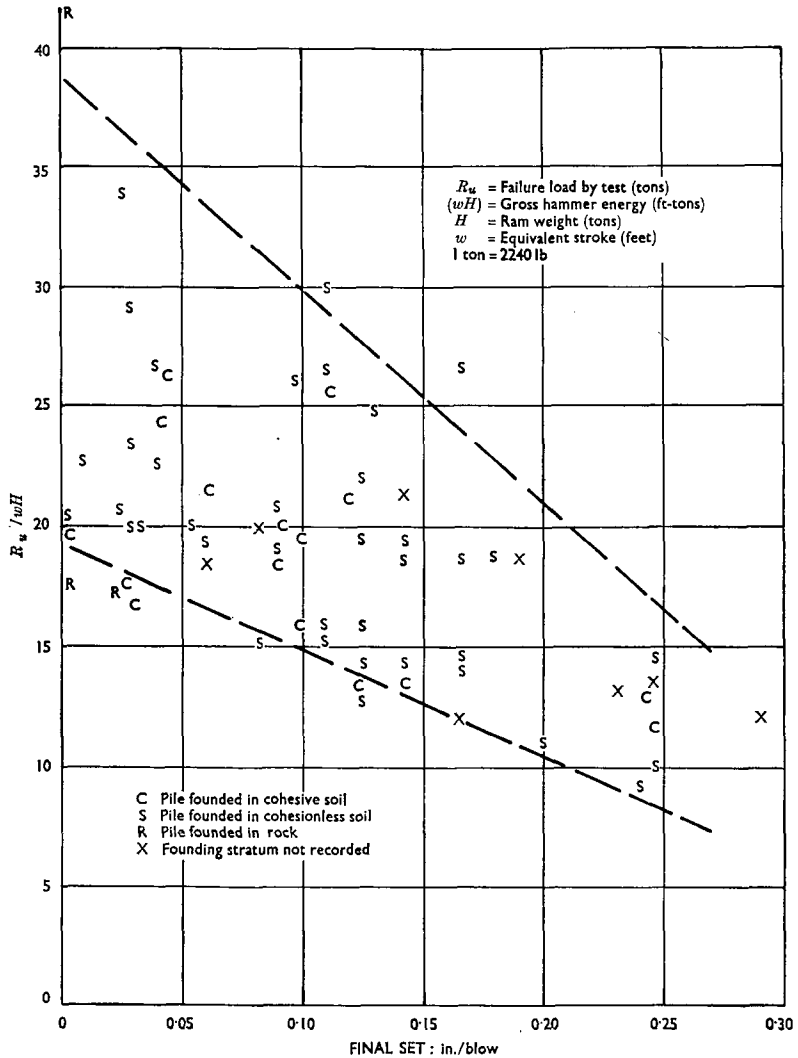


Fig. 5. Steel bearing piles : test loads to failure

124. Thus if a final set of 0.10 in./blow (10 blows/in.) is selected, the limits of R_u/wH are 15 and 30. If a typical drop for a single-acting hammer is taken, as, say, $3\frac{1}{2}$ ft, the limits for R_u/H become about 50 and 100. Thus a single-acting hammer with a 10 ton ram can be expected to achieve an ultimate driving resistance of between 500 and 1000 tons when used on a steel bearing pile driven to a set of about 10 blows/in.

125. Out of the 65 cases plotted, twelve piles had lengths between 100 and 190 ft, while there were 20 cases with pile lengths from 70 to 100 ft. A slight tendency can

be observed for the points representing piles founded in hard cohesive soils (c) to be located nearer the bottom of the groups of points but the scatter is almost as great as with piles founded in cohesionless soils (s).

126. A similar investigation was also carried out using the data of a large number of concrete piles test loaded to failure, but this yielded different results and in general there was much greater scatter.

127. I suggest therefore that hammer size and energy are the main criteria in determining the ultimate resistance of a pile driven to a given set, and that the effect of pile weight and length are relatively insignificant when the normal order of scatter of results is taken into account. Thus using the maximum design load of 1600 kips compression given in § 66 and assuming a factor of safety of two, the required ultimate bearing capacity is about 1400 tons. The minimum hammer energy required (wH) would then be 1400×2240 divided by between 15 and 30, that is, between 210 000 and 105 000 ft lb/blow.

128. There are cases when piles must be driven to a greater penetration than that at which sufficient ultimate resistance has been developed to carry a given vertical load, for example when a particularly large uplift resistance is required or when future scour may reduce the effective penetration. The hammer size should then be selected on the basis of the estimated higher ultimate resistance which would occur at the lower level. However, as very large hammers are expensive and there are economic limits to size, consideration should also be given to reducing the end resistance of the pile in such cases. One possibility is to use a composite pile formed of four steel H piles of 12 in. \times 12 in. nominal size welded together in cruciform formation. As 12 in. \times 12 in. H piles are available in weights up to 190 lb/ft, a composite pile of this type could have a weight of up to 760 lb/ft, and if in mild steel could be used for working loads of up to 1000 tons (or higher if high yield steel is used), subject to checking of the buckling strength of any upstanding part of the pile.

129. When a steel tube pile is being used, the use of a large internal drop hammer is also worth considering. A cylindrical drop hammer for a 36 in. dia. pile will weigh 30 tons if the hammer length is about 25 ft, and if operated at a 5 ft drop the energy per blow will be 330 000 ft lb. Relatively low speed of operation can be balanced by the use of a significantly heavier hammer, bearing in mind the low cost of the drop hammer itself.

130. It would be of great interest if the Authors could give more details of the piles which were used. How were the required penetrations decided, if the piles were not simply driven to a calculated set? Any test loading information now available would also be of great value. I would also be interested in the Authors' comments on how their findings compare with the attached diagram of pile test results.

Mr A. C. Scott, George Wimpey & Co. Ltd

The Authors are to be congratulated on their Paper, which could aptly be titled 'Engineering in deep water'. They have drawn attention to the methods existing for obtaining sea bed cores or soil samples necessary for the design of any engineering structure above or below the water covered areas of the world.

132. In my view the methods used in this field of activity are governed by three main factors. The first of these is the size of vessel available (or possibly the size of vessel the client can afford). The second is the water depth. The third is the plant available.

133. In general, the larger the barge or vessel applied to the work, the greater the likelihood of completing the work effectively and at lowest cost. However, as the craft becomes larger, the mooring arrangements become heavier. Furthermore, if weather conditions are so bad as to prevent work proceeding, a large vessel involves a charter cost greater than a smaller vessel. Similarly if weather conditions are perfect a smaller vessel would possibly complete a survey without difficulty. The choice of vessel therefore is a matter involving the exercise of considerable judgement.

DISCUSSION

134. It is noted that the Authors favour the use of a specially equipped floating vessel using the 'quick method' of boring. Unfortunately, this method requires a large drill and derrick to handle the drill pipe and this in turn necessitates a relatively large vessel. Furthermore, it is not always possible either to locate such craft at short notice or economically practicable to apply such craft to work of short duration where the mobilization and demobilization costs are relatively prohibitive.

135. The cost of purchasing and specially equipping a vessel for marine survey work could amount to £150 000, and few contractors would regard this a justifiable commercial outlay when consideration is given to factors such as the discontinuity of work in this field, the weather conditions prevailing in UK waters in winter and the insistence of many clients on hard price tendering for this class of work.

136. In regard to water depths, the experience of my Company has been that the conductor tube method (§ 41) is effective in depths up to 250 ft and this was successfully used for borings in mid-Channel in water depths exceeding 240 ft on a survey for the Channel Tunnel project.

137. In depths exceeding 250 ft, conductor tube methods or 'quick methods' (with the exception of the multi-million dollar 'Mohole' project) do not apply, due to current forces on conductor tubes, ship mooring problems and other factors. This focuses attention on the availability of remotely controlled plant located at sea bed level. At present a remotely controlled vibrator-driven sampler exists which takes 20 ft long disturbed samples of overburden materials from depths of up to 250 ft and my Company has used a rotary drill to recover 10 ft samples of shale from water depths of nearly 500 ft. All remotely controlled machines of which I am aware are 'one run' machines, i.e. they have the ability to penetrate the sea bed to a depth equal to the length of core barrel (or rod in the case of wire line methods) in line with the chuck of the drill and thus these machines have a definite limitation imposed on them. For this reason I fully endorse the Authors' reference to development work being necessary in this area of civil engineering.

Mr J. N. McFeeters, Consulting Engineer, formerly Manager, Civil Engineering Division, BP Trading Ltd

I should like to refer to three aspects of the work described in this very interesting Paper with special reference to costs:

- (a) *The design of semi-submersibles.* As stated in § 27 the *Sea Quest*, costing over £3½ million, had a down-time for repairs of 4 months or more in its first two years of life. The loss to the company during this period would be a very large sum.
- (b) *The installation of fixed platforms.* In § 92 one learns that the cost of standby time of construction equipment due to unworkable sea state was £0.585 million out of £0.945 million. This is presumably exclusive of Company overheads and certainly of loss of production due to time delays. It would be informative if the Authors could give an approximate figure for the cost of such delays.
- (c) *The burying of submarine pipelines.* The line from West Sole took 70 days to lay and 13 months to bury, or 15 months for some 40 miles, without achieving the design depths in all sections (§§ 75 and 82).

While the North Sea is by no means a criterion for costs elsewhere, the above illustrates the early 'state of the art' in three main sectors. This is emphasized by 28 major exploration rig mishaps in thirteen years.

139. Those not familiar with the design of semi-submersible drill rigs may not appreciate that there are few criteria for design from the structural engineering or shipbuilding industries.

140. Applied forces on steel-framed structures can be assessed within reasonable limits and fatigue is seldom critical. A ship with its stressed-skin and bulkheads has

a large built-in factor of safety while there is immense store of practical knowledge of sizing of members in the light of experience. Apart from such novelties as connexions between 9 ft and 36 ft dia. members, the applied loads to the floating structure which is a drill rig, and their repetitions, are still unknown and this is aggravated by the scale of their application.

141. The nearest comparison comes from the aircraft industry. Here the customer normally demands a warranty from the manufacturer covering all structural defects for a specified number of flights (say 10 000) with a shared warranty for a further number (say up to a total of 20 000) at which stage relief of warranty is given. The manufacturer has a still higher number of flights (say 30 000) in his 'design aim'. The 'flight' is specified by the customer in a 'flight plan', e.g. the $1\frac{1}{2}$ h average duration of a European Service.

142. This specification covers the knowledge that up to 60% of the fatigue stressing may be associated with the ground/air phase of take-off and landing with a further 20% for climb and drop, leaving 20% for level flight at altitude. In order to make the warranty operative, accelerometers and other recording instruments are installed in each plane. The manufacturer can be relieved of this warranty for usage not in accordance with the flight plan and for 'matters outside the state of art' as well, of course, as normal maintenance. To meet all these requirements, testing of full-scale models of the frame is undertaken by repetition stressing of the wings by jacking, pressure testing of cabins, and various repetitions of simulated 'test flights' imposing alternative 'gusts'. The testing of a new model may well cost £1½–2 million.

143. It is not claimed that such procedure can be applied in the case of the individual structures comprising the world's drilling rigs but perhaps it contains an ideal to be aimed at. Meanwhile a true factor of safety of 6, as in much of the aircraft industry, may well be a reasonable criterion.

144. It would be interesting if the Authors would compare the delays to construction due to weather while installing fixed platforms with those on the semi-submersible *Sea Quest* on location. These would surely point to the inefficiency (hence high cost) of the type of equipment used for installing the towers or to the need for another type of design of the towers themselves to eliminate such losses.

145. In the depths of 100 ft or thereabouts, would a jack-up construction platform as used in the Arab Gulf not have been cheaper overall? In deeper water, using floating equipment, would improvement and very large cost savings not result from a basic redesign of such equipment? Recognizing that the eccentric loads from a crane-hook differ in kind from the mainly centralized loads from the drill string and require different architecture of the resulting craft, it would appear that this equipment would be much easier to operate (if the contractor has the will) as well as offering greater profitability to employer and contractor alike than some more esoteric proposals now being studied for underwater developments.

146. The burying of the pipe under the conditions described must also be characterized as inefficient, although the barges were immensely powerful. If jetting is to be successful in boulder clay, a water pressure higher than 2000–2500 lb/sq. in. at the nozzle may be necessary and larger water quantities would have to be pumped. Perhaps this marks the upper limit of practical jetting. A new direct cutting underwater trencher will soon be available and may provide an answer. Fortunately, not every soil is a boulder clay and most can be jetted satisfactorily: this may prevent development of new techniques. Would the use of explosives for pre-trenching have been economic?

Mr G. Halbron, SOGREAH, Grenoble

The operation of the semi-submersible platform *Sea Quest* in the North Sea is of considerable interest as it is the first example of the use of a structure of this type in the very arduous conditions of the North Sea. It is common knowledge that one of the main difficulties with floating drilling rigs is anchoring. Could the Authors give

DISCUSSION

details of the anchorage arrangements used for *Sea Quest*? Have these arrangements always performed satisfactorily or have they given trouble under certain weather conditions? Do the Authors feel that a self-propulsion system would be a desirable aid in certain cases?

148. Have any measurements of the rig's movements due to waves and currents been made and correlated with natural conditions, or is such a series of measurements envisaged?

Messrs G. B. Marriott and V. J. R. Sutton

The Authors wish to thank the contributors to the written discussion and are gratified at the wide range of interest which has been demonstrated.

149. On the design of submarine pipelines, **Mr Flaxman** will appreciate that not only the sea bed conditions but such factors as recommendations by regulatory bodies, the degree of exposure of the pipeline route, contractors' plant available, and the water depth along the route will all have bearing on the adopted design. These can be governed, for example, by the laying procedure and the time of year when the laying was carried out rather than by the operating conditions.

150. With reference to **Mr Muir Wood's** comments, the Authors freely recognize that there is a great deal to learn about the magnitude of wave forces and the dynamic response of offshore structures. The oil industry is certainly not complacent about the present state of affairs. Since the Paper was prepared the results of several wave and wave force investigations carried out in the Gulf of Mexico have been published¹⁶ and we think he will be impressed by the considerable efforts now being made by the oil industry to improve the design of offshore structures. However, the difficulties and expense of this type of research should not be overlooked; a modest investigation for measuring wave forces on a platform in the Gulf of Mexico was recently costed at £400 000. (See also § 175.)

151. Whatever may be the technical shortcomings of the analysis techniques, we can at least claim that our structures are surviving the acid test of time. In the Arabian Gulf we now have about 70 fixed platforms, some of which have been in place for over ten years. Many are in the incipient breaker regime and there has not been a single instance of structural failure which could be ascribed to wave action.

152. In the recent research work in the North Sea and in the Gulf of Mexico the emphasis has changed to the measurement of waves and structural stresses rather than upon determination of wave forces. An examination of some of the work¹⁶ shows that of the possible errors in estimating wave forces, the wave height assessment was the major source, whereas the possible inaccuracies in the wave force theories were comparatively minor. The inference is drawn that reasonably satisfactory correlation between theory and practice exists, maximum errors being of the order of 15%, provided that an accurate assessment of the maximum storm wave characteristics has been made.

153. On the questions raised about well-drilling programmes, the problems mentioned by **Mr West** in § 106 are reduced if it is planned to place cement in all casing strings up to within about 200 ft below sea bed level, the outer conductor tube being cemented to surface. This programme, we feel, minimizes the effects of temperature differences and the possibility both of washout and of subsidence. Temperature variations then result in stresses being locked into the system of casing strings, which are designed accordingly.

154. We agree with **Mr West** that, when drilling afloat, a permissible off-hole displacement of about 3% of the riser length is indeed very conservative, and that today a displacement of about twice this value is becoming normal practice.

155. With regard to the question in § 108 on site investigations, no attempt had been made up to the time covered in the Paper to measure in situ shear strengths for the reason that the borings were carried out from a floating craft, and with the

equipment employed in situ testing was not feasible. Since the Paper was prepared, we have made some in situ soil strength determinations using U4 sampling and SPT equipment during percussion drilling, from a jack-up platform. Equipment we have available for in situ strength determinations is described in § 42, but this has not yet been tried out.

156. The question in § 109 about the maximum design wave raised interesting philosophies having bearing on such matters as the required life of the marine platform, the probability of collision damage and the allowable structural stresses. We hope it will suffice to say that our practice in adopting the 100 year storm wave criterion is based upon design philosophies accepted some twelve years ago for structures in the Arabian Gulf and that these have been shown so far to be adequate. It should be borne in mind (see § 51) that we design to normal working stresses with no overstress allowance for transient loading.

157. It will be appreciated that the values of C_D vary with the effective diameter of the member and those of C_m with the diameter and water particle velocity. Examples of the values adopted are:

Effective diameter of member <i>in.</i>	C_D	C_m
12	0.64	1.36
48	0.76	1.43

158. On the question of the occurrence of scour around legs and piles of fixed structures in the North Sea, BP has, after survey, been able to select sites where there are only 6 in.–2 ft of sand and this overlies very hard boulder clays having a cohesive strength of up to 4000 lb/sq. ft. In these conditions scour has been no problem and no allowances have been made for it. Periodic diving checks have been made over the past 3 years and confirm that no scour is occurring.

159. In reply to Mr West's question in § 112, as mentioned in § 46, the use of computer programs for final design of structures using fifth order wave theory loading is becoming a necessity, but preliminary and checking work may be done by hand. However, once the forces acting at individual points have been tabulated, a structural analysis using hand calculations without using a computer check may be adopted.

160. In reply to the questions raised by Mr Ruxton in § 114, the location to which the design criteria quoted in § 50 apply is latitude 53° 46' 27" North, latitude 01° 01' 44" East, where the local water depth is –104 ft CD. This location enjoys certain attenuation effects, for example from the Dogger Bank, and the wave height of 56 ft is the maximum which occurs statistically once in 100 years. The figures for the conversion from wave length to wave period given by Mr Ruxton are first order values and do not include the correction for wave height (there is an interplay between wave length, period and height). If the correct water depth is used and the conversion from wave length to wave period taken to the fifth order, the value of 13.5 s will be found to be correct.

161. The storm surge height of 7.5 ft adopted (conservatively) at this location happens to be the same as quoted by Rossiter, but we would think that the 1953 storm surge could have a return period of about 400 years.

162. The Authors thank Mr Kohring for his observation. We are not aware that particular thought has been given to the possibility of piles wandering, causing high stress within the jacket members. However, on giving the matter some thought the following comments may help:

- (a) there is approximately 2½ in. difference between the o.d. of the pile and the i.d. of the jacket, and the bottom jacket connexion is specially strengthened; therefore some bending and rotation of the pile within the jacket can occur without causing excessive stress;

DISCUSSION

(b) although the piles when driven can be 240 ft long, they are none the less constrained by the jacket for approximately 145 ft of their length; therefore the section of the pile extending below the jacket, being driven into the soil, can be considered stiff. When driving the hollow pile into a material such as very firm boulder clay (BP experience) it does not require much penetration before the material itself serves as a guide to prevent wandering. If the piles are driven into soft sand it is unlikely that wandering would occur in the early stages of driving and in the later stages the radius of curvature would be such that excessive stresses should not be produced within the jacket.

163. In reply to **Mr Cornfield**, if we underemphasized the magnitude and complexity of the problems resulting from severe site conditions, this is regretted since we are only too conscious of these problems and the great efforts that are made to combat them.

164. Since the publication of the Paper, 30 in. dia. steel tube test piles of 1½ in. wall thickness × 260 ft length have been driven successfully in 10 ft increments down to a maximum of 60 ft into the hard boulder and lias clays and tested to failure at each level in both tension and compression. These tests show that the true soil shaft adhesion values are considerably higher than those normally used in industry, and confirm **Mr Cornfield's** opinion that overdriving of piles takes place. For a recent platform installed by BP the wave equation analysis was employed to determine loadings that could be achieved when using various hammers in association with piles of different wall thicknesses and different penetrations. These analyses did in fact give a very close correlation with the results achieved by the pile test.

165. There is a very real danger in assuming that pile capacities can always be determined from hammer size, energy and set. This approach normally indicates that the greater the hammer energy the greater the pile capacity will be given for a given set. On a platform recently installed at least one pile was driven to refusal at 33 ft employing a hammer of 180 000 ft/lb capacity. A 135 000 ft/lb hammer of different characteristics was then employed and drove the pile a further 20 ft. Repeatedly on this platform installation the 135 000 ft/lb hammer drove the piles further after refusal had been obtained with the larger hammer. It seems that the stiffness (wall thickness) of the pile is important and also that the hammer energy is not necessarily the main criterion, other factors such as hammer strike velocity, stress wave characteristics and the elastic compression of the pile also appear to be very important.

166. Regarding §§ 128 *et seq.*, until we can be certain from static soil mechanics analysis exactly how deep piles must penetrate to achieve a required bearing capacity, and likewise be able to estimate with certainty the hammer capability, we *must* allow for the greater flexibility offered by tubular steel piles. These allow one to drill out, predrill and under-ream, and extend by 'add-ons', and incidentally are much better suited to the developed techniques of grouting and shimming the piles within the jacket and setting the deck sections. The proposed four 12 × 12 H beams weighing 760 lb/ft appear to be expensive in steel and require much extra welding compared with our 36 in. pipe piles, the 1 in. and 1½ in. wall sections used weighing ca. 360 and 460 lb/ft. It is confirmed that our test results fall approximately on the lower limit of **Mr Cornfield's** diagram.

167. The Authors endorse **Mr Scott's** comments on the choices open on drilling vessel selection (§ 133). The choice of vessel involves judgement certainly, but in off-shore operations the cardinal factors are adequate accuracy with maximum speed, particularly in the North Sea where perfect operating conditions can be met in January and gale conditions lasting for days or weeks can occur in July. Not only is the choice of vessel but also the choice of drilling method influenced by sea and weather conditions. We do not agree that a large or relatively large vessel is necessary in handling the equipment used in the 'quick' method of boring, but we have

used a large vessel so that the chances of staying on location during rough weather are improved. Likewise we have used the rotary 'quick' method rather than the conductor tube 'slow' method for the same reason, although it may not give such ideal results. It may be possible to hold location for 24 h, but to do so for two or three days is much more unlikely.

168. **Mr McFeeters's** comparison of the structural problems involved in the design of semi-submersibles with those of the aircraft industry is very pertinent. It is readily agreed that the 'state of the art' with regard to offshore structures is 'early', particularly as far as North Sea operations are concerned. This is generally accepted to be one of the worst areas of the world in which to operate.

169. Costs relating to early BP platforms are given in § 92 but in June/July 1969 platform C was installed with virtually no time lost due to bad weather. Installation time was cut by driving an increased number of 1 in. and 1½ in. wall piles of uniform section to depths within the hammer cap ability and bearing capacity limits based on a given set. The cost of the steel requirements has increased but the high work barge costs that would result from drilling out procedures are considerably reduced.

170. The Authors cannot agree that a work barge on jack-up principles would be a step forward for water depths up to 100 ft for the following reasons:

- (a) a work barge can work in areas where it may be unsafe for a jack-up to set down its legs due to inadequate bearing capacity of the sea bed;
- (b) it is difficult to conceive a 'jack-up' design which allows a large 300 ft × 100 ft open work area unimpeded by protruding legs;
- (c) a work barge is mobile and its position can be altered for particular operations involved with jacket launching, etc. merely by pulling on its anchor cables;
- (d) work barges have the capacity to lift loads of 500 short tons rotating and 800 short tons with a fixed jib causing the barge to list approximately 3°; such loads would produce very high leg loading on a jack-up;
- (e) work barges as at present conceived were developed to meet American offshore conditions. They are very suitable also for offshore conditions in many parts of the world and have been used with a fair measure of success in the North Sea. It would seem impractical to develop a new concept of barge to meet the particular conditions here when it is unlikely that the scope of work would justify it, even if it were shown that there was some merit in such a design.

171. **Mr McFeeters** raises interesting points in § 146 regarding the laying of pipelines underwater. It must be admitted that the laying of the lines in hard boulder clays is inefficient and often unsatisfactory. The present technique has been developed whereby the pipelines are first fabricated and laid on the sea bed and subsequently a bury barge traces the pipeline and carries out a simultaneous trenching and burying operation by jetting out the material beneath the pipeline. To develop a new technique whereby the trench is cut ahead of the pipelaying operation involves problems with the trench being in-filled with sand and difficulties with laying the pipeline in the trench. If the trench is cut to the side of the pipeline after laying there are problems with moving the pipeline into the trench. A further practical problem with a direct cutter underwater is one of survey and setting out. Few would claim that in North Sea conditions the present scheme is satisfactory but it would seem that the answer would lie in the development of present equipment rather than developing new techniques.

172. In reply to **Mr Halbronn**, although *Sea Quest* was one of the first semi-submersibles in the North Sea, other rigs of this type have worked there; these are *Ocean Prince*, *Ocean Viking*, *Ocean Traveller*, *Staflo* and *Sedneth II*. The Authors would agree that the mooring of these structures is the main problem and one which is likely to be with us for some time to come. Most of the floating rigs in operation in the North Sea have experienced mooring problems at some time in their career. Wind, wave and current forces under extreme weather conditions are enormous and

DISCUSSION

tension in the mooring of up to 800 000 lb is possible. To date, *Sea Quest* has been anchored in very good holding ground and tensions of up to 370 000 lb have been sustained without pull out of the anchors. The main problems have been mechanical damage of the ropes, fatigue and kinking. Design changes to the winches and rope fairleads have effected a big improvement, but kinking of the ropes due to built-in torque continues to be a problem. The *Sea Quest* has nine moorings, each 3400 ft long, of 2½ in. dia., 6 × 36 rope, with independent wire rope core. The anchors are 30 000 lb Danforth type.

173. Self propulsion would be a very desirable feature but would be practically impossible to install on *Sea Quest*. There have been reports of a drilling contractor having placed an order for a self-propelled semi-submersible.

174. Tests have been carried out at the National Physical Laboratory on a model of *Sea Quest* at 75 ft draught (the normal drilling draught), in the moored state. It was found that the response of the vessel proved to be greater in irregular wave spectra than in monochromatic waves, and there was good correlation between measurements on model and prototype.

175. In conjunction with British Ship Research Association, accelerometers have been installed aboard *Sea Quest* and measurements of roll, pitch and heave are taken as a continuous routine. Yaw movement can be observed by a non-recording gyro compass and it was assumed that surge and sway movements could be observed from an installed Baylor well position indicator. Surge and sway readings have, however, proved to be most unsatisfactory, as the apparatus relies upon the inclination of a taut wire attached near to the sea bed, and the straightness of this wire is influenced by winds and waves whenever these become appreciable. Nevertheless the three degrees of motion important to the drilling operation are satisfactorily and continuously monitored. It may be of interest that strain gauges permanently located at structural connexions at points of high stress on *Sea Quest* are continuously recorded. With reference to § 52, the use of these records is directed to the assessment of the fatigue life of the structure rather than to the direct measurement of wave or wind forces.

References

15. ROSSITER J. R. A North Sea storm surge. *Phil. Trans. roy. Soc. A.*, 1954, **246** 371–400.
16. EVANS D. J. *Analysis of wave force data. First Offshore Technology Conference*, Houston, 1969. Paper 1005. See also Papers 1006–1009.