OFFSHORE PILING PRACTICE IN THE NORTH SEA AND ARABIAN SEA
LA PRATIQUE DES PIEUX EN NORD MER ET EN MER ARABE

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SYNOPSIS

Foundation design for offshore platforms in the Arabian Sea is based on practice from the Gulf of Mexico. This paper compares that practice with the larger hammers, piles and crane barges used in the North Sea. Analyses of axial capacity and driveability of piles follow similar methods in all areas. However, allowance has to be made for local soils such as carbonate sands. Soil resistance during pile driving in sand appears to be no more than that assessed for axial capacity, for two sites in the Arabian Sea. Use of large hammers imparts vibrations into the platform structure. Analysis for such vibrations are described. The stability of an offshore structure during piling may be assessed from the critical wave height for stability rather than as a function of safety for a given installation wave.

INTRODUCTION

The simplest offshore jacket structures have legs through which piles are driven, with bracings between the legs. Topside loads are applied directly onto the piles. Where loads are too great for the one main pile at each leg, skirt piles can be driven through guides parallel to the leg and grouted into sleeves at the base of the jacket (see left side of Fig. 1). Piles may be installed in several pieces, to suit the hook height and capacity of available crane barges, and safe lengths of pile stick-up. Foundation layouts of leg piles and skirt piles are common in the Gulf of Mexico, Southern North Sea, and Bombay High. The largest hammer used in the Bombay High field is the Menck 5000, (see Table 1).

Major platforms in the North Sea use clusters of piles at each corner of the structure. Leg piles are not used in deeper water because of the time required to weld add-ons offshore. Vertical piles are now driven by underwater hammers, so pile guides and followers are generally not required (see right side of Fig. 1).

The largest underwater hammers used to date in the North Sea are the IHC S-2300, and the Menck MHU-3000 (see Table 1).

Piles may be connected to structures by welding above water or grouting or swaging underwater. High strength grouts or swaged connections, as developed for the North Sea, give shorter and hence cheaper connections.

The grout bond formulae given by UK Department of Energy (1990) are based on an extensive series of large-scale tests for North Sea structures. Mechanical swaging is now used for most subsea templates in the North Sea, and has been tested for fatigue endurance by Lowes et al (1992).

Table 1. Comparison of Pile Driving Hammers

Hammer	Ram Weight	Lift Weight	Rated Striking Energy
Menck MRBS 5000	50t	150t	750kN-m
IHC S-2300	100t	315t	2300kN-m
Menck MHU-3000	168t	535t	3000kN-m

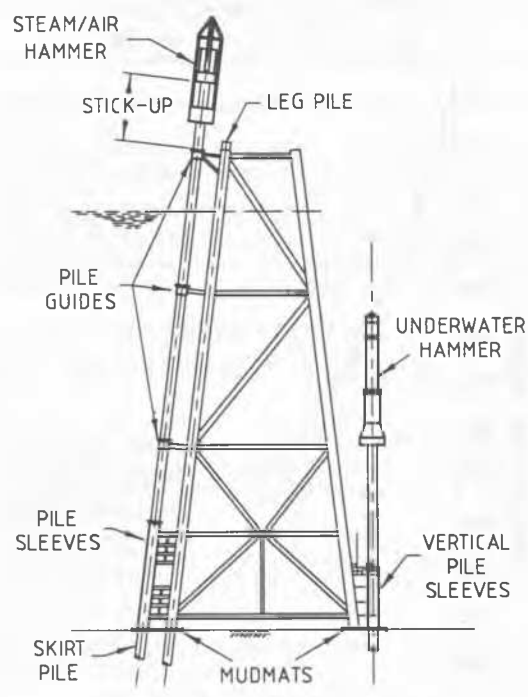

FIG. 1 OFFSHORE PILED FOUNDATION CONCEPTS

AXIAL CAPACITY OF PILES

API RP2A (1991) recommends that the capacity of piles be calculated as the lesser of plugged and unplugged capacities with:

* unit skin friction in clay $f = \alpha\ Cu$

  where α depends on the shear strength (Cu), overconsolidation ratio and overburden pressure on the clay.

* unit skin friction in sand $f = K\sigma v^1 \tan \delta$

  where

  K = coefficient of lateral earth pressure
  σv^1 = effective vertical stress
  δ = soil/pile skin friction

* unit end bearing in clay $q = 9\ Cu$

* unit end bearing in sand $q = Nq\ \sigma v^1$

Piles in the North Sea have generally been designed to the 14th Edition of API RP2A, with $\alpha = 0.5$ for undrained shear strength greater than 72 Kpa, and K = 0.7 in compression and 0.5 in tension. Unit skin friction and end bearing in calcareous sand are limited as recommended by API RP2A. For driven piles in cemented carbonate sands, limiting skin friction is less, because sand particles arch between the pile and intact cemented material. Cemented carbonate sands are found offshore India, Australia and Southern Africa.

Predicted pile capacities for a predominantly clay site offshore India are shown in Fig. 2, for two different methods of calculation:

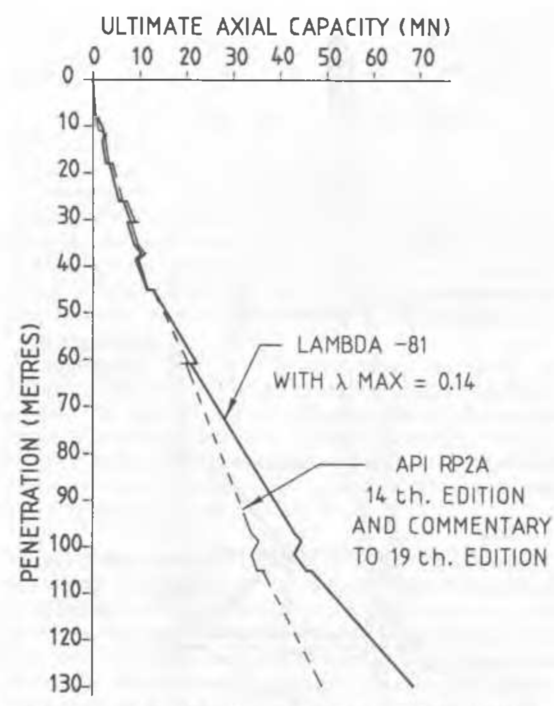

FIG. 2 AXIAL CAPACITY Vs DEPTH COMPARISON

a) API RP2A 14th Edition.

b) λ - 81 method of Kraft et al (1981), with skin friction in clay

$$f = \lambda\ (\sigma v^1 + 2\ Cu)$$

where λ is a constant for a given pile penetration, derived from empirical correlations. Kraft et al (1981) recommend a minimum value of λ of 0.14, which governs for deep piles. Fig. 2 shows capacities based on a correlation by Kraft et al (1981) between λ and length. Higher capacities are predicted from the variable-α method of API RP2A (19th Edition). The 14th Edition of API RP2A is conservative at deeper penetrations. Some consultants recommend a single tailored value of alpha as a mean fit to other predictions.

End bearing in sand layers above/below clay layers is assumed only to be mobilised 2-3 diameters below, or 3 pile diameters above, weaker layers, as recommended by API RP2A (1991).

PILE DRIVEABILITY

Driveability of piles can be assessed from analysis of Soil Resistance at the time of Driving (SRD) as a function of:

i) depth of penetration
ii) blow count, by wave equation analysis

For SRD vs depth, outside skin friction in sand during driving is generally assumed to be the same as for axial capacity. However, skin friction in clay during driving is generally less than for ultimate axial capacity. Toolan et al (1977) recommend that skin friction in clay during driving be assessed from the remoulded shear strength. However, remoulded shear strengths must be carefully assessed, to avoid irrational values of sensitivity. The stress-history method of Semple et al (1981) gave a better correlation of observed driving behaviour at a North Sea site where piles were driven through overconsolidated clay into normally consolidated clay (Rigden et al, 1983).

For offshore India, hardest driving conditions are generally found in dense or cemented sand layers (Stockard, 1986), calcisiltite or calcarenite. Blowcounts vs depth are presented for two sites offshore India in Figs. 3 and 4. Back analyses were performed using conventional wave equation analysis, assuming a hammer efficiency of 75%. Skin friction and end bearing were calculated:

* in clay, as recommended by Semple et al (1981).
* in sand, as for axial capacity (see above)
* internal skin friction 50% of external, as recommended by Stevens et al (1982) for lower bound predictions.

The first site had calcareous sand from 31m to 40.5m below mudline for which a limiting end bearing of 10MPa was assessed by the soil consultant. The predicted SRD in the sand layer is 11MN (unplugged) or 17MN (plugged) for 1.219m (48") piles. Wave equation analyses predict 20-40 blows/foot for the Menck 5000 hammer. Piles

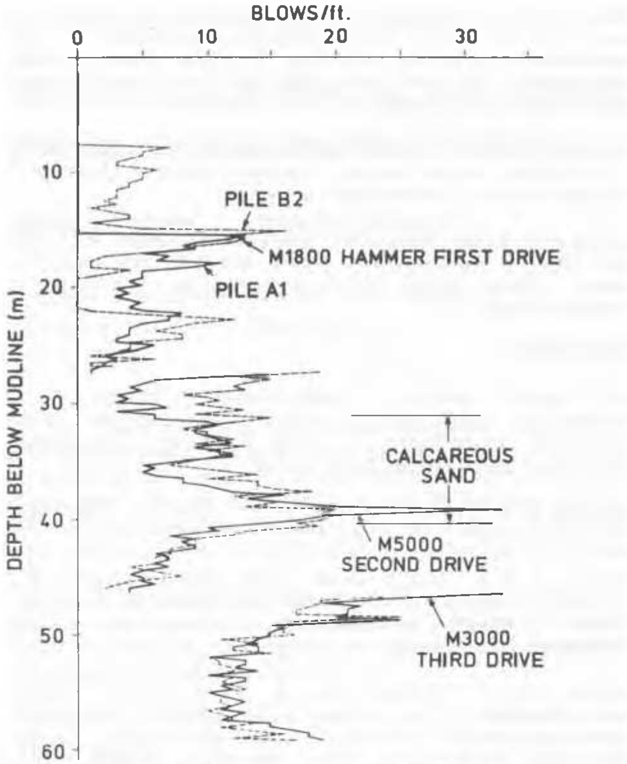

FIG. 3 BLOWCOUNT VS DEPTH (SITE 1)

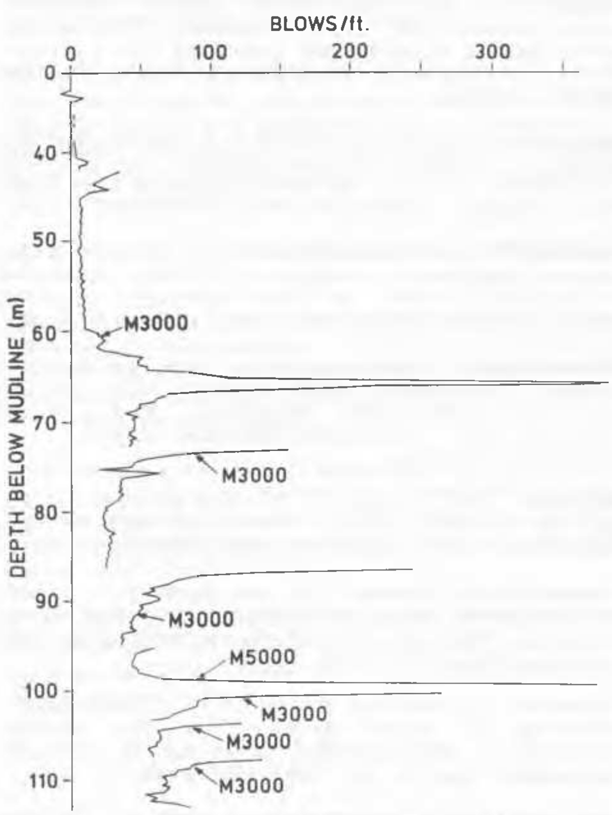

FIG. 4 BLOWCOUNT VS DEPTH (SITE 2)

were driven from 28m to 46m penetration by the Menck 5000 hammer at a maximum blow count of 33 blows/foot(see Fig. 3).

The second site had calcarenite layers which gave significantly higher blowcounts (Fig. 4). The predicted SRD in the calcarenite layer at 99m penetration is 20MN (unplugged) or 30MN (plugged) for 1.524m (60") piles with limiting end bearing of 10MPa. Wave equation analyses predict blowcounts of up to 700 blows/foot of the Menck 5000 hammer for a SRD of 30MN, assuming plugged behaviour with a maximum end bearing in the calcarenite of 10MPa. Only one pile was driven through the calcarenite layer at 99m penetration, at blowcounts of up to 700 blows/foot (see Fig. 4).

Stevens et al (1982) recommend the use of an upperbound soil resistance for very dense sand and rock. The upperbound soil resistance is derived by increasing skin friction in sand by 30%, and increasing end bearing by 50% in sand and 67% in clay. Use of the upper bound soil resistance predicts significantly higher blow counts than recorded at these two sites.

PILE DRIVING VIBRATIONS

When battered piles are driven by steam hammers, the jacket structure vibrates, particularly at the start of a drive when contact forces are greatest between pile or follower and leg or guide. Damage to anodes and other attachments is rare, but not unknown (Salama et al, 1988). When piles are driven by an underwater hammer, vibrations of the structure are applied only through the pile sleeves.

Vibrations of the order of 40 - 100 g were measured by Mobil during pile driving with the Menck MHU1700 underwater hammer at Beryl 'B' in 1983 (Thompson et al, 1985). The magnitude of vibrations can be estimated by:

a) estimate energy loss into the pile sleeve

b) compute mass plus added mass of pile sleeve.

c) hence compute maximum velocity of pile sleeve assuming that it vibrates as a unit.

d) examine acceleration time records from instrumented pile tests to idealise the form of the stress wave in the pile (typically a half-sine wave of duration 5 - 10 seconds).

e) compute maximum acceleration.

Potential fatigue damage can then be assessed from:

f) natural frequency of the attachment, and hence dynamic amplification factor for initial undamped vibrations from Clough et al (1975).

g) maximum stress, Smax, from one hammer blow, multiplied by stress concentration where appropriate.

Fatigue damage to attachments from n hammer blows may be calculated as:

$$D = n\,K^{-1}\,(2\,S_{max})^{m}(1-\exp(-2\pi m\xi))^{-1}$$

where

ξ = damping factor (typically 0.02 in water)

and the permissible number of cycles N for a stress level S is expressed in terms of empirical constants K and m (UK Department of Energy, 1990) as:

$$N = K S^{-m}$$

Anodes and other attachments designed by the above procedure have successfully resisted pile driving vibrations.

UNPILED STABILITY

An installed but unpiled structure must resist sliding, leg uplift, and bearing failure. The capacity of the mudmat at each corner of a jacket can be expressed as an interaction between horizontal and vertical capacities (Fig. 5). The bearing capacity for purely vertical loads (point A in Fig. 5) reduces to point B as horizontal load increases. The horizontal capacity with no vertical load (point D in Fig. 5) is caused by passive resistance on pile sleeve and leg extensions below mudline. As vertical load increases, horizontal capacity increases due to friction between the sea bed and the underside of the mudmat and higher passive pressure from the surcharge. A maximum horizontal capacity (BC in Fig. 5) is set by failure around all leg and sleeve extensions.

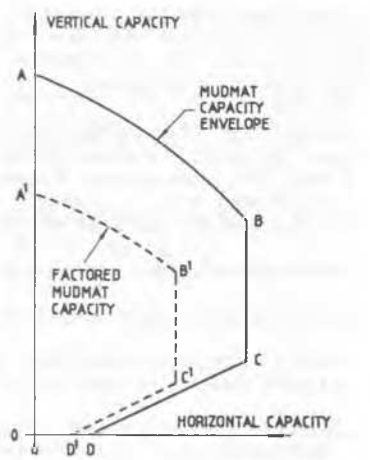

FIG. 5 MUDMAT CAPACITY INTERACTION DIAGRAM

API RP2A (1991) recommends a minimum factor of safety against bearing and sliding for an installation wave. Factored capacities A^1, B^1, C^1 and D^1 are indicated in Fig. 5. An alternative approach, as adopted for many North Sea jackets, is to compute foundation forces for a range of wave heights and find the critical wave heights which just cause sliding, zero leg reaction or bearing failure. The risk to the structure can then be assessed from the probability of exceeding critical wave heights during installation.

CONCLUSIONS

Pile driveability in the Arabian Sea can be assessed by similar methods to those used for the North Sea. However, end bearing in sands and soft rocks must be carefully assessed. Soil resistance during driving in the case records presented is not as high as the upper bound reported by Stevens et al (1982).

Unpiled stability may be assessed as the risk to a critical wave height rather than a factor of safety on an installation wave.

Ultimate pile capacity varies between API RP2A and the λ method but such variations are much less than the differences in drivability predictions.

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