Design and installation of piles in chalk

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AMOCO INTERNATIONAL OIL COMPANY has installed drilling and production platforms in Rough Field, in the British sector of the North Sea. The platform site is located about 30 miles (48km) east of Hull; an approximate location of the study area is shown in Fig. 1. Each platform has eight 36in (914mm) OD main pipe piles. A quasi-effective stress method was used to compute capacity and design penetration of piles in chalk. Piles were driven to design penetrations which averaged 183ft (56.8m) for drilling platform and 130ft (39.6m) for production platform, using Menck 1500, 2500, 2500SL and 8000 hammers. Very long delays which occurred during pile driving were utilised to study the effect of soil-set on pile driveability.

This Paper describes the procedures used in designing and installing the piles, analysis of soil resistance during pile driving and the effect of delay on pile driving.

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Site conditions

Field exploration

The soil conditions at the site were defined by two borings about 200ft (61m) apart, drilled to penetrations of 113ft (34.4m) and 225ft (68.6m) below sea-floor. The boring and platform locations are shown in Fig. 2. The borings were drilled in 125ft (38.1m) of water from the drilling ship, Explorer.

Soil samples were obtained with a wire-line operated sampler as described by Vijayvergiya and Krause. According to this method, a boring is made to the required sampling depth utilising an open-ended drill bit. The drill pipe is lifted 2.5ft. (0.6-1.5m) and a wire line sampler is lowered through the drill string. An open-ended tube, attached at the bottom of the sampling device, is driven into the soil by raising and lowering a sliding hammer with a drop of 3-5ft (0.9-1.5m). The hammer is raised and dropped repeatedly until about 18-24in (457-610mm) of penetration is achieved.

Foundation conditions

The soil stratigraphy revealed at the location of the two borings is generally the same. Detailed boring logs have been presented by Vijayvergiya & Cheng, 1976. In brief, hard grey and brown silty clay extends to approximately 47ft (14.3m) below the mudline. The silty clay in Boring 1 overlies a very stiff to hard light grey and white chalk to at least 225ft (68.6m) below the sea floor.

The chalk is very brittle and contains seams of clay and claystone and, occasionally, thin gravel layers. It is believed that this chalk belongs to the Upper Cretaceous Period.

The interpreted undrained shear strength and remoulded strength profiles at the site are shown in Fig. 3. In view of the brittle character of chalk and the disturbance associated with drilling and sampling operations, reliance was placed on consolidated-undrained triaxial tests in arriving at the undisturbed strength profile.

Pile design procedures

Review of load test data in chalk

From land based pile load test data, Tomlinson has suggested using allowable values of unit friction, f, and unit end bearing, q, of 0.30tsf and 20tsf, respectively. Meigh, based on results of several pile load tests in chalk, has concluded that Tomlinson's values include a safety factor of about three. Kee and Claypham have recommended allowable values of f ranging from 0.20tsf to 1.00tsf, including a safety factor of two. These values are based on correlations with Standard

![Fig. 1. General site location](image1)

![Fig. 2. Plan of borings](image2)
Penetration Test Values, $N$, as shown in Fig. 4, Lake and Simons have observed ultimate $q$ values ranging from about 60tsf to 155tsf from plate loading tests performed in deep holes. Wakeling found the foundation behaviour of softened chalk similar to granular soil and suggested the use of an effective stress approach for foundations on chalk.

**Effective stress method** suggested by Vijayvergiya and Focht was utilised to compute pile capacity. According to this method, the skin frictional capacity, $Q_s$, along the shaft is computed in the following manner

$$Q_s = \lambda (\bar{\sigma}_n + 2c_u) A_s$$

where $A_s = \text{Embedded surface area of the pile}$,

- $\bar{\sigma}_n = \text{Mean undrained shear strength along the embedded length}$
- $c_u = \text{Mean effective vertical stress along the embedded length}$
- $\lambda = \text{A non-dimensional friction coefficient}$

The values of $\lambda$ are based on analysis.

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Fig. 3. Undrained shear strength profile based on Borings 1 and 2

Fig. 4. Relationship between $f$ and $N$

Fig. 5. Frictional coefficient, $\lambda$, vs. pile penetration

Fig. 6. Pile capacity curves, 36in (914mm) OD pipe piles
lyses of data from 47 pile load tests conducted at various geographic locations. The non-dimensional friction coefficient, $\lambda$, is found to depend on pile penetration. The relationship between $\lambda$ and pile penetration is presented in Fig. 5.

The skin frictional capacity, $Q_s$, was computed from the relationship presented in Eq. 1 and the values of $\lambda$ presented in Fig. 5. This procedure resulted in a unit skin friction value of slightly over 1.00tsf below 170ft (51.8m) penetration. The end-bearing capacity, $Q_e$, was computed using a value of $q = 50$tsf. Computed pile capacity curves for 36in (914mm) OD pipe piles in tension and compression are presented in Fig. 6.

**Design pile penetrations**

The piles were designed for ultimate compression loads of 1500 tons for the production platform and 1800 tons for the drilling platform. The ultimate loads were based on a safety factor of 1.5.

According to computed ultimate pile capacity curves (Fig. 6) pile penetrations of 130ft (39.6m) and 160ft (48.8m) were required for production and drilling platform, respectively, with compression governing penetration requirements. At these depths, the factor of safety in tension was greater than 1.5.

**Axial load-movement curves**

Most design procedures assume that the ultimate capacity of a pile is a unique quantity; this is not true. The mobilised capacity of a pile at any given penetration is a function of axial pile movement. This fact is well-known from pile load test data, but usually overlooked. For deeply penetrating piles, it is important that load-movement relationships be estimated for piles at the sea floor level to achieve compatibility of superstructure and foundation deformations.

Relationships between axial load and pile movement were obtained for pile penetrations of 130ft (39.6m) and 160ft (48.8m). A procedure similar to the one suggested by Coyle and Reese was utilised. This procedure is based on the assumption that as the pile undergoes vertical deformation under a load, the soil provides resistance at the tip and along the sides of the pile. The magnitude of resistance acting on the pile is dependent on the strength and stress-deformation characteristics of the soil.

In analysing load-movement behaviour, a pile is divided into a number of segments as illustrated in Fig. 7a. An axial movement at the pile tip, $Z_e$, is assumed and a corresponding load at the tip, $Q_e$, is determined from the estimated tip load-movement curve for the soil (see typical relationship in Fig. 7b). This tip load is then used to compute the elastic deformation of the bottom pile segment. The movement of the centre of this segment, $Z_m$, is obtained by summing the assumed tip movement and one-half of the elastic deformation for that segment. Load-transfer along the surface of the segment $Q_m$ is then computed using a load-transfer curve of the type shown in Fig. 7c. The axial load acting at the top of the pile segment then becomes the sum of $Q_e$ and $Q_m$. The elastic deformation of the bottom pile segment is then recomputed using an average of $Q_e$ and $Q_m$ and the load-transfer value along the surface of the segment is revised based on the new elastic deformation value.

This procedure is repeated until the difference between the computed load-movement values converges to a tolerable value. The load on the top of this pile segment then becomes the load on the bottom of the next segment. This procedure is repeated for several assumed values of pile tip movement to obtain a load-movement curve for the pile head. Unit skin friction and end-bearing versus deformation relations, used in the above described analyses, were developed in accordance with the paper given by Vijayvergiya to the Ports 77 operation at Long Beach. These relations and the elastic properties of the 36in (914mm) OD pipe piles were input into a computer programme for developing the required pile load-movement relationships.

Results of these analyses are presented graphically in Fig. 8 showing the axial pile load versus resulting vertical movement at mud-line for the platform piles at 130ft (39.6m) and 160ft (48.8m) penetrations. The movement of pile at mudline corresponding to design loads was found to be acceptable.

**Pile installation procedures**

In the hard soils of the North Sea it has been a common practice to combine a driven pile with grouted insert pile. This results in considerable delay and expense in installation because of the time-consuming drilling and grouting operations. Based on drivability analysis, it was concluded that 36in (914mm) OD pipe piles could be driven to about 160ft (48.8m) penetration. Consequently, it was planned to install piles with hammers of rated energies ranging from 226,000-325,000 ft. lb. (31,245-44,932 kgf.m). A somewhat smaller hammer was planned to maintain continuous driving during periods when bigger hammers were inoperative. Contingency plans were made for drilling out the soil plug in case of refusal at inadmissible shallow depths. All but one pile were installed to adequate penetrations by driving alone with Menck hammers 1500, 2500 and 2600SL. A typical view of a hammer in operation is shown in Fig. 9. Average pile penetration for seven piles was 183ft (55.8m) at the location of the drilling platform against the required penetration of 160ft (48.8m). The eighth pile met refusal at a depth of 120ft (36.6m) due to increased soil resistance caused by long delays and, con-
Fig. 9. Menck hammer in operation

Fig. 10. Typical pile driving data

Fig. 11. Suggested relationship between delay and relative set-up

Fig. 12. Comparison of static and driving resistances
Conclusions

Based on the experience gained during soil exploration, design and installation of piles, and analyses of the observed pile driving data, the following conclusions may be drawn:

(1) Considerable judgement and experience is needed in interpreting the shear strength data on chalk. Consolidated-undrained tests should be performed to obtain meaningful shear strength characteristics.

(2) A quasi- effective stress method used in this Paper can be applied for predicting ultimate static capacity of piles in chalk. For the subject hard chalk, values of ultimate unit skin friction of 1.0stf and unit end-bearing of 1000lb appear appropriate.

(3) A nonlinear relationship exists between the delay time and the increase in soil resistance. A measure of this increase has been defined in terms of the relative setup, \( F_s \). This ratio, \( F_s \), increases rapidly with time and tends to reach a maximum value of about 1.8 for delays of several months.

(4) Soil resistance during continuous driving was about 65% of the static capacity computed by the \( \lambda \) method.

(5) The most probable static capacity is at least 15% greater than that computed by the \( \lambda \) method. Thus the actual factor of safety may be in the order of 1.70 rather than 1.5 used for the pile design.

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References


