A new interpretation of soil resistance for pile driveability analysis

Une nouvelle interprétation de la résistance des sols pour l'analyse de la possibilité de battage des pieux

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SYNOPSIS: Backcalculations of offshore pile and conductor driving records from the North Sea have been performed. The backcalculations supports the theory of friction degradation in clays during driving. For sands, the analyses show higher side friction than reported by others. The performed analyses have resulted in a model for static resistance during driving, and a quantification of soil variability.

INTRODUCTION

Prediction of pile driveability has since the development of Stress-Wave theory, (Smith 1962) improved in accuracy, and several approaches have since then been presented. The overall accuracy of todays methods is however still not acceptable for all types of soil conditions , and site specific correlations are preferred for a good prediction.

Most commonly used driving resistance models consist of a static resistance part and some damping contribution, which together form the dynamic resistance, in general formulated as:

$$R_d = R_s + D \quad (1)$$

where:

R_d = Dynamic resistance during driving
R_s = Static resistance during driving (SRD)
D = Damping.

One category of researchers has mainly been involved in development of the formulation of the damping contribution, and several alternatives exist. Most commonly used is the Smith type formulation where the damping is:

$$D = R_s \cdot J_s \cdot v \quad (2)$$

where:

J_s = Smith damping factor
v = Pile segment velocity.

It is however argued that the Smith damping factor has no physical meaning, and that the above model is inadequate. An alternative damping formulation, also widely used is:

$$D = J \cdot v \quad (3)$$

where:

J = Viscous damping
v = Pile segment velocity.

The viscous damping parameter is considered by many to be a better parameter that more accurately can be correlated to basic soil properties. Other formulations include non-linear damping, (Zandwijk et. al. 1983), while others model the damping contribution with separate and independent parameters, (Nguyen et. al. 1988).

The static resistance, R_s, is usually modelled as a linear elastic-plastic spring, and the other category of researchers has mostly focused on determining the soil spring yield level, and how to correlate this to basic soil parameters. Most emphasis has up to now been put on correlations of resistance in clay, and in particular to the static side friction along a pile. No prevailing method exist, and different approaches may give varying results.

The static resistance, R_s, is usually the major contribution in the total dynamic resistance. It is also shown by detailed back-calculations, (Middendorp and Zandwijk 1985), that different damping formulations give very similar dynamic resistances. It can be concluded that the uncertainty in the static resistance contribution by far exceeds the uncertainty in the damping contribution.

This paper focuses therefore on static resistance correlations both in sands and clays. It is the authors opinion that further development of advanced soil-pile resistance models has little value since there exist no commonly accepted soil parameter related basis.

Also, the paper focuses on the inherent uncertainty of any driveability prediction due to spatial soil variability. A quantification of the effect of soil variability is proposed in order to initiate a basis for a future probabilistic approach to this problem. Such approach has been followed in the design of the OSEBERG C jacket, performed for NORSK HYDRO A/S by Norwegian Petroleum Consultants, NPC Civil. The back-calculations are based on data from installation of the nearby OSEBERG B jacket.

CASE RECORDS

The basis used in the following analyses is the data obtained from the installation of the OSEBERG B piles and conductors, (well casings).

OSEBERG B is an 8-legged steel frame jacket installed in 1987 at the OSEBERG field in the

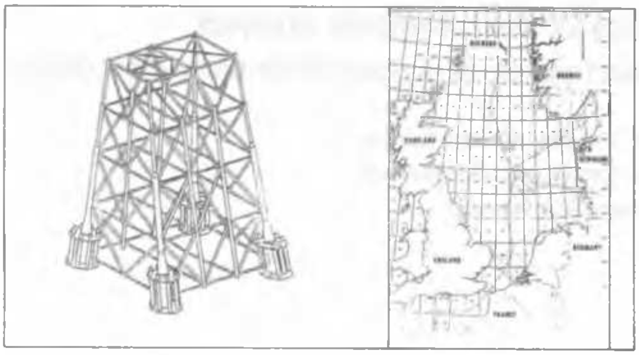

Figure 1. Oseberg B jacket.

North Sea, (see figure 1). The waterdepth is approximately 110m, and the jacket is supported by 32 piles arranged in four groups, one in each corner. The soil profile consists of dense sands and overconsolidated clays as shown in figure 2.

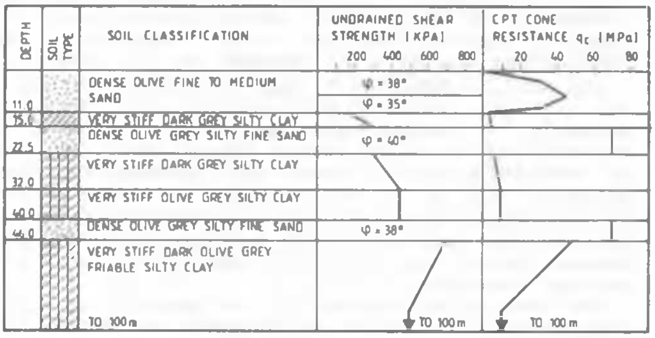

Figure 2. Soil conditions, Oseberg B.

The piles are steel tubulars with dimensions OD 2.134m (84") and wall thickness of 65mm. They were driven to 35m depth with a MHU 1700 hydraulic underwater hammer. Average efficiency of the hammer was measured to 0.73, corresponding to an input energy to the piles of 1240 kNm/blow. All piles were driven to target penetration without intermediate stops. Logging of inside soil column in five of the piles showed that no plugging had occurred during driving.

The blowcount records for the piles are shown in figure 3 as mean values along with +/- one standard deviation.

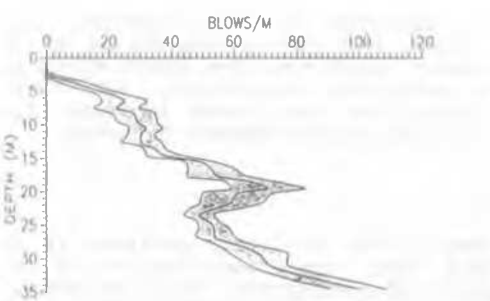

Figure 3. Blowcount record for piles.

The 31 conductors were installed from the jacket top with an S-400 hydraulic hammer. Conductor dimensions are OD 0.764m (30") with 38mm (1.5") wall thickness. A driving shoe, consisting of a ring of 12.7mm (0.5") thickness and 300mm height, was mounted on the outside immediately behind the tip, and also at certain intervals along the conductor.

The energy input on top of the follower was continously measured and recorded for each conductor. The average energy for all the conductors has been calculated at characteristic depths, and is presented in figure 4a. A range showing +/- one standard deviation is also included.

The conductors were driven to 115m depth using two different procedures. For most of the conductors, the installation sequence was driving to 50m, inside drillout and predrill to 105m, and then driving to 115m. For the rest of the conductors, an additional drill-out sequence was carried out to 30m, when the conductor tip had reached this level.

In general, the driving was halted at given depths for add-ons. These depths were 29m, 50m, 79m and 108m. The blowcount record, shown in figure 4b, reflect these stops as slight increases in blows at these levels.

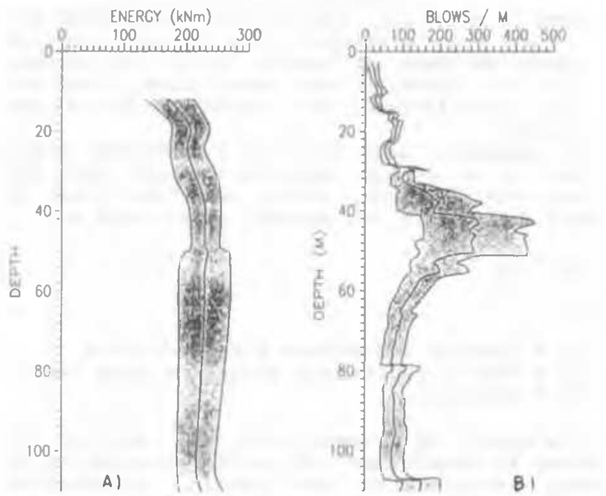

Figure 4. Blowcount and input energy record for conductors

It can also be seen from the blowcount record that there is a consistent decrease in blows per meter immediately after redriving from 50m. The blowcount is reduced as a function of penetration, and stabilizes at about 70 to 75m depth. It is therefore likely that no additional resistance is added to the conductors at depths greater than 50m, and hence the recordings reflect a degradation of outside friction along the conductor in the soil above 50m.

Such degradation of unit soil friction is earlier reported to occur in clay layers, (Heerema 1980), (Zandwijk et. al. 1983). A decrease in recorded blows per meter without a simultaneous decrease in soil strength, indicating the same effect, is also reported to occur during driving of the foundation piles for the Statpipe Riser platform, (Langø et. al. 1988).

BACKCALCULATIONS

Based on the recorded average blowcounts, back-calculations were performed using the Wave Equation program WEAP 86, (Goble and Rausche 1986). Soil quakes were set to 2.5mm both for tip and side resistance, and Smith's type of damping parameters were chosen. Tip resistance in clay was set to 9 times undrained shear strength.

By analysing in particular the changes in blowcount when the pile enters a sand layer from a clay layer, or visa versa, the different contributions to static resistance can be isolated and determined. This has been carried out at levels with distinct blowcount changes between mudline and 22m depth. The main conclusions from these analyses are:

* For plain pipes, the unit side friction in very dense sand is found equal to 1.5 times the static side friction calculated according to API (1987)

* The reduction in outside friction due to the outside mounted driving shoe is approximately 35% relative to above.

* Unit tip resistance in very dense sand is found equal to 3 times the static tip resistance calculated according to API (1987)

The unit side friction found for plain pipe is significantly above most common practice. The reduction in friction due to the outer driving shoe is similar to what is earlier reported for an inner driving shoe, and the unit tip resistance found is close to what is reported by Zandwijk et. al. (1983).

When the frictional contribution in the sand layers, as well as the effect of outside driving shoe is known, the unit side friction in clay can be determined as a function of driving depth for the conductors. By starting the backanalysis when the conductors are redriven from 50m depth, it is found that the undegraded unit side friction compares well with static side friction in clay calculated after API (1987). Further it is found that the friction is reduced to approximately 40% of its start value after 20 to 25 m driving. The degradation curve established is shown in figure 5.

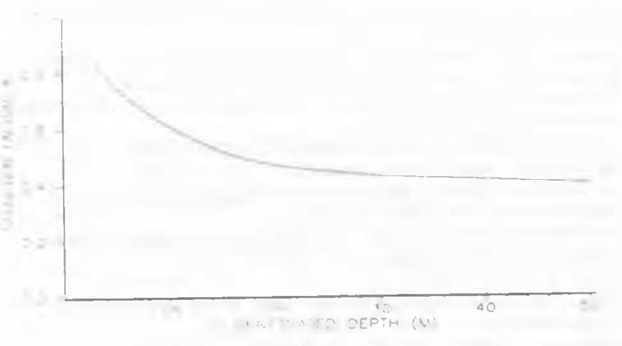

Figure 5. Degradation curve for clay.

This curve is in good agreement with the observations from Heather as reported by Heerema (1980) and Zandwijk et. al. (1983). A reduction in frictional resistance in clay is also reported by Semple and Gemeinhardt (1981). They assume that the reduction is instantaneous, utilizing no degradation concept. If however the degradation concept had been applied, the final reduction for their cases would have been higher and compare well with the observations from Oseberg.

By use of the unit resistances developed from above, the static resistance during driving, SRD, was post-predicted for the piles and the conductors. The results of the predictions together with the back-calculated ranges of SRD-values are shown in figure 6a and 6b. It can be seen that a good correspondance is obtained, using the described method.

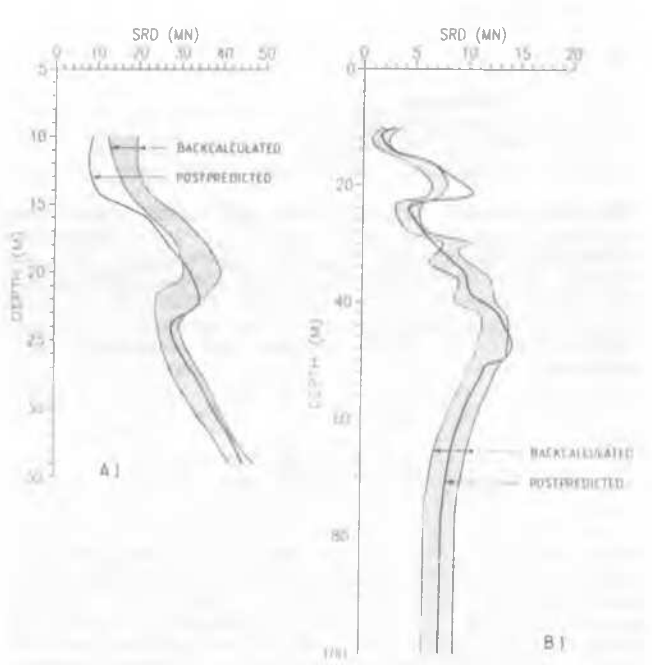

Figure 6. Postpredicted SRD of piles and conductors compared to backcalculated from blowcount curves.

It should be noted that the piles did not experience soil plugging even though the accumulated static inside friction exceeded the plugged tip capacity after API (1987) by a factor of 1.6. The conductors were known to have experienced plugging, and the best fit with the observations was found using a ratio of static inside friction to tip capacity of 2.5. These observations are in good agreement with other observations, and show that pile plugging during driving is a different phenomenon than pile plugging during static loading.

STATISTICAL EVALUATION OF UNCERTAINTIES

As shown in the blowcount curves presented, there is a considerable spread in the observations. This variability is due to several factors related to the soil, the pile and the driving equipment. However, at a given depth and for a certain pile and driving equipment, the two major factors contributing to blowcount variability are the static resistance during driving, SRD, and the input energy to the pile.

For the conductors, the variability of input energy in terms of standard deviation about the mean value is also evaluated as previously shown in figure 4a. The unknown variability of soil strength (SRD) can then be evaluated by the following approach:

In general we have at given depths for the conductors:

$$B = f(E, SRD) \quad (4)$$

where:

B = Blowcount
E = Input energy
SRD = Static resistance during driving
f = A general non-linear function

The blowcount, energy and SRD have their mean values μ_B, μ_E and μ_{SRD} , and associated standard deviations σ_B, σ_E and σ_{SRD}. At each depth, the function f can be linearized around the mean values of energy and SRD, and on basis of this linearized function we have the following expressions for mean values and standard deviations:

$$\mu_B = f(\mu_E, \mu_{SRD}) \quad (5)$$

$$\sigma_{SRD} = \frac{1}{\partial f/\partial SRD}\sqrt{\sigma_B^2 - (\frac{\partial f}{\partial E})^2 \cdot \sigma_E^2} \quad (6)$$

When the mean value of SRD is calculated from equation 5), the standard deviation on SRD can be evaluated at various depths when the standard deviations of energy and blowcount is known, by utilizing equation 6). This is carried out for the conductor observations at characteristic depths, and the coefficient of variation is calculated for each depth. In order to evaluate the isolated effect of energy variability, the above is also carried out assuming that the standard deviation of energy is equal to zero, and the average coefficients of variations found are:

$\Omega = 0.175$ (assuming no spread in energy)

$\Omega = 0.157$ (energy spread taken into account)

From the difference in the coefficient of variations it can be deducted that the energy variation only accounts for some 10-15% of the total uncertainty.

Similar evaluation is also performed for the pile driving observations. However, since the energy variability had not been evaluated, the standard deviation of energy was assumed zero.The average coefficient of variation found was:

$\Omega = 0.122$

The difference in the back-calculated soil variation from the piles and conductors may seem significant, but can be naturally explained by two effects: Firstly, the piles involve larger soil volumes and hence is less affected by local soil variations than the conductors, and secondly, a larger amount of the total resistance of the piles is taken by side friction, which also tends to smoothen local variations.

The quantification of soil strength variability is to the authors opinion important numbers that should be used as basis for determination of a likely upper bound resistance curve for design purpose. The fact that the uncertainty is related to the most commonly used distribution of variables, should provide an important basis for a probabilistic approach to driveability predictions in the future.

This approach was used for prediction of the drivability of the OSEBERG C piles. A factor was applied on the best estimate SRD values, which was calculated by the previously explained methods. It was a requirement that the upper bound SRD should be equal to or higher than the mean value plus two standard deviations. From the backcalculations, a coefficient of variation on SRD of 0,125 was assessed, corresponding to a factor of 1.25 on the mean SRD values.

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