Soil model for pile driveability predictions based on CPT interpretations

Modellisation des sols pour la prediction du battage des piles basee sur l'interpretation des CPT

T.Alm – *Aker Technology, Oslo, Norway*
L.Hamre – *NOTEBY A/S, Oslo, Norway*

ABSTRACT: Through the later years, the authors have been involved with pile driveability predictions for large, open-ended piles installed in different soil conditions in the North Sea, and a prediction model was presented in 1998. Since then, the database of back-calculated records have been continuously updated, and in particular, information from installations of long piles in normally consolidated clays has been gathered. Based on this complete database, a new and improved soil model is presented in the paper. This soil model is directly correlated to CPT measurements hence avoiding biased individual interpretation of measured data. Comparisons between the back calculated and the post predicted soil resistances are shown for a large number of locations as a verification of the new model.

RÉSUMÉ: Durant ces démières années les ingenieurs ont étès confrontées au probleme du battage des piles dans différents types de sol en mer du Nord. Un model pour prédire le comportement des piles durant le battage a étè présenté en 1998. Depuis, l'analyse des battages efféctués a donné une base nouvelle pour en affiner la prédiction. En particulier les informations de l'installation de longues piles dans un sol argileux normalement consolidé ont étè rassemblées. Basé sur cette masse de données un nouveau model de représentation des sols a étè développé et est présenté dans cet article. Ce model est directement corrolé au mesure CPT et par là évite toute interprétation individuelle des données. Le model a étè verifié en comparant la resistance des sols calculée avec celle des battages réellement effectué et cela pour un grand nombre de locations.

1 INTRODUCTION

In the continuous search for an optimum pile design in terms of minimum cost, the offshore industry is designing piles with increasing diameters and penetrations in order to arrive at maximum pile capacities. Documentation of achievable penetration is performed through driveability predictions, and models of good reliability is thus of vital importance. Fatigue damage due to driving will, for many cases, govern the pile design, and reliable models for driveability predictions are a requirement for realistic fatigue calculations as well.

Alm & Hamre (1998) have previously presented a model for driveability prediction based on the friction fatigue concept. In this concept, friction values were based on observed friction values from cone penetration tests (CPT), along with basic soil properties such as undrained shear strength and friction angles. The model was established based on back-calculations from a number of offshore pile installations in typical North Sea soils, and the model predicted generally well for most cases.

However, as additional data from later pile installations have been gathered, the statistical basis for the model has improved, and revealed possibilities for better correlation. In particular, driving records through deep deposits of normally consolidated soils showed that the previous method over-predicted the resistance to some extent.

During the process of updating the database, it became evident that individual interpretations of in particular undrained shear strength profiles were a significant source of variability in the predictions. It was therefore decided to correlate directly to measured cone resistances. By doing this, the variability in interpretation of input profiles was significantly reduced.

2 PILE DRIVEABILITY PREDICTIONS

A general description of pile-driving analyses by wave equations is given by Smith (1960), and further details of such analyses are given by Goble et.al. (1997) through the user manual of the GRLWEAP computer program. In general, the total resistance to driving is summed up by a static resistance part (SRD) and damping. The static resistance part is calculated in a similar manner as for the capacity of a friction pile, and the most common models for such calculations have been presented by Toolan & Fox (1976), Semple & Gemeinhardt, (1981), and Stevens et. al. (1982). These models produced generally prediction on the conservative side, in particular for long piles.

Heerema (1980) first introduced the friction fatigue concept, and corresponding prediction models have been presented (1979, 1980 and 1981), with updates by Zandwijk et. al. (1983). These models were however not easily available for common use in standard wave equation programs, and alternative friction fatigue concepts were presented by Alm et. al. (1989), Colliat et. al. (1993, 1996), and by Alm & Hamre (1998).

This paper presents an updated model for static resistance based on the friction fatigue concept. The damping part of the resistance is not covered here, as this is assumed to be sufficiently accurate modeled through the parameters previously presented by Alm & Hamre, (1998). The same required displacement to achieve frictional yield (i.e. quake) is also assumed as presented earlier.

3 THE DATABASE

The basis for this extended study includes data from 18 different installations performed at 16 different locations, and a summary of the key data with regards to pile and hammers are shown in Table 3 at the end of the paper.

The conductor and pile data for the Oseberg B jacket are at the same location, and also the Sleipner Riser Jacket and the Flare Tower is situated so close that identical soil conditions is used. This is also the case for the installation of the Ekofisk Bridge Support 6, which is close the Ekofisk 2/4 X jacket.

The total database includes installation data from a variety of soil conditions representing typical North Sea soils. Very dense top sands are found at Oseberg and Sleipner, and at Varg, a unique dense sand layer of approximately 35 meters thickness is encountered. Highly over-consolidated layers of clay with undrained shear strength values of 400 to 600 kN/m^2 are found at Sleipner B, Oseberg, Ekofisk, and Frøy, with the highest extremes of up to 800 kN/m^2 found at Heimdal. Significant clay layers with more moderate OCR's are included at Sleipner Riser /Flare and at Embla, while major layers of soft clays and silts are encountered at Oseberg Øst, Veslefrikk and Brage. The water depth at the different locations ranges between 70 and 170 meters.

The database consist of piles with diameters ranging from 72" to 108", but the installation data from long 30" conductors at the Oseberg location have also been included for comparison. The deepest pile penetration is 90 meters, except for the Oseberg conductors that were driven to 115 meters. A total number of approximately 150 support piles and 14 conductor piles have been individually evaluated.

All piles were driven with modern hydraulic underwater hammers with reliable performance, namely the Menck MHU 1000, 1700, 2100 and 3000, and the IHC Hydrohammer S-400 and S-2300. These hammers hit the pile top directly through an insert anvil, with no energy absorbing cushions present. All hammers have equipment for continuous impact energy monitoring, and individual energy profiles have been included for most of the piles. At 7 of the installations, the piles were driven directly on the pile top, while for the remaining, a short insert front follower was introduced at the last stage of driving.

4 BACK CALCULATIONS

The back-calculations are performed using the same procedure as previously described by the authors. In short, this involves the following steps:

- A general relation between SRD and blowcount is established for each individual set of pile and hammer configuration, using the computer program GRLWEAP.
- The relation between SRD and blowcount was established for different hammer energies, and a continuous and non-linear function relating SRD, blowcount and pile top energy was developed for each pile and hammer configuration. Hammer efficiency, i.e. the ratio between pile top energy and hammer impact energy, was taken as 0.85 for the MHU hammers and 0.95 for the Hydrohammers.
- For all piles where individual energy and blowcount profiles have been recorded, continuous SRD profiles were then back-calculated for each site using the functions established above.
- The average SRD profile, along with the statistical variations around the mean value was thus determined for each location, and used as the basis for the development of the updated SRD model based on the different soil conditions.

5 THE UPDATED SRD MODEL

The major contribution to SRD is due to side friction, and a realistic model for friction during driving is therefore of vital importance. The updated model will be based on the same principles as the previous one, utilizing the friction fatigue concept both in sand and clay.

The general formulation of side friction along a pile during driving will be:

$$f_s = f_{sres} + (f_{si} - f_{sres}) \cdot e^{k \cdot (d - p)} \qquad (1)$$

This formulation is the same as for the original model by Alm & Hamre (1998) and have thus the same advantages over other models as documented previously. The best fit have been found for the following relations:

For clays, the initial friction is taken as the recorded CPT sleeve friction, while the residual friction was found to be a function of the normalized cone tip resistance through the following formulation:

$$f_{sres} = 0.004 \cdot q_T (1 - 0.0025 \cdot q_T/p_o') \qquad (2)$$

With the above formulation, the residual friction is slightly reduced with increasing normalized cone resistance as visualized in figure 1.

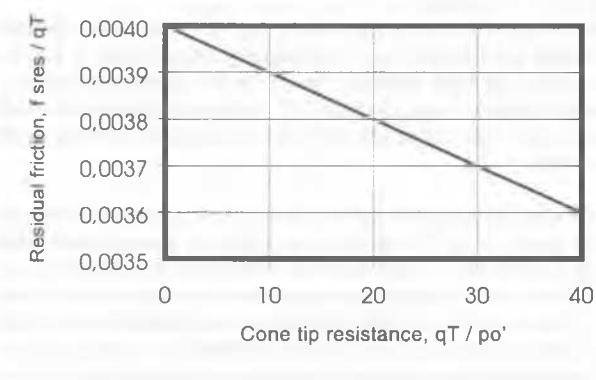


Figure 1. Residual clay friction ratio vs. cone resistance ratio

For sands, initial friction is taken as the basic static friction formulation, i.e.:

$$f_{si} = K \cdot p_o' \cdot \tan(\delta) \qquad (3)$$

No upper limit on unit friction is included. This is the same formulation as presented in the previous model, but the lateral stress coefficient K, is now directly linked to the cone resistance as presented by Jardine & Chow (1996), using the following formulation:

$$K \cdot p_o' = 0.0132 \cdot q_T \cdot (p_o' / p_a)^{0.13} \qquad (4)$$

The best fit for residual values was found for relatively low values, 20% of the initial friction.

However, when using the above relation for lateral stress coefficient, one must take into account that this formula has been established under the assumption that the friction occurs only on the outside of the pile wall. This assumption must therefore also be included when establishing the sand friction, either by including only the outside friction in the calculation, or more convenient, by reducing the unit friction to 50 % of the above, and applying on both inside and outside of the pile wall.

The shape factor for degradation has now been found to be properly described using the same formula, both for clays and sands using the following relation:

$$k = (q_T/p_o')^{0.5} / 80 \qquad (5)$$

With this formulation, a rapid degradation will occur for dense sands, while the opposite will be the case for soft clays. Examples of degradation curves are visualized in figure 2.

Unit tip resistance in clays are taken as 60 % of the total cone resistance, while for sands, a somewhat more advanced formulation was found to be required to obtain optimum correlation. This formulation was found as:

$$q_{TIP} = 0.15 \cdot q_T \cdot (q_T/p_o')^{0.2} \qquad (6)$$

With the above formulation, the unit tip resistance will increase with increasing sand density, and will be in the range of typically 0,35 to 0,55 times the cone resistance when sand density ranges from loose to very dense. These values are lower than what should be expected from bearing capacity relations, which should be the q_T value directly, with a reduction factor due to different shape factors for a circular cone and a pile steel annulus. The reason for this lower value is likely the effect of wedged pile tips where the actual tip area in practical design often is reduced to approximately 50 %. In the SRD predictions, the full pile tip area, not reduced for wedging, should be used in combination with the above relation.

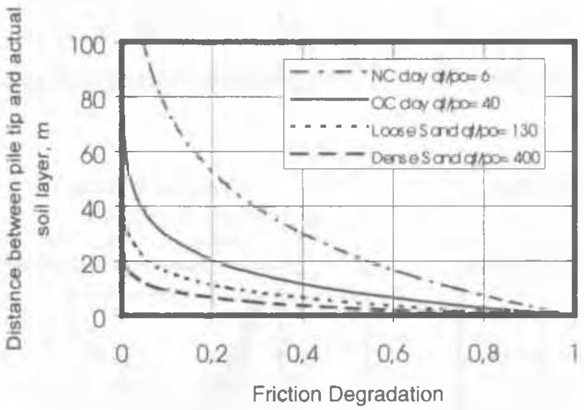


Figure 2. Examples of degradation shapes for different soils

The total static resistance is then calculated similarly to pile bearing capacity principles, and is contributed by pile tip resistance and side friction along the pile. For all cases, unplugged piles have been assumed during driving.

6 RESULTS OF POSTPREDICTIONS.

The results of the post-predictions are shown together with the back-calculated SRD profiles on figures 4 through 19 for each of the 18 different sets of installation data considered. These post-predictions are based on an interpreted average of a series of recorded CPT profiles at each site. In addition, a suggested upper bound resistance is included, taken as a 25% increase of the predicted best estimate.

In general it can be seen that the described model gives very good correlation to the back-calculated data. In average, the predicted best estimate curve lies very close to, or slightly above the back-calculated ones. Also, it can be seen that the upper bound curve generally covers the range of the back-calculations, with some exceptions. The majority of such exceptions are however related to resistances occurring immediately after stops in the driving sequence, which are due to hammer breakdowns or stops when followers are inserted. The following comments are made to the results where significant deviations to the model predictions seems to occur:

At the typical soft clay sites, like Brage, Veslefrikk and Oseberg Øst, the model is seen to predict very well. At Oseberg Øst, one pile is seen to lie above the upper bound model prediction. This is believed to be caused by the sand layer being locally more dense than used as basis. In this sand layer, it may also be seen that the driving resistances vary considerably.

At the clay sites with moderate over-consolidation, like Embla, and Sleipner Riser and Flare, and the upper 40 meters of Sleipner B, the model predicts generally very well. At Sleipner Riser and Flare however, the model slightly under-predicts the resistance from 25 to 50 meters. This is believed to be due to unreliable hammer performance for some piles, as reported for the Sleipner Riser installation. At Sleipner B, CPT data was scarce in the upper part, and the actual data showed relatively large discrepancies to the laboratory test results. It is believed that the CPT design profiles are too low, leading to the under-prediction shown.

Heavy over-consolidated soils with very dense clay layers are encountered at Oseberg B and Gass, and at Frøy, Huldra and Heimdal. For all these cases, the model predicts excellent, except for one pile at shallow depths at Frøy, and for two piles with very low resistance in one corner of the Heimdal Jacket. The discrepancies are at Frøy is likely to be caused by low hammer performance, while at Heimdal, it is likely that the soil conditions in this corner of the platform is different than used in the basis for the predictions.

For the Oseberg B conductors, the model does to some extent over-predict the resistance throughout the profile to 50 meters depth. This is however believed to be caused by the external driving shoe mounted at the conductor tip, leading to reduced outside friction. After driving to 50 meters penetration, the inside soil column was drilled out, and further drill-out using an under-reamer was performed to about 105 meters depth. In the model for prediction of SRD below 50 meters, initial friction values was assumed for all layers above 50 meters, and a gradual friction degradation was included according to the presented model.

At Varg and Jotun, thick layers of medium dense to dense sands are encountered, and for both locations, the model predicts very well. One may however observe that resistances generally vary more rapidly than in typical clay sites, which is due to the nature of variability in density normally found in sand layers. One pile at Varg shows however significantly higher resistance than any model would predict, and this anomaly can not be explained by locally high sand density. Hitting of a boulder or partial plugging due to other reasons may however be plausible explanations.

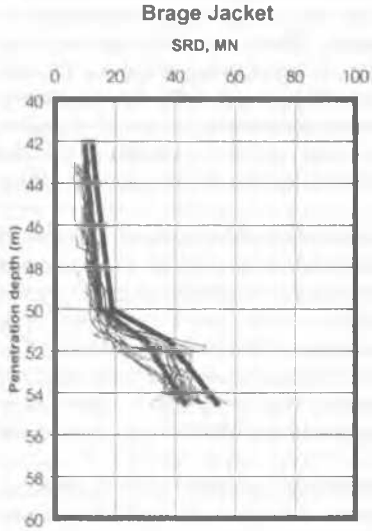
Mixed layers of over-consolidated clays and dense sands are encountered at Ekofisk and Oseberg Sør. Good correlation is found at both sites, but at Oseberg Sør, the variability is found to be high, leading to under-prediction in some depth ranges. This is caused by variable hammer performance that occurred during the first face of the pile driving.

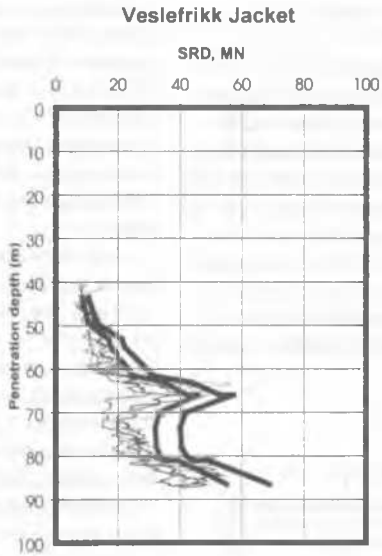
7 RECOMMENDATIONS FOR USE.

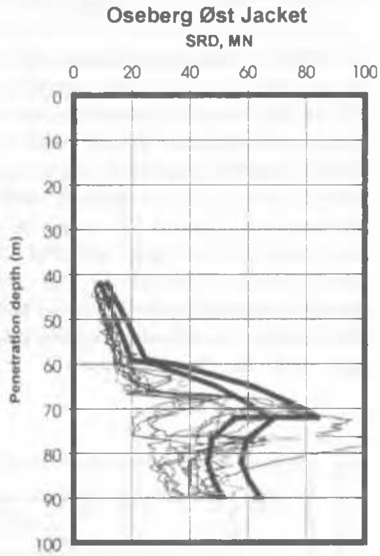
In summary, the model is shown to predict resistances lying close to or slightly above the average of the back-calculated values. Additional conservatism is also included since optimum hammer performance has been assumed, leading to maximum back-calculated resistances. The model will therefore produce best estimate resistance profiles lying slightly on the conservative side. The soil profiles to be used in predictions should be interpreted as characteristic profiles, i.e. average values on the conservative side.

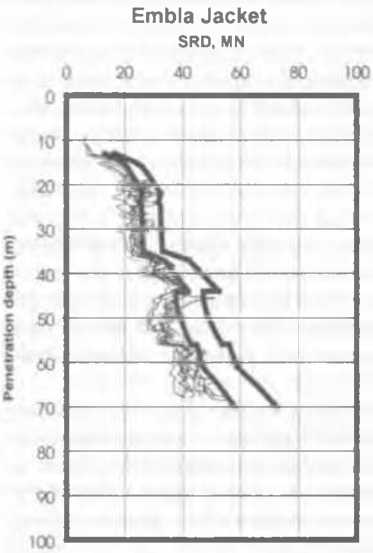
For prediction of the upper bound resistances, the effect of soil variability should be taken into account. Ideally, a factor representing the local variability of the actual layer should be established, but sufficient data to evaluate this are seldom available. Based on the back-calculations, a factor of 1.25 is suggested, which has been earlier shown to cover the variability for most cases within the average plus 2 standard deviations.

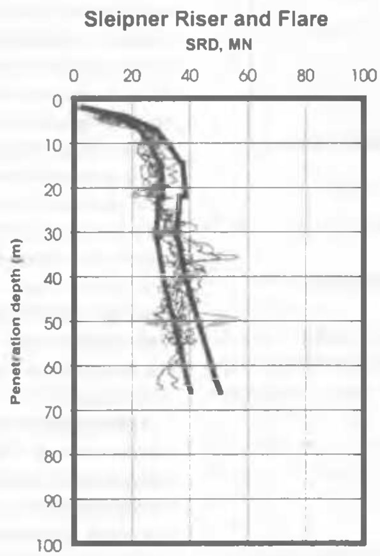
However, for some cases, and in particular for thick sand deposits at large depths, a higher factor may be found appropriate, and for critical cases, sensitivity studies with factors above 1.25 are thus recommended.

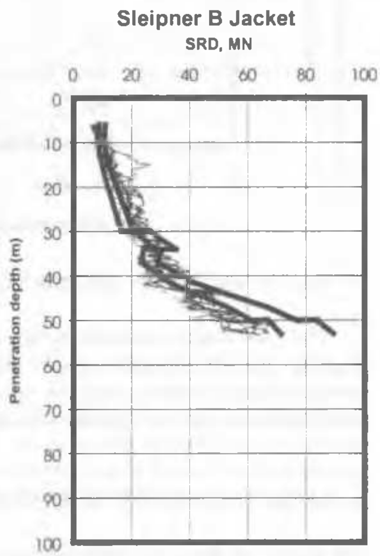


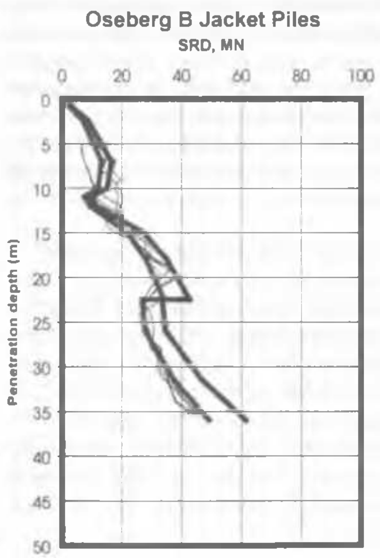


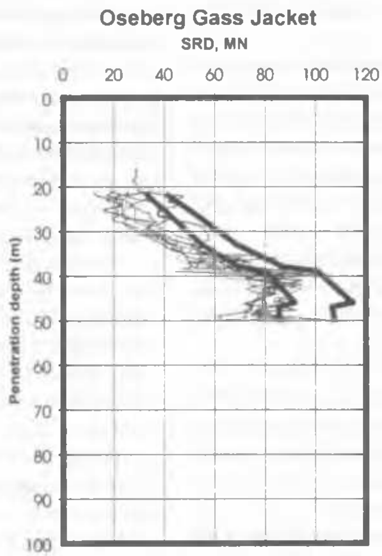




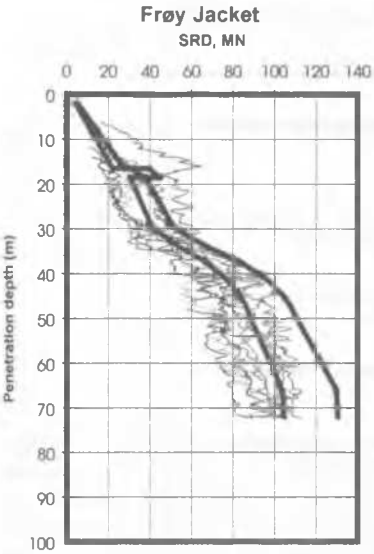


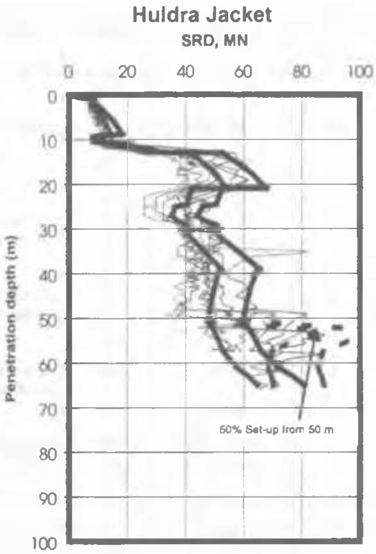


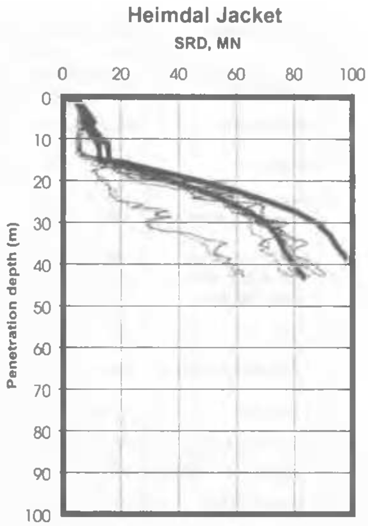


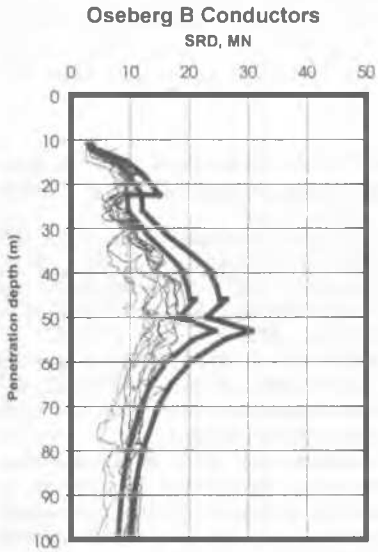


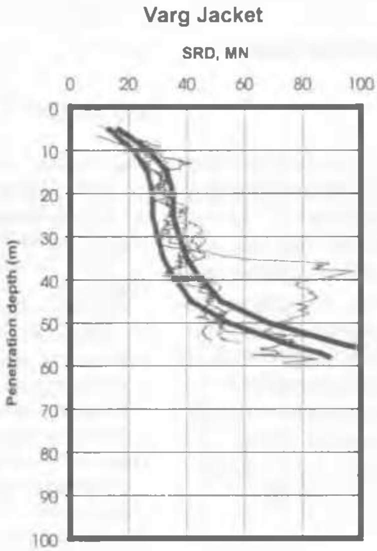
Figures 4 – 11 Results of back-calculations and post-predictions

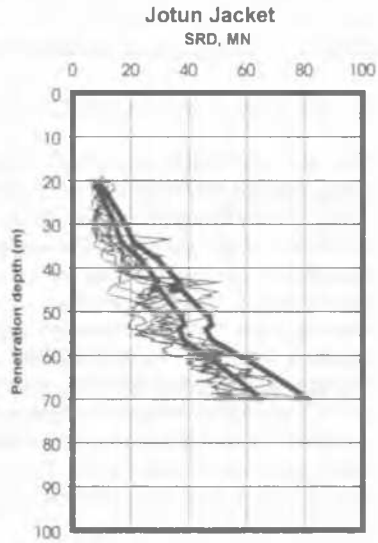


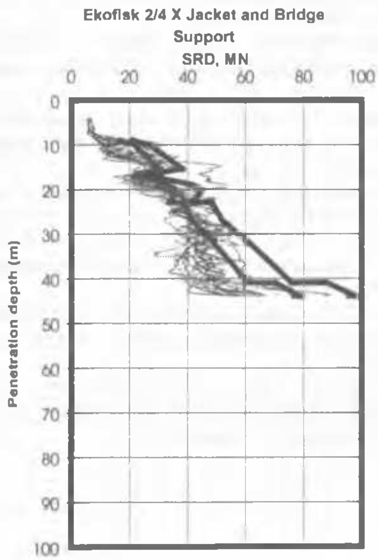


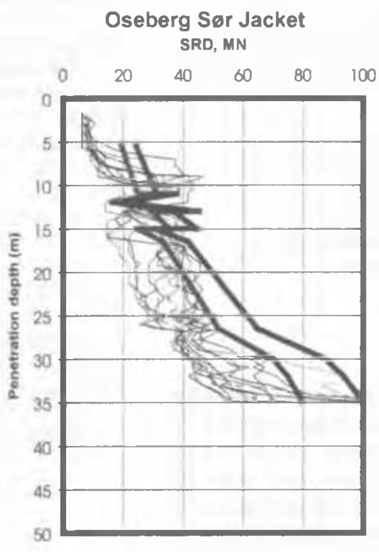












Figures 12 – 19 Results of back-calculations and post-predictions

Field / Structure	Inst. Year	Pile Dia. (inches)	Pile pen. (m)	Number of piles	Hammer Make	Follower	Comments
Veslefrikk Jacket	1989	84	87	16	MHU 1700	Yes	
Huldra Jacket	2000	96	57.5 / 66	8	MHU 3000	Yes	Set up due to installation of follower increases SRD
Brage Jacket	1993	96	54.5	24	MHU 2100	Yes	
Oseberg B Jacket	1987	84	35	32	MHU 1700	No	
Oseberg B Conductors	1987	30	115	-	S - 400	No	Driven to 50 m, drilled open with an underreamer. Set up and outside friction included.
Oseberg East Jacket	1998	96	90	12	MHU 1700/3000	Yes	
Oseberg Gas Jacket	1999	96	50	8	MHU 3000	Yes	
Oseberg Sør Jacket	2000	96	35	12	MHU 3000	Yes	Efficiency of hammer unstable. Reduction of 50% included for some piles.
Frøy Jacket	1995	96	72	8	MHU 1700/3000	No	
Heimdal Riser Platform	2000	84	36 / 43	8	S2300/MHU 2100	No	Low resistance for two piles. Possible varying soil conditions.
Jotun Jacket	1998	108	65 / 70	8	S-2300	Yes	
Sleipner Flare Jacket	1993	84	75	3	MHU 1700	Yes	Sleipner Flare and Riser combined in the back-calculations. Loose hammer ballast.
Sleipner Riser Jacket	1992	84	65.5	8	MHU 1700	Yes	
Sleipner B Jacket	1995	84	53	8	MHU 1700	No	
Varg Jacket	1997	96	54 / 59	4	MHU 3000	No	Possible large boulder.?
Ekofisk 2/4 X Jacket	1996	84	44	16	S - 2300	Yes	2/4 X Jacket and BS6 combined.
Ekofisk BS 6	1996	84	42	3	S - 2300	Yes	
Embla Jacket	1992	72	68	8	MHU 1000/1700	No	

Table 3 Data base of back-calculated North Sea pile installataion

8 SUMMARY AND CONCLUSIONS.

The new soil model developed is proven to give very reliable predictions of SRD profiles for a variety of typical North Sea Soils. The SRD profile can be used to predict blowcount curves by means of the wave equation method, provided that damping coefficients and quake values are included as recommended earlier by the authors. On average, the predicted resistances are slightly above the back calculated and thus slightly on the conservative side. The model, without additional factors, is thus considered appropriate for prediction of best estimate curves.

For prediction of upper bound resistance, the effect of soil variability should be taken into account. A factor of 1.25 is normally sufficient to cater for this. For critical cases, however, sensitivity studies with factors above 1.25 are recommended.

9 NOMENCLATURE

f_s = Pile side friction (kN/m^2)
f_{si} = Initial pile side friction, (kN/m^2)
f_{sr} = Residual pile side friction, (kN/m^2)
d = Depth to actual clay layer, (m)
p = Pile tip penetration, (m)
k = Shape factor for degradation, (-)
$q_{T\,IP}$ = Unit pile tip resistance, (kN/m^2)
q_T = Total cone tip resistance from CPT, (kN/m^2)
p_o' = Effective overburden pressure, (kN/m^2)
p_a = Reference pressure = 100 kN/m^2
δ = Constant volume friction angle (degrees)
K = Horizontal stress ratio after driving (-)

10 ACKNOWLEDGMENTS

The installation data and relevant soil information for the different locations included in this study have been released by the different Oil Companies operating at each location. The authors acknowledges permission to publish these data from Statoil as operator at Veslefrikk, Sleipner, and Huldra, Norsk Hydro Produksjon a.s as operator at Brage, Oseberg, Heimdal and Varg, Phillips Petroleum Company Norway, as operator at Ekofisk and Embla, and Elf petroleum Norge AS, as operator at Frøy.

REFERENCES

Alm, T., Bye, A & Kvalstad, T. 1989. New Interpretation of Soil Resistance for Pile Driveability Analysis. Proceedings of 12 th. ICSMFE, Rio de Janeiro.

Alm, T. & Hamre, L. 1998. Soil model for driveability predictions. Offshore Technology Conference, OTC, Houston, OTC paper no. 8835.

Colliat, J.L., Vergobbi, P. & Puech, A. 1993. Friction Degradation and Set-Up Effects in Hard Clays Offshore Congo and Angola. 25 th. Offshore Technology Conference, OTC, OTC paper no. 7192.

Colliat, J.L., Boisard, P., Puech, A. & Vergobbi, P. 1996. An Improved Pile Driveability Model for Soft and Hard Clays from Offshore Pile Driving Case Histories. Fifth International Conference on the Application of Stress-Wave theory to Piles. Orlando.

Goble, Rausche, Likins, & Associates, Inc. 1997. GRLWEAP - Wave Equation Analysis of Pile Driving. Program Manual and updates.

Heerema, E.P. 1979. Relationships between wall friction, displacement velocity and horizontal stress in clay and in sand, for pile driveability analysis. Ground Engineering.

Heerema, E.P. 1980. Predicting pile driveability: Heather as an illustration of the "friction fatigue" theory. Ground Engineering.

Heerema, E. P. 1981. Dynamic point resistance in sand and in clay, for pile driveability analysis. Ground Engineering.

Jardine, R. J. & Chow, F.C. 1996. New design methods for offshore piles, Report to Marine Technology Directorate, MTD, Publication no. 96/103.

Semple, R.M. & Gemeinhardt, J.P. 1981. Stress history approach to analysis of soil resistance to pile driving. Offshore Technology Conference. OTC paper no. 3969.

Smith, E.A.L. 1960. Pile Driving Analysis by the Wave Equation. Journal of the Soil Mechanics and Foundation Division, ASCE Vol.86.

Stevens, R.S., Wiltsie, E.A. & Turton, T.H. 1982. Evaluating pile driveability for hard clay, very dense sand and rock. Offshore Technology Conference. OTC paper no. 4205.

Toolan, F.E., and Fox, D.A. 1977. Geotechnical planning of piled foundations for offshore platforms. Proceedings, Institution of Civil Engineers, London, Part 1, 62.

Zandwijk, C. van, Dijk, B. P. van, Voeten, J.A.J. and Heerema, E.P. 1983. An improved pile driveability theory for Gulf of Mexico soils. Offshore Technology Conference. OTC paper no. 4503.