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# Driven piles –dynamic and PDA based estimates

## Piles pilotées - estimations dynamiques et basées sur des PDA

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**ABSTRACT:** The pile driving record is an essential monitoring tool for driven piles. At the same time, a dynamic pile driving equation allows for an immediate estimate of the total compressive resistance of the pile without further measurements or assumptions. In this respect, the Danish Pile Driving Equation (DPDE), despite its simplicity (energy transferred per blow, set per blow, EA and length of the pile), has proven to be a very good predictive tool. Pile Driving Analysis (PDA), based on the analysis of the stress wave travelling down and up the pile, allows for more detailed estimates of both shaft and toe resistance. However, this requires good insight into the soil profile and relies on signal matching with a low match quality factor in the analysis. Using data for a number of pile driving sites in Denmark and abroad for full displacement piles (typically precast concrete piles), and open-ended steel pipe piles (up to 3 m diameter) the PDA and DPDE results are compared. In view of the inherent uncertainties in geotechnical predictions, the agreement was very good, with a deviation typically of less than 10% between PDA and DPDE based resistances. The data also allow for inferences on the impact from the hammer size, set-up and potential “hard driving”.

**RÉSUMÉ:** Le relevé des pieux est un outil de surveillance essentiel pour les pieux battus. En même temps, une équation dynamique de battage de pieu permet une estimation immédiate de la résistance totale à la compression du pieu sans autre mesure ou hypothèse. À cet égard, l’équation danoise de battage de pieu (DPDE), malgré sa simplicité (énergie transférée par coup, jeu par coup, EA et longueur du pieu), s’est révélée être un très bon outil de prévision. L’analyse de l’enfoncement du pieu (PDA), basée sur l’analyse de l’onde de stress se propageant vers le haut et le bas du pieu, permet d’obtenir des estimations plus détaillées de la résistance de l’arbre et de l’orteil. Cependant, cela nécessite une bonne compréhension du profil du sol et repose sur une correspondance de signal avec un facteur de qualité de correspondance faible dans l'analyse. En utilisant des données pour un certain nombre de sites de battage de pieux au Danemark et à l'étranger pour des pieux à déplacement total (généralement des pieux en béton préfabriqué) et des pieux en tube d'acier ouverts (jusqu'à 3 m de diamètre), les résultats de PDA et de DPDE sont comparés. Compte tenu des incertitudes inhérentes aux prévisions géotechniques, l'accord était très bon, l'écart étant typiquement inférieur à 10% entre les résistances basées sur le PDA et le DPDE. Les données permettent également de tirer des conclusions sur l’impact de la taille du marteau, de son montage et de la conduite difficile» potentielle.

**Keywords:** pile driving equation; PDA, full displacement piles; open ended piles

## 1 PILE RESISTANCE, DRIVEN PILES

The pile driving record provides the initial insight in the variation with depth of axial resistance of piles installed by impact, diesel or hydraulic hammers.

A more direct estimate of the pile resistance is obtained using a dynamic pile driving equation, e.g. the Danish Pile Driving Equation (DPDE):

$$Q_d = \frac{\eta h G}{s+0.5 s_0}; \ \ s_0 = \sqrt{2\eta h G \frac{l_p}{A_p E_p}} \qquad (1)$$

where $\eta hG$ (kNm) is the transferred energy, $s$ (m) the set per blow, $E_pA_p$ (kN) the pile stiffness, and $l_p$ (m) the pile length.

Despite its simplicity Eq. (1) provides a very good estimate of the pile resistance. Based on COWI's experience with full displacement piles and tubular piles (with partial or full plugging) the ratio between PDA and Eq. (1) measurements is close to unity (from 0.95 to 1.25).

The high ratio is usually associated with "hard driving" where most of the energy is lost as elastic energy in the pile material at very small set values, $s \leq 0.05\ s_0$, i.e. close to refusal. Here, Eq. (1) typically underestimates the resistance by 15-20%.

A multitude of pile driving equations are available (Garland et al., 2012) but application requires diligence and site specific or well winnowed experience. EQ. (1) works for coarse grained soils, clay till and even for limestone.

However, accompanying PDA measurements and loading tests are required to make credible quantitative rather than qualitative predictions on pile capacity for any pile driving equation.

### 1.1 Pile driving as "exploratory tool"

For the Musandam piling (Steenfelt et al., 2010), initial pile driving of Ø1.422 m tubular steel piles with additional pile length was used as a "ground investigation tool" due to failure of drilling rig to provide the requisite exploratory boreholes. The general strata sequence was soft marine sand deposits, upper Calcarenite (caprock), limestone gravel in clay matrix, lower Calcarenite/Calsisiltite (caprock) and Limestone. The caprock and limestone deposits had unconfined compressive strength in excess of 6 MPa.

The dynamic pile capacity using Eq. (1) is shown in Figure 1 together with the interpreted soil strata. The top caprock layer (below a very thin layer of marine deposits) affords a very high resistance and a subsequent "shadow effect" after penetration. The lower caprock and the limestone provides very high resistance most likely due to a combination of plugging and high compressive strength in the limestone. The one pile penetrating about one metre into the limestone showed very hard driving (refusal), at the maximum hydro hammer energy of 145 kNm.

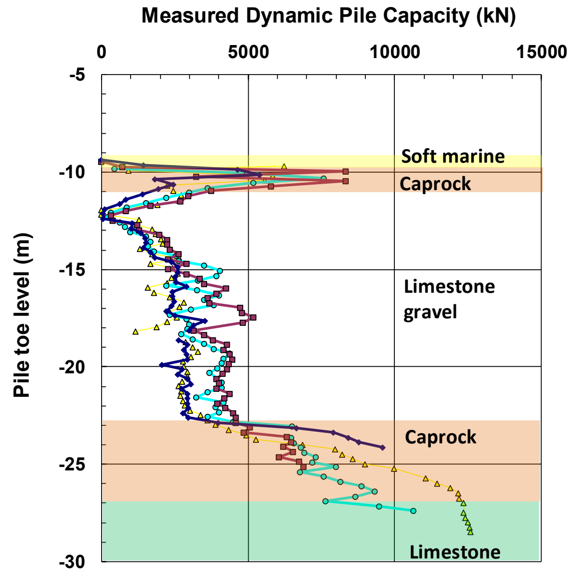


*Figure 1. Driving records using Eq. (1) for four "ground investigation piles" and interpreted strata*

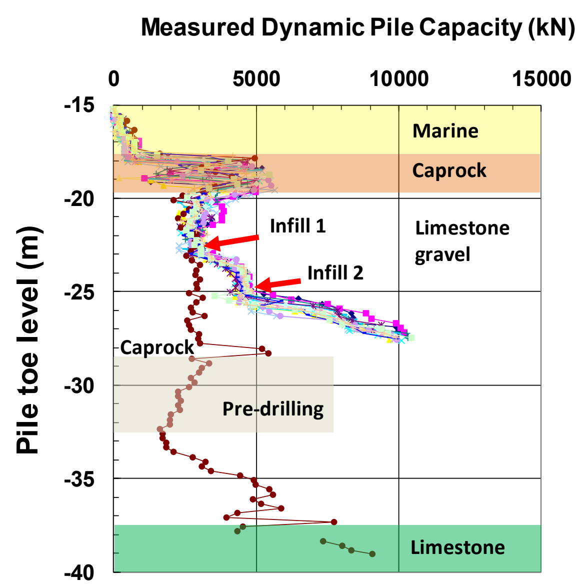


*Figure 2. Driving records for Loading Platform piles. Effect of pre-drilling versus gravel infill (plugging)*

For the loading platform at deeper seabed level pre-drilling in front of the pile toe was used to ensure penetration through the second caprock layer and to establish the limestone level as seen in Figure 2. Pre-drilling very significantly reduced the capacity until the toe level reached the pre-drilling level.

### 1.2 *Refusal criteria*

As described above hard driving in EQ. (1) is a function of energy, pile length and pile stiffness. This is roughly the same as standard practical limits for sustained driving using hydraulic hammers, $s$ ~ 2mm (Tomlinson and Woodward, 2008). The refusal criterion from AASHTO is 248 blows/250 mm for 1.5 consecutive meter. Thus, hard driving with $s \leq 1$ mm implies a risk of fatigue in the piles if associated with very high blow counts.

The remedy is application of a hammer with sufficient energy to avoid hard driving.

### 1.3 *PDA versus DPDE*

The obvious disadvantage of the DPDE is the failure to distinguish between shaft and toe resistance of the piles. The Pile Driving Analysis allows for this split albeit with rather sensitive parameter fitting and demands on the MQ (Match Quality parameter). Ideally, this should be between 1 and 3, but up to 5 may be acceptable.

For piles with no impedance variation along the pile (i.e. no change in diameter, wall thickness etc.) the agreement between PDA and loading test results are found to be very good. For 100 m long Ø1.5 m tubular steel piles driven through silty, very fine to medium sand the ratio of load test/PDA was found to be 0.93 ± 0.04. For Ø3.0 m piles (more than 120 m long) driven through the same deposit the agreement between PDA and DPDE was within ±5% in cases where hard driving was not experienced (hammer energy sufficient). However, when the set was s < 1 mm the ratio of DPDE/PDA dropped to 0.83. Using the popular modified Hiley equation the ratio Hiley/PDA was 1.75, i.e. gross overestimation.

These large diameter piles also demonstrate the difference between the dynamic resistance and the PDA measurements. The PDA provides a "frozen" picture of the pile after installation to the required depth, whereas the dynamic measurements describe the evolvement of the resistance as the pile progresses to the final toe level.

Thus, the upper parts of the pile/soil interface have experienced very excessive displacements (up to 120 m) and the dynamic effect of the driving involving up to 30,000 blows. This results in gradual reduction in shaft resistance until the level recorded by the PDA measurements.

This can easily be seen in Figure 3 and Figure 4, where the DPDE records and the accumulated PDA interpreted external shaft resistances, respectively, are compared for six Ø3.0 m tubular steel piles for the same bridge pier.

The piles were driven with a 2400 kJ hydraulic hammer using the full energy from app. level -100 m. From this level hard driving was experienced with set values at or slightly above $s$ = 1 mm.

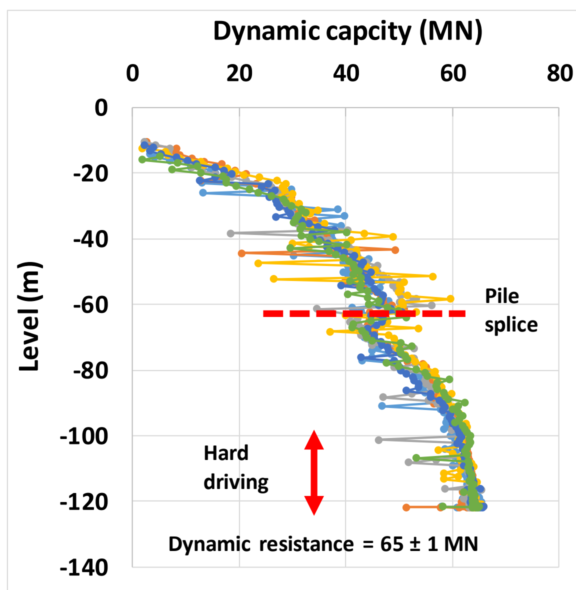


*Figure 3. Driving records using Eq. (1) for six Ø3.0 m tubular piles for the same bridge pier*

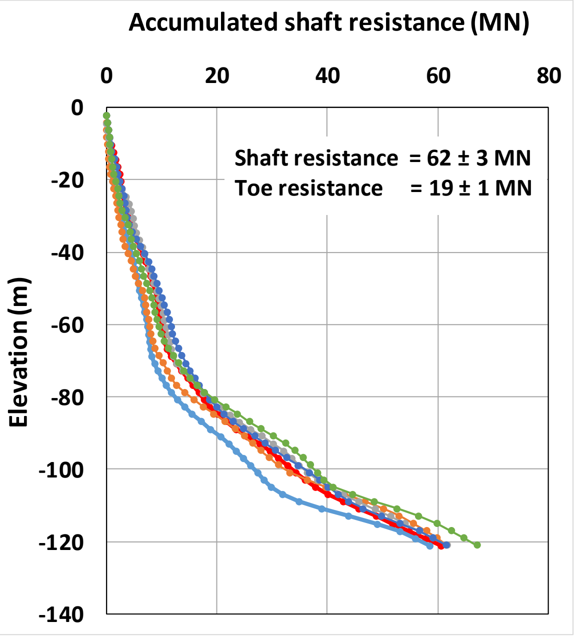


*Figure 4. Accumulated shaft resistance based on PDA measurements for piles in Figure 3*

The 126 m long piles were spliced when the toe of the bottom part (~71 m) was approximately at level -62 m. Before splicing a substantial amount of sand was removed to facilitate final driving.

The degradation in resistance at the top of the pile is obvious from the two graphs even taking the toe resistance into consideration.

## 2 FULL DISPLACEMENT PILES

In Denmark driven, prefabricated full displacement concrete piles are widely used. For a number of sites comparable data from pile driving (dynamic capacity), PDA and loading tests (unfortunately only to a lesser degree) are available.

To achieve very high stiffness of the foundation system, piles were driven through competent glacial deposits (mainly clay till) into limestone. These sites therefore allow an answer to a recurring question:

"Can prefabricated concrete piles be driven through competent clay till into limestone to achieve an axial design resistance more than 1000 kN without damaging the piles"?

The partial factor of safety is 1.95 and hence a dynamic capacity of more than ~2000 kN is required.

### *2.1 Project Sluseholmen, Copenhagen*

For this project 923 prefabricated concrete piles (300x300 mm) were installed late 2006. Dynamic capacity for 185 piles (full driving record for six test piles and partial record for 59 piles), PDA measurements on 7 piles and tension tests on four piles are available.

Fill is met from ground level at +1.85 to +2.0 m DVR90 underlain by clay till ($c_u$ ~ 300 kPa) from -1.0 m to -2.7 m DVR90 over København limestone ($\sigma_c$ ~ 2 MPa) from level -5.7 to -7.2 m DVR90.

The piles were driven with a Junttan hydraulic hammer with hammer weights from 40-63 kN.

The dynamic resistance in the fill is insignificant, almost constant in the clay till and increasing very sharply when entering the limestone. It was assumed that $Q_{dyn}$ > 1500 kN corresponded to the top fissured limestone (glacially disturbed) with $\sigma_c$ ~ 2 MPa. Refusal corresponds to $\sigma_c$ ~ 4 MPa, i.e. intact H2/H3 limestone.

As seen from Figure 5 a dynamic pile capacity well over $Q_d$ = 2000 kN can be obtained without damaging the piles. The seven test piles with PDA measurements showed $Q_d/Q_{PDA}$ = 0.95 ± 0.20. The relatively large scatter is largely due to the variable nature of the top of the limestone surface (level, strength and fissuring).

The ratio between shaft resistance based on geostatic calculation and PDA was 1.04 ± 0.27.

As several piles were tension piles (prevention of uplift of the basement floor) four piles were tested in tension (Figure 6). However, these piles had no additional reinforcement and hence the piles had lower capacity compared to the soil! The guarantied characteristic tension capacity of the piles was 397 kN at 100% mobilization of the yield stress.

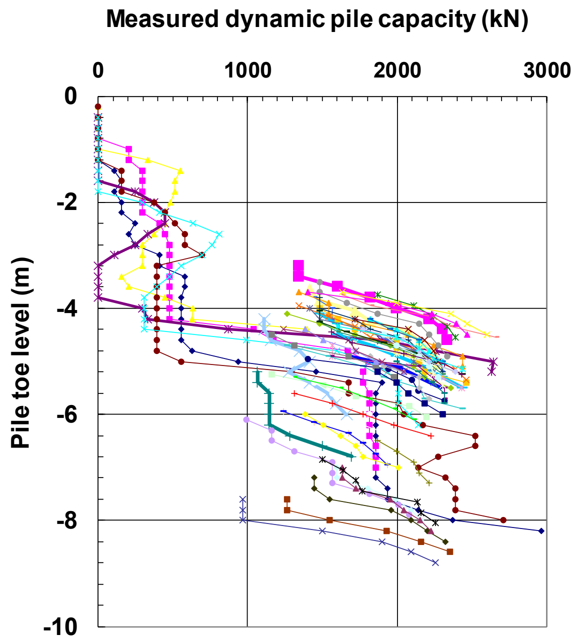


*Figure 5. Measured dynamic pile capacity for test piles and piles with partial driving record*

*Figure 6. Tension pile testing at Sluseholmen*

The average, reduced geostatically based shaft resistance (in compression) was 486/1.04 = 467 kN and the maximum required characteristic tension capacity was 439 kN. The maximum design resistance in tension was 225 kN compared with the 5% lower confidence limit for the four test piles of 238 kN.

Despite being the weak link, the test piles proved that sufficient tension capacity was available and that the 100% shaft resistance from PDA evaluation was relevant.

## 2.2 *Nordhavn, casting yard for the Øresund link tunnel elements*

For the sliding ramps and the production facility in Nordhavn, Copenhagen, 494 prefabricated, 400 x 400 mm$^2$ concrete piles were driven during 1995/96 (for the tunnel elements for the Øresund link project between Denmark and Sweden; Figure 7).

The generalized soil profile indicated: 8.4 m variable fill (over original seabed at level -6.1 m), 1.9 m mixed glacial deposits and 2.7 m glacial till overlying København limestone.

The top of the limestone was glacially disturbed but of hardness H2/H3 ($\sigma_c \sim$ 3-5 MPa) followed by considerable amounts of H4, H5 ($\sigma_c >$ 25 MPa) from level -13 m DVR90 (from -10.7 to -14.8 m).

*Figure 7. Piles for sliding ramp, Nordhavn*

Very strict demands on the settlements of the ramp rails required dynamic capacity > 4500 kN. With the available hammer weight of 60-80 kN and maximum fall height of 0.6-0.8 m it was only possible to achieve $Q_d$ ~ 3500-4000 kN for the 12-18 m long piles and only by applying "hard driving" with $s < 0.5$ mm! The maximum obtainable value for $s = 0$ was $Q_d = Q_0 = 4600$ kN demonstrating the incommensurability of dynamic demand and available hammer capacity.

In one area 66 piles failed to meet the requirement and 29 of these were either damaged or showed $Q_d \sim 3500$ kN. As a result, 148 piles were analysed in detail to provide a site-specific correlation between dynamic and PDA measurements. For 59 piles with PDA measurements the magnitude of set-up was investigated with a view to reduce the initial dynamic capacity demand.

The value of $Q_d/Q_{PDA} = 1.06 \pm 0.13$ showed a clear scatter for the 59 piles re-driven with time delays of 0 – 16 days, but the two sets of measurements were not statistically different.

The initial driving results were $Q_d \sim 3899 \pm 503$ kN whereas the re-driving showed an average of $Q_d \sim 4794 \pm 536$ kN with time delays of 0 to 33 days. Unfortunately, it was not possible to produce a non-ambiguous development of the pile set-up, but the re-driving produced significant increases in capacity.

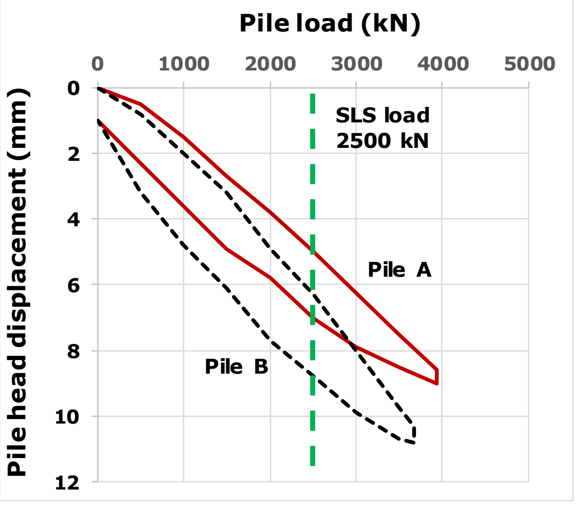


*Figure 8. Pile loading tests, Nordhavn*

Two static loading tests were carried out, but the load capacity only allowed for loads of 3930 and 3675 kN, respectively and hence the ultimate pile capacity could not be found. The almost linear relationship between load and displacement (see Figure 8) suggests an ultimate capacity >>4500 kN. The pile head displacement was only 5.0 mm and 6.25 mm at the SLS load of 2500 kN, almost completely recoverable.

As the decisive condition for the piling was in fact the SLS condition, limited (differential) settlements, the demand on minimum dynamic capacity (ULS) was not governing in practice. The combined results of loading tests, PDA and dynamic measurements allowed a reduction to initial dynamic capacity of 3500 kN, and no piles were subsequently damaged during driving.

## *2.3 Lindholm precast yard, Storebælt WB*

A large number of prefabricated concrete piles were driven to support various structures at the pre-fabrication yard for the West Bridge of the Storebælt link project (see Figure 9).

The area of the yard was in part on reclaimed land over substantial amounts of weak postglacial/late glacial deposits of sand, clay and gyttja (3-5 m but occasionally up to 15 m).

*Figure 9. Lindholm pre-fabrication yard for the West Bridge deck and pier elements*

Below these deposits 2 to 5 m thick layers of very competent clay till, from level -4.0 m to -6.0 m, was underlain by Danian limestone. The top of the limestone was found at level -8.0 m to -10 m.

The upper 5 to 8 metres of limestone was unindurated (H1) but exhibited increasing induration with depth.

Presumably due to the unindurated limestone in the top few metres of the limestone deposit, a very pronounced increase in pile capacity was observed within hours between initial driving (Eq. (1), DPDE) and re-driving (PDA). The piles were driven from ground level at +1.8 m using a follower to have final pile head a maximum of 1.6 m below ground level. The pile toe level was typically between -17 m and -21 m but at the south-western part, where the limestone surface was deeper, to levels from -32 to -37 m.

The onshore pile length varied slightly due to the depth of the limestone:18-20 m for 250x250 $mm^2$ piles, 18 m for 300x300 $mm^2$ piles and 20-24 m for 400x400 $mm^2$ piles. For the longer, jointed piles (offshore and 400x400 mm piles in general) ABB spigot and socket joints were applied.

The piles were driven using a semi-hydraulic "Uddcomb" hammer (80 kN with 1.0 m fall height) with 153.5 kNm energy for offshore works (1140 piles; pile lengths 21-37 m) and a Hitachi/Banut 60/50 kN hammer for onshore works (3840 piles; pile lengths 18-24 m).

Based on the general Danish experience the initial capacity from driving was assessed by Eq. (1) from the final 3 m driving records. For all test piles and 9.2% of the production piles full readings of driving resistance were recorded (blow counts and PDA measurements). The test piles were furthermore re-driven, and load tested at selected time intervals.

Static loading tests were carried out for both 400 mm (16 tests) and 250 mm piles (8 tests). The piles were driven in pairs with ~2 and ~4 m penetration into hard limestone, respectively.

The maximum pile loads were 3850 kN and 2300 kN, respectively, but alas also for this site the load arrangement did not allow assessment of the failure load.

A very significant gain in capacity, a factor 2.24±0.65, was observed within 1 hour of initial driving. The gain was 2.40±0.97 after 1 day, 2.91±0.98 after 2-9 days and 2.70±1.04 after 13-32 days. Only 4 of the 24 piles with loading tests was carried to near failure. For these the factor was 3.88±1.20 after 8-20 days. Even without reaching failure the 24 loading tests showed an increase of 2.37±1.20 (8-29 days after initial driving).

The overall increase in capacity 20+ days after initial driving was 200% - 300% resulting in dynamic capacities $Q_d$ ~ 4000 kN (3000 to 4500 kN) for 400x400 mm piles and ~ 2400 kN (1640 to 3800 kN) for 250x250 mm piles. Despite the achieved very high capacities only 114 broken piles had to be replaced (out of a total of 4980 piles), i.e. 2.2%.

The case history shows that it is possible to drive prefabricated concrete piles to very high dynamic capacities in limestone and that a significant gain takes place within a very short period after driving. The relatively thick layer of unindurated limestone is likely the main reason for the very dramatic increase in capacity within hours of driving.

## 3 TAGMARKEN WINDFARM, THY

The site for six onshore wind turbines in Thy, Denmark (Figure 10), showed significant deposits of gyttja (organic) and post glacial clays (12 to 31 m), followed by glacial clay till/sand (1-8 m) over limestone.

Thus, the foundation solution chosen involved 44 to 67 driven, precast 300x300 mm concrete piles for each foundation (22 to 40 m long).

*Figure 10. Wind turbines, Tagmarken, Denmark*

The design capacity of the piles was based on measured dynamic resistance (Eq. (1), DPDE) during initial driving and the average gain shown by subsequent PDA measurements after 1 and 8 days on pile No. 1, 13, 24 and 35 for each turbine position as shown in Table 1.

Due to the variable ground conditions the gain in capacity varies considerably, but for all piles a logarithmic increase in characteristic capacity with time after installation is observed.

*Table 1. Average measured capacity (standard deviation), gain factors and extrapolated capacity for test piles*

| Turbine/pile No; | length (m) | Initial* measured (kN) | Factor after days 1 | 8 | Design* value (kN) |
|---|---|---|---|---|---|
| 2 | 22 | 780 (184) | 1.70 | 2.28 | 1779 |
| 3 | 26 | 671 (49) | 1.94 | 2.29 | 1537 |
| 4 | 22 | 915 (163) | 2.37 | | 2096 |
| 5 | 33 | 1705 (260) | 1.28 | | 2182 |
| 6 | 40 | 511 ( 79) | 4.85 | 6.01 | 3070 |
| 7 | 30 | 1752 (109) | 1.23 | | 2155 |

** Characteristic values*

All piles exhibited characteristic capacity well over 2000 kN after 8-20 days except for piles 2-1 and 2-35 where this value may only be obtained after more than 40-50 days.

The data prove that high capacity can be obtained for precast concrete piles driven through glacial deposits and into limestone. The gain with time after installation is partly due to gain in the glacial clay and partly due to gain in the top softer limestone deposits.

## 4 CONCLUSION

Based on experience from open and close ended piles the case histories demonstrate very good agreement (within ± 10%) between the Danish Pile Driving Equation (DPDE) and PDA derived pile capacities. However, for hard driving the DPDE underestimates the capacity, due to the high elastic energy loss, by up to 20-25%.

Re-driving at different times after initial installation allows pile set-up to be evaluated and serves as an aid to avoid initial hard driving. Site specific calibration of DPDE with PDA where no prior well winnowed experience exists is a prerequisite for speedy evaluation of pile capacity using the DPDE.

The case histories for piles driven into limestone show that it is possible to obtain very high axial capacities without pile damage if pile set-up is taken into consideration and sufficient hammer energy is provided to avoid hard driving.

## 5 REFERENCES

Garland, L., Fellenius, B.H., Holtz, R.D. (2012). Pile driving formulas. *Pile Driver, Q2, Vol. 2*, 5 pp. Official publication of the Pile Driving Contractors' Association.

Steenfelt, J.S., Albert, G., Sørensen, C. 2010. Innovative plugging method for driven piles at Musandam Rock Loading Facility, Oman. *Proc. Second International Conference on Coastal Zone Engineering and Management (Arabian Coast 2010),* November 1-3, 2010, Muscat, Oman, 10 pp. (ISSN 2219-3596).

Tomlinson, M., Woodward, J. (2008). *Pile design and construction practice, 5th ed.,* Taylor & Francis, Abingdon, UK.