5th Fatigue Design Conference, Fatigue Design 2013

Pile fatigue assessment during driving

Jean Chung, Régis Wallerand, Morgane Hélias-Brault
Subsea 7, 1 Quai Marcel Dassault, 92156 Suresnes Cedex, France

Abstract
The aim of this paper is to present the methodology currently followed in the offshore industry for the prediction of the pile fatigue induced by pile hammering during installation. A few decades ago piles were thicker, of lower yield stress and hammers were less efficient. Today, to be cost effective, it is common to use high strength steel resulting in lower pile thickness and increased driving stress together with better hammer efficiency to reduce installation time. Hence fatigue damages are more important. Evaluation of fatigue damages depend on various parameters such as applied stresses, number of blows, Stress Concentration Factor (SCF) and S-N curves. It is shown how fatigue damages are dependant on the choice of the values used for the SCF and S-N curves and also on applied stresses by the hammer, and in the end the impact of these parameters on the residual available fatigue life for the in-place conditions.

© 2013 The Authors. Published by Elsevier Ltd. Open access under CC BY-NC-ND license.
Selection and peer-review under responsibility of CETIM
Pile; Driving; Fatigue

1. Introduction

Offshore platforms are subjected to harsh environment and variable loads such as waves, currents and winds. When designing a platform supported by a substructure called jacket, fatigue analysis is performed for all tubular nodes and the foundation piles for in-place conditions.

The common way to install the piles is by driving with an impact hammer. The piles are generally subjected to high dynamic loads close to the steel yield stress. The pile fatigue damage is defined during driving for all the butt welds of the pile either done offshore for the assembly of pile sections or in the yard for the prefabrication of the pile sections.

The available fatigue life for in-place conditions is thus reduced due to initial fatigue damages for pile installation during driving. Piles welds cannot be inspected and/or repaired as they are embedded in soil or inserted in jacket
legs. Hence safety factor for fatigue damages requested by Total, Shell or Exxon Mobil is equal to 10. Usual service life of an offshore structure is 20 years which means a fatigue life of more than 200 years. Moreover, in some cases the structures are built in yards which are located very far from the field with transport duration about a month. Hence fatigue damages due to transportation loads have also to be accounted for in the determination of the residual fatigue life for in-place conditions.

The input to the fatigue analysis comes mainly from the pile driving assessment. A description of the most common method followed to assess the Soil Resistance to Driving (SRD), and the way the pile driveability is derived will be made. The pile driveability provides the range of stresses experienced by the piles during the driving process and the number of impacts required to drive the piles to the required penetration. A good pile driveability analysis is thus crucial to a good fatigue assessment, and data from the field are most wanted to guarantee improved predictions.

2. Characteristics of offshore piles

The most common type of offshore pile is a pipe pile, which is typically driven into the seabed by a hammer. Common pile outer diameter (OD) ranges from 30" for tripods to 84" for big 8 legs platforms. Pile wall thickness starts from 1". Embedded length in seabed ranges from 80m to 100m for Gulf of Guinea platforms for instance. Pile make-up consists of the detailed design of wall thickness and other features of pile sections. There may be a driving shoe at the lower end, tapered for easier driving, and with a thicker wall section to cater for stress concentrations and non-uniformities in the soils and rocks encountered. The central sections may have a thinner wall because they will support less loads during the operational phase of the platform design life. The upper sections and the sections at mud line level may have a thicker wall because of the larger stresses they see during operations.

3. Pile driving offshore

The principal methods of installing an offshore driven pile are described by Toolan and Fox (1977). Figure 1 illustrates the driving operation for a pile that is installed through a jacket leg. With the jacket supported by mud-mats, the first few sections of pile are lowered to the seabed and allowed to break through a seal, if present. A hammer is installed on the stack, and is used to drive a segment of the pile into the seabed. When one segment has been driven to the limit of travel, the piling equipment is lifted off and about 1 m of top of pile is cut off. Another pile segment is lifted on and welded in place. The girth weld is normally subjected to non-destructive testing (Magnetic Particle Inspection and Ultrasonic Testing), after which the piling equipment is lifted back on. The operation is repeated until the required pile penetration below the seafloor has been achieved. The hammer is then removed, and shims are welded in the annulus between the pile and the leg.

![Figure 1 – Offshore pile installation sequence](image-url)
4. Effect of a hammer blow

Figure 2 shows typical elements of an offshore hammer. The hammer has a driving system, drop weight or ram, anvil, one or more cushions, and a temporary pile cap or cage or helmet. The ram maybe guided as a piston in a chamber, or by a central rod. The working fluid may be air, steam, oil, or a diesel—air mixture. The driving system may be an external generator of fluid under pressure (external combustion systems), or may be integral with the ram chamber (diesel hammers).

In single-acting hammers, high pressure is used to drive the ram upwards in each stroke, with gravity used to drop the ram onto the anvil. In double-acting hammers, high pressure is used to move the ram in both directions. The driving system lifts the ram and drops it onto an anvil or striker plate, or onto a cushion or cap block. The cushion absorbs some of the damaging high-frequency components of the blow, and helps to spread the stress evenly across the width of the element beneath it.

When the hammer hits the anvil, stress is transmitted through the cushioning systems into the top of the pile. The top of the pile moves downwards, and a compressive stress—strain wave starts to travel down the pile.

The wave travels at the speed of sound in steel, about 5100 m/s. Energy is transmitted into the ground through frictional slip at the soil—pile interfaces. This causes a shear wave to travel outwards from the pile into the soil. The consequent loss of energy reduces the energy of the stress wave travelling down the pile, an effect that can be modeled as radiation damping.

A partial reflection occurs when the wave reaches the seafloor, although this is minor except for a very stiff seafloor. Partial reflections also occur whenever the stress wave reaches a boundary between soil layers with different stiffnesses, and at imperfections and changes in the section properties of the pile. This can be advantageous because equipment can be installed to measure and analyze the reflected wave received at the pile head. This allows problems to be identified early.

The information can also be used to assess the ultimate axial capacity of the pile (Likins et al., 2008). When the compressive stress wave reaches the pile tip the pile shoe moves rapidly into the ground, transmitting more energy into the ground. The ground does not fully spring back: instead, there is some permanent set there. This consumes some more energy. The remaining energy is then reflected and travels upwards into the pile, sometimes as a tensile
wave (sign of stress and displacement in opposition with the compression wave) which produces some elastic rebound in the upper parts of the pile. The overall elastic rebound may be fully dissipated by soil—pile friction, before the wave reaches the pile top.

One result of these events is a permanent set at the top of the pile. The set can be deduced from the blow-count, typically expressed in blows per foot or quarter of a meter. Easy driving corresponds to blow-counts of around 10 blows/foot. Hard driving is above 50 blows/foot. Refusal is generally defined in the contract, and is taken to have occurred if the blow-count reaches 200-300 blows/foot.

5. Pile drivability methodology

In a pile drivability study, upper and lower bound predictions are made of the numbers of hammer blows per foot of penetration needed to drive the pile into the seabed, and of the maximum compressive and tensile stresses induced in the pile during driving. Predictions depend on the characteristics of the hammer and the associated equipment, the pile dimensions, the soil properties, and how far the pile has penetrated into the seabed. The following strategy is usually used:

(a) For a given pile diameter and wall, estimate the upper and lower bounds of the soil resistance to driving. The SRD is extracted from the ultimate axial pile capacity that would occur under static loading conditions. The estimates are plotted as a graph of the SRD versus pile tip penetration into the seabed.

(b) For a given pile-driving hammer and required final pile tip penetration, calculate the relation between the SRD and hammer blows required to drive the pile per foot or per quarter of a meter of movement. This is done using a wave equation analysis described below. It gives a ‘bearing graph’. Upper and lower bound graphs may be needed, but a single graph can be used if the bounds are close.

(c) For each pile penetration and bound, the SRD is read from the SRD profile, the corresponding blow-count is determined from the relevant (upper or lower bound) bearing graph, and the result is plotted on the blow-count—penetration graph.

As well as blow-count data, maximum compressive and tensile stresses will be calculated in step (b). The pile designer will estimate the fatigue damage caused to the pile by combining this information with results from step (c).

5.1. Calculations for the SRD

The SRD is the ultimate axial pile capacity that is experienced during the dynamic conditions of pile driving. Predictions of the SRD are usually calculated by modifying the calculation for the ultimate static axial pile capacity in compression. API RP 2A and ISO 19002 refer to several methods proposed in the literature.

The recommendations of Stevens et al. (1982) are widely used. The recommendations were based on 58 case histories of installations of large-diameter pipe piles at 15 sites in the Gulf of Mexico. The case histories were analyzed using a coefficient of lateral earth pressure (ratio of horizontal to vertical normal effective stress) K=0.7 for sand (API RP 2A now uses 0.8), and with the 1981 API RP 2A method for clay. This is now superseded (Randolph and Murphy, 1985), but is given in the API commentary. Some alternatives for hard soils are proposed by Colliat et al. (1993). The approach for carbonate sands is very different now (Kolk, 2000; Rausche and Hussein, 2000).

A designer using Stevens et al. is recommended to start from the 1981 API RP 2A calculation to evaluate the ultimate axial pile capacity. This code provides guidance to calculate shaft friction f and end bearing q in order to evaluate the ultimate bearing capacity Q_d:

\[ Q_d = Q_f + Q_p = f A_s + q A_p \]  

With:

- \( Q_f \) = skin friction resistance,
- \( Q_p \) = total end bearing,
- \( f \) = unit skin friction capacity (see section 6.4 on pile capacity for axial bearing loads of API RP 2A for more details on f calculation),
\( A_s = \) side surface area of pile, 
\( q = \) unit end bearing capacity (see section 6.4 on pile capacity for axial bearing loads of API RP 2A for more details on \( q \) calculation), 
\( A_p = \) gross end area of pile.

This is then modified, to obtain four curves of the SRD versus depth:
- upper bound, pile assumed plugged,
- upper bound, pile assumed coring,
- lower bound, pile assumed plugged,
- lower bound, pile assumed coring.

A plugged behavior is assumed when the soil plug inside the pile moves with the pile during driving, a coring behavior is assumed when there is a relative movement between the pile and the soil both on the outside and outside wall of the pile. These considerations modify the surfaces \( A_s \) and \( A_p \) considered in equation (1).

The lower bounds are estimates for the case of continuous driving. The upper bounds may go some way towards accounting for set-up and for uncertainties in soils data or hammer performance.

The modified curves are determined using soil properties determined from site investigation data in a way that would give a reasonable upper bound on static capacity, rather than a reasonable lower bound that is used in a capacity calculation.

5.2. The one-dimensional wave equation

A wave equation analysis is a calculation that takes account of the dynamic response of the pile and soil during driving.

The relationship between SRD and blow count for a given hammer/pile/soil combination can be derived using commercially available pile driving software package such as GRLWEAP (PDI, 2010), TNOWAVE, and others, which uses the wave equation approach prepared by Smith (1960).

In Smith approach a one dimensional formulation for longitudinal wave transmission due to end impact is followed. To simplify, it is assumed that the input energy of the driving equipment (minus the loss in hammer mechanism) is equal to pile resistance (including soil tip and side resistance and pile stiffness) multiplied by its movement through the soil or permanent set. The hammer and pile system is discretized into a number of distinct elements: masses and springs. Hammer system is considered as masses and springs acting on top of the pile and the pile is divided into a series of masses connected by weightless springs to represent the pile stiffness. It is found to be important to model the several distinct components of the hammer system as the ram, cap block, pile cap and cushion. Springs and dashpots are used to represent the frictional resistance of the soil and the point resistance from the soil below the pile toe.

Based on the method described above, available software packages can simulate motions and forces in the foundation pile when driven by an impact hammer and computes the following outputs:
- The blow count of a pile under one or more assumed ultimate resistance values and other dynamic resistance parameters for soil (depending on the pile penetration), given a particular hammer and driving system,
- The axial stresses in the pile, averaged over its cross section associated with the assumed capacity value(s),
- The energy transferred by the hammer to the pile for each capacity analyzed.

From these results, the following can be indirectly determined:
- The pile(s) bearing capacity at the time of driving or re-striking, given its observed penetration resistance (blow count),
- The stresses during driving,
- A suitable hammer for driving a given pile in a given set of soil conditions.
5.3. Pile driving fatigue

The practice presented here refers to DNV-RP-C203. Justified deviations have been sought and obtained in recent projects. Fatigue damage due to pile driving is calculated using the methodology presented here above and extracting the results obtained with GRLWEAP software.

The basic methodology includes:
- Using GRLWEAP outputs, calculate blow counts versus depth and associated stress range,
- Appropriate Stress Concentration Factors (SCF) and S-N curves are selected,
- Using the number of blows and stress range data, driving damage per depth increment is calculated.
  Palmgren-Miner rule is then applied to sum the incremental damage and obtain the total damage due to driving.

Cumulative fatigue damage $D$ of the piles is evaluated with Palmgren-Miner rule:

$$D = \sum \frac{n_i}{N_i}$$

With:
- $N_i =$ number of cycles at failure for a given stress ratio variation $\Delta \sigma_i$ (see section 5.3.2 for more details),
- $n_i =$ number of cycles inflicted to the structure for the $\Delta \sigma_i$ stress variation.

It should be noted that the stress ratio variation $\Delta \sigma_i$ is not constant with time as during pile driving the energy of the hammer is adapted to soil resistance.

5.3.1. Stress concentration factor

DNV-RP-C203 provides guidance on fatigue design of offshore steel structures and more precisely on the calculation of the stress concentration factors to be adopted in fatigue analysis.

For piles with wall thickness transitions, which can be seen as tubular butt weld connections (see figure below), DNV-RP-C203 (revision 2000 used for the following project described) recommends the SCF to be calculated using equation (3) below:

$$SCF = 1 + \left\{ \frac{6(\delta_e + \delta_m)}{t} \right\} \frac{1}{1 + (T/t)^{2.5}} e^\alpha$$

Where:

$$\alpha = \frac{1.82L}{\sqrt{Dt}} \frac{1}{1 + [T/t]^{2.5}}$$

$\delta_e =$ eccentricity due to concentricity, out of roundness and centre of eccentricity; eccentricity due to out of roundness giving normally the largest contribution to eccentricity,
$\delta_m =$ 0.5 (T-t) eccentricity due to a change in thickness,
$T =$ thickness of thicker plate,
$D =$ nominal diameter of tubular connection,
$t =$ thickness of thinner plate,
$L =$ length over which the eccentricity is distributed.
5.3.2. S-N Curves

S-N curves are used to determine the fatigue life of a structural steel joint (see figure below). The relevant S-N curve is a function of the weld type used to join the pile sections.

The form of the S-N curve and the number of cycles at failure for a given stress ratio variation is given by:

\[
\log(N) = \log(\bar{\alpha}) - m \log \left( \Delta \sigma \left( \frac{t}{t_{\text{ref}}} \right)^k \right)
\]  

(5)

Where:

- \(N\) = predicted number of cycles to failure for stress range \(\Delta \sigma\),
- \(\log(\bar{\alpha})\) = intercept of the design S-N curve with the log N axis,
- \(m\) = negative inverse slope of the S-N curve,
- \(\left( \frac{t}{t_{\text{ref}}} \right)^k\) = the scale effect with:
  - \(t_{\text{ref}}\) = reference thickness equal 25 mm for welded connections other than tubular joints,
  - \(t\) = pile thickness, \(t = t_{\text{ref}}\) is used for thickness less than \(t_{\text{ref}}\),
  - \(k\) = thickness exponent on fatigue strength as given in DNV RP-C203.

Values of \(\log(\bar{\alpha})\), \(k\) and \(m\) are provided in Table 2.3-2 of DNV-RP-C203 for the relevant S-N curve. More details on the weld type and associated S-N curve are provided in Appendix 1: Classification of structural details of DNV-RP-C203 (see extract in table below). It should be noted that for driving analysis an S-N curve F or F3 is usually considered depending of the presence or not of a backing bar.
6. Pile driving fatigue analysis from a past project

From a past project in the Gulf of Guinea, the following pile driving input data can be summarized:

- Pile is of uniform thickness, that is 42" OD x 2.0" WT with a nominal yield stress of 345 MPa,
- Pile is divided in 4 segments P1 to P4
- Yard welds are made from one side only without a backing element,
- Offshore welds are made from one side only but with a backing element due to the presence of the pile stabbing guide used as an installation aid for installing the next pile section on the previously installed pile element.

6.1. Pile driving results

The driving results of the piles down to the target penetration of 95.5m can be summarized in the following table.

| Depths                | Pile sections being driven | Hammer setting energy (%) | Maximum compressive stress in pile (MPa) | Number of primary blows | Associated secondary stress (MPa) | Number of secondary occurrences |
|-----------------------|----------------------------|---------------------------|-----------------------------------------|-------------------------|----------------------------------|---------------------------------|
| 16.9 m to 23 m        | P1+P2                     | 50                        | 185                                     | 1530                    | 219, 92                          | 119, 1411                       |
| 44.5 m to 50 m        | P1+P2+P3, P1+P2+P3+P4     |                           |                                         |                         |                                 |                                 |
| 71.3 m to 73.4 m      |                            |                           |                                         |                         |                                 |                                 |
| 73.4 m to 77 m        | P1+P2+P3+P4               | 60                        | 202                                     | 1325                    | 88, 1325                         |                                 |
| 23 m to 31 m          | P1+P2                     | 75                        | 225                                     | 3292                    | 158, 109                         | 189, 3103                       |
| 50 m to 58 m          | P1+P2+P3, P1+P2+P3+P4     |                           |                                         |                         |                                 |                                 |
| 77 m to 85 m          |                            |                           |                                         |                         |                                 |                                 |
| 31 m to 44.5 m        | P1+P2                     | 100                       | 254                                     | 4489                    | 53, 72                           | 1200, 3889                      |
| 58 m to 71.3 m        | P1+P2+P3, P1+P2+P3+P4     |                           |                                         |                         |                                 |                                 |
| 85 m to 95.5 m        |                            |                           |                                         |                         |                                 |                                 |
| Total number of blows |                            |                           |                                         |                         | 10636                            |                                 |

Table 1 – Summary of results for pile driving fatigue analysis
From the above table it can be seen that, for each individual maximum compressive stress the blow wave induces secondary compressive or tensile stresses. Sometimes the maximum delta stress is even higher than the main compressive stress but the number of occurrences is low.

6.2. Selection of SCF

Stress Concentration Factors (SCF) to be used have a huge effect on fatigue damages. So, the choice of this factor should be carefully done.

The value of SCF for butt weld from an important oil Company is given in their specifications and is equal to 1.27 considering a misalignment of 10% of pile wall thickness.

From OTC 7453, a formula is given for the calculation of the SCF at butt weld. In our case, with a constant pile thickness and considering a misalignment of 10% of pile wall thickness the value of SCF becomes 1.32 which is slightly greater than the above value of 1.27.

SCF for butt welds can be issued from DNV RP-C203 May 2000 page 18. Considering again a misalignment of 10% of pile wall thickness and a 30°-30° bevel weld, SCF is found to be 1.24 which is slightly less than 1.27.

Hence, at that time, SCF were close to each other ranging from 1.24 to 1.32 in our case.

However, in the next revisions of DNV RP-C203 as from October 2001 page 18, it is said that a misalignment of 10% of pile wall thickness is already inherent in the S-N curve, accordingly $\delta_i + \delta_m$ is reduced by $\delta_0 = 0.1t$ in
equation (3). In that case the value of SCF is reduced to 1.0 considering that $\delta_m$ is equal to 0.1$t$ and $\delta$, is equal to zero (no thickness variation).

6.3. Pile fatigue results

Damage ratios have been calculated as per HSE, AWS and DNV and are shown thereafter.

6.3.1. HSE

For butt welds from one side only with permanent backing (offshore welding), the F curve is used and is obtained from P curve in air with a classification factor of 1.34 which is to be applied on the nominal stress.

SCF considered is 1.27 together with a scale effect, $(t/t_{ref})^k$ with $t_{ref}$ of 16 mm, with pile wall thickness $t$ of 50.8 mm and with the exponent coefficient $k$ of 0.3. $S_o$ and $N_o$ are respectively the stress range and the allowable number of cycles at the change of slope in the S-N curve. $N$ is the allowable number of cycles at stress range $\Delta \sigma$ and $n$ is the applied number of stress cycles.

| HSE : Class 1.34 |
|------------------|
| $t$ | 50.8 mm |
| $t_{ref}$ | 16 mm |
| $k$ | 0.3 |

| LOG K1 | 15.537 |
|--------|--------|
| 12.182 | 3 |

| So | 53 MPa |
|-----|--------|
| $N_0$ | $1.00E+07$ |

**Figure 7 – Fatigue damages as per HSE**

The damage is already greater than 1 without any additional Company safety factor. It can also be seen that damage due to secondary waves is negligible compared to damage caused by primary waves. Hence, we could neglect damage due to secondary waves in the coming pages. If we consider the case of yard weld without permanent backing the classification factor increases to 1.52 and, as expected, the damage increases also and becomes 1.762.

Hence, with HSE rules the damage is high and even greater than 1 due only to pile driving stresses.

6.3.2. AWS D1.1

We use the C1 curve for butt splices, complete joint penetration groove welds, as welded. This curve is issued from Table 2.6 for circular sections. SCF is again taken as 1.27. There is no thickness correction in the specifications and influence, if any, of backing bar is not considered.
Total damage is less than 1 but does not still include Company safety factors. This curve is not much used today because of the absence of thickness correction and differentiation between with or without permanent backing strip.

### 6.3.3. DNV RP-C203

This DNV rules are today commonly used for assessing fatigue damages including pile driving damages.

In the revision of May 2000, calculated SCF was equal to 1.27 considering a misalignment of 10% of pile wall thickness. The SCF reduces to 1.0 in the version from October 2001 as discussed above.

For the case of offshore weld with a permanent backing element, we shall use the F curve for circumferential butt weld. A scale effect, $(t/t_{ref})^k$ with $t_{ref}$ of 25 mm and exponent coefficient $k$ of 0.1. A comparison is made with SCF equals either to 1.27 or 1.0. The case for yard weld without backing strip is also studied.

**Figure 8 – Fatigue damages as per AWS**

**Figure 9 – Fatigue damages as per DNV for two values of SCF and F curve**
It can be seen that damage is reduced by more than 2 by considering a SCF of 1.0 instead of 1.27. Hence, DNV is now less conservative considering the 10% misalignment already included in the S-N curve.

However, using F3 curve has the inconvenience of doubling the damage ratio. For yard welds SCF can also be considered as equal to 1.0 because the butt weld is performed by automatic machines.

7. Conclusions

It can be seen that evaluating pile driving fatigue damages is not an easy task and depends a lot on the input data. This is now becoming a bit easier as some Oil Companies now include in their specifications the DNV RP-C203 but there is still nothing said about what has to be done to evaluate pile fatigue damages.

Nevertheless, the smallest damage ratio of 0.227 using DNV rules is still high and does not comply with safety factor of 10 as required by some Oil Companies. Discussions with Company to reduce this safety factor may be necessary, even if the welds are not accessible and cannot be repaired or inspected, the loads during driving are only temporary and are not present during all field life.

Moreover we shall not forget that, to this pile driving damage, we have to add fatigue damages due to in place conditions. Fortunately fatigue damages due to in-place conditions can be considered negligible by having the butt weld far from where the bending moment in the pile is a maximum.

However, according to A.H. Priest and I.M. Gaunt, it appears that pile driving which induces compressive stresses has no effect on the fatigue life of the butt welds. This could be explained by the initial residual axial stresses, due to process, in the welds which are tensile on the root side and compressive on the toe side of the welds. Hence on the root of the weld the additive compressive axial stresses of driving counterbalance the initial residual axial tensile stresses. On the toe side the axial compressive stresses are increased and yielding occurs. When the driving compressive loads are removed the final residual compressive stresses decreased due to the yielding of the steel. And as there is equilibrium between axial compressive and tensile stresses, tensile residual stresses are also reduced on the root side. This suggests that driving of piles could be a phenomenon that produces stress relief and would be rather beneficial. In that case there would be no need to account for driving fatigue damages in the evaluation of pile fatigue for in-place conditions. Of course, this has to be confirmed with large scale tests.
References

[1] A.H. Priest and I.M. Gaunt, The effect of pile driving on the fatigue of welded tubes. Eighth International Conference on Offshore Mechanics and Arctic Engineering, The Hague, March 19-23, 1989.

[2] Barbour, R.J. and Erbrich C.T., 1994. Analysis of in-situ reformation of flattened large diameter foundation piles using ABAQUS. Proc. UK ABAQUS Users Conf., Oxford.

[3] Aldridge, T.R., Carrington, T.M. and Kee, N.R., 2005. Propagation of pile tip damage during installation. Proc. Int. Conf. Frontiers in Offshore Geotechnics, eds S. Gourvenec and M. Cassidy, Taylor and Francis, 823—827.

[4] Toolan, R.E. and Fox, D.A., 1977. Geotechnical Planning for Piled Foundations for Offshore Platforms. Proc. ICE London, Part 1, 221—243.

[5] Likins, G., Piscsalko, G., Roppel, S. and Rausche, F., 2008. PDA Testing: State of the Art. Proceedings of the Eighth International Conference on the Application of Stress Wave Theory to Piles, Lisbon, Portugal, 395—402.

[6] API, 2000. RP2A, Recommended Practice for Planning, Designing and Installing Fixed Offshore Platforms — Working Stress Design, 21st edition, and subsequent updates, American Petroleum Institute.

[7] ISO, 2007. International Standard ISO 19902. Petroleum and natural gas industries: Fixed steel offshore structures, International Standards Organization.

[8] Stevens, R.S., Wiltzie, E.A. and Turton, T.H., 1982. Evaluating Pile Driveability for Hard Clay, Very Dense Sand, and Rock. Paper OTC 4205, Offshore Technology Conference.

[9] Randolph, M.F. and Murphy, B.S., 1985. Shaft Capacity of Driven Piles in Clay. Paper 4883, Offshore Technology Conference.

[10] Colliat, J.L., Vergobbi, P. and Puech, A., 1993. Friction, Degradation and Set-Up in Hard Clays Offshore Congo and Angola. Paper OTC 7192, Offshore Technology Conference.

[11] Kolk, H.J., 2000. Deep foundations in calcareous sediments. Proc. Int. Conf. Engineering for Calcareous Sediments, ed. K.A. Al-Shafei, Balkema, 2, 313—344.

[12] Rausche, F. and Hussein, M., 2000. Pile driving in calcareous sediments. Proc. Int. Conf. Engineering for Calcareous Sediments, ed. K.A. Al-Shafei, Balkema, 2, 345—359.

[13] PDI, 2010. GRLWEAP (Computer Program for) Wave Equation Analysis of Pile Driving, Version 2010—1. Pile Dynamics Inc.

[14] DNV, 2000, Recommended Practice. DNV-RP-C203, Det Norske Veritas

[15] DNV, 2001, Recommended Practice. DNV-RP-C203, Det Norske Veritas

[16] HSE, Offshore Installations : Guidance on design, construction and certification, Fourth Edition, Section 21, Steel

[17] AWS D1.1:2010, Structural Welding Code-Steel

[18] OTC 7453, Structural Design, Fabrication, and Installation of Offshore Conductor Pipe, G.R. Lang Jr and B.J. Wood, Mobil Research & Development Corp.